ISSN 0280-5731, LMV-Rapport 1999:3

Reports in Geodesy and Geographical Information Systems

GEODESY AND SURVEYING IN THE FUTURE

—

The Importance of Heights

March 15-17, 1999, Gävle, Sweden

PROCEEDINGS

Edited by Mikael Lilje

Organised by
FIG Commission 5
Nordic Commission of Geodesy, Height Determination Group
National Land Survey

FIG

NATIONAL LAND SURVEY

NGK

GEODESY AND SURVEYING IN THE FUTURE

The Importance of Heights

March 15-17, 1999, Gävle, Sweden

PROCEEDINGS

Edited by Mikael Lilje

Organised by
FIG Commission 5
Nordic Commission of Geodesy, Height Determination Group
National Land Survey

PREFACE

FIG Commission 5, NKG working group on height determination together with the National Land Survey of Sweden sponsored and organised the seminar *Geodesy and Surveying in the Future – The Importance of Heights*. The seminar was held the 15-17th of March, 1999, at the National Land Survey, Gävle, Sweden.

The background to the seminar is that we celebrate 25 years of motorised levelling in Sweden in 1999. We started to explore the levelling technique in the beginning of the 70s and produced the first test levelling in 1974. The motorised levelling technique has since then been adopted by several countries all over the world and we have ourselves been using it extensively in the third national precise levelling project.

A major aim with the seminar was to present and discuss as many height determination techniques as possible and preferably to cover them from the scientific to the user application point of view.

FIG is an abbreviation of Féderation Internationale des Géomètres (International Federation of Surveyors). Commission 5 works within the fields Positioning and Measurement and is divided into five different working groups. Information about the Commission can be found on the homepage of the commission (www.lm.se/fig5) or on the FIG homepage (http://www.ddl.org/figtree/). Jean-Marie Becker, Sweden, is chair of FIG Commission 5 and Matt Higgins, Australia, is v. chair. Mikael Lilje, Sweden, is the secretary of the commission.

NKG is the Nordic Commission of Geodesy and is divided into several working groups. One working group deals with Height Determination and is chaired by Jean-Marie Becker.

About 115 persons from over 20 countries participated at the seminar and the atmosphere was very pleasant. Good presentations together with a lot of discussions and combined with a smoothly run seminar made the event to a success. The Organising Committee would like to take this opportunity to thank especially all the speakers but also each and everyone present in Gävle for making the seminar such a pleasant and successful one.

We hope that everyone reading this book will find the work and efforts of the Organising Committee as well as all the presenters worthwhile.

Jean-Marie Becker

FIG Commission 5, chair

CONTENTS

Technical Session 8: How to meet the end-users needs

List of participants

Jean-Marie Becker
National Land Survey of Sweden
801 82 Gävle
Sweden
Tel: +46 26 63 37 28
Email: jean-marie.becker@lm.se

Gunnar Berg
Stadsbyggnadskontoret
801 84 Gävle
Sweden
Fax: (+46) 26 17 83 79
Email: gunnar.berg@gavle.se

Harry Bergqvist
MMPG
Metria
801 82 Gävle
Sweden
Fax: (+46) 26 65 41 67
Email: harry.bergqvist@lm.se

Jerker Boman
Student
Royal institute of Technology
Drottning Kristinas Väg 30
100 44 Stockholm
Sweden

Colin Bray
Ordnance Survey Ireland
Phoenix Park
Dublin 8
Ireland
Fax: +353 1 820 4156
Email: cbray@osi.ie

György Busics
College for Surveying and Land Management
1-3. Pirosalma Street
PO Box 52
8002 Székesfehérvár
Hungary
Fax: +36 22 327 697
Email: bgy@geo.cslm.hu

Paul A Cross
Department of Geomatic Engineering
University College London
London
WC 1E 6BT
United Kingdom
Fax: +44 171 380 0453
Email: paul.cross@ge.ucl.ac.uk

Szabolcs Csepregi
College for Sureying and Land Management
1-3. Pirosalma Street
PO Box 52
8002 Székesfehérvár
Hungary
Fax: +36 22 327 697

Phil Davies
Geodetic Adviser
Room C510
Ordnance Survey of Great Britain
Romsey Road
Southampton SO16 4GU
United Kingdom
Fax: (+44) 1703 792 351
Email: pdavies@ordsvy.gov.uk

Leendert J. Van Dijk
Sokkia B.V.
PO Box 1292
1300 BG Almere
The Netherlands
Tel: (+31) 36 5322880
Fax: (+31) 36 532 62 41
Email: leendert.vandijk@sokkia.nl

Alan Dodson
Institute of Engineering Surveying and Space Geodesy
University park
Nottingham
NG7 2RD
United Kingdom
Fax: +44 115 951 3881
Email:
Alan.Dodson@nottingham.ac.uk

Henri Duqenne
CNAM
ESGT
1, Boulevard Pythagore
F-72000 Le Mans
France
Tel: +33 2 43 43 31 46
Fax: +33 2 43 43 31 02
Email: Henri.Duquenne@esgt.cnam.fr

Francoise Duquenne
CNAM
ESGT
1 Boulevard Pythagore
Universite du Maine
72000 Le Mans
France
Tel: (+33) 243 433 146
Fax: (+33) 243 433 102
Email:
Francoise.Duquenne@esgt.cnam.fr

Pelle Edmark
Metria
801 82 Gävle
Sweden
Email: pelle.edmark@lm.se

Britt-Marie Ekman
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 37 47
Fax: +46 26 61 06 76
Email: britt-marie.ekman@lm.se

Artu Ellmann
Technical Inspection Department
Estonian National Land Board
Mustamäe tee 51
Box 1635
EE0006 Tallinn
Estonia
Fax: +372 66 50 604
Email: artu@maaamet.ee

Lars E Engberg
Geodetic Research Division
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 30 37
Fax: +46 26 61 06 76
Email: lars.e.engberg@lm.se

Björn Engen
Geodetic Institute
National Mapping Authority
N-3500 Hönefoss
Norway
Tel: +47 321 18 100
Email: bjorn.engen@gdiv.statkart.no

Andreas Engfeldt
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 30 00
Fax: +46 26 61 06 76
Email: andreas.engfeldt@lm.se

Patrik Ericsson
Student
Inst för tillämpad Naturvetenskap
Mitthögskolan
871 88 Härnösand
Sweden
Fax: +46 611 861 60

Per-Ola Eriksson
Metria
801 82 Gävle
Sweden
Tel: +46 26 63 37 23
Fax: +46 26 65 41 67
Email: per-ola.eriksson@lm.se

Bengt Eurenius
National Land Survey
801 82 Gävle
Sweden
Fax: +46 26 65 40 95
Email: bengt.eurenius@lm.se

Thomas Fischer
Nestle&Fischer
GmbH & Co. KG
D-72280 Dornstetten
Germany
Fax: +49 7443 240169
Email: tfischer@nedo.de

Yang Gao
Department of Geomatics Engineering
University of Calgary
2500 University Dr. N.W.
Calgary, Alberta T2N 1N4
Canada
Fax: +1 403 284 1980
Email: gao@ensu.ucalgary.ca

Erik W. Grafarend
Geodätisches Institut Universität Stuttgart
Geschwister-Scholl-Str. 24 D
D-70174 Stuttgart
Germany
Tel: +49 711 121 3390
Fax: +49 711 121 3285
Email: grafarend@gis.uni-stuttgart.de

Hans Granegård
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 37 45
Fax: +46 26 61 06 76
Email: hans.granegard@lm.se

Armi Grinstein
General Manager
Armi Grinstein Geodetic Eng. Ltd.
2, Habonim ST.
Ramat-Gan 52462
Israel
Fax: +972 3 575 01 27

Hansbert Heister
Institut fur Geodäsie
Universität der Bundeswehr, Munchen
Werner-Heisenberg-Weg 39
D-85577 Neubiberg
Germany
Fax: +49 89 60 04-39 04
Email: heister@glabse.bauv.unibw-muenchen.de

Matt Higgins
Department of Natural Resources
Locked Bag 40
Coorparoo Mail Centre
Brisbane QLD 4151
Australia
Fax: +61 7 3891 5168
Email: matt.higgins@dnr.qld.gov.au

Olav Hoftuft
StatensKartverk
Geodesidivisjonen
Kartverksveien 21
3500 Honefoss
Norway
Fax: +47 32 11 81 01
Email: olav.hoftuft@gdiv.statkart.no

Clas-Göran Hurtig
Luftfartsverket
ANS/ASP
601 79 Norrköping
Sweden
Fax: (+46) 11 19 22 46
Email: mailto:claes-goran.hurtig@ans.lfv.se

Norbert Höggerl
Bundesamt fur Eich- und Vermessungswesen
Schiffantsgasse 1-3
A-1025 Wien
Austria
Fax: (+43) 1 211 76 2224
Email. ABT.V1@BEV.GV.AT

Johannes Ihde
Bundesamt fur Kartographie und Geodäsie
Karl-Rothe-Str. 10-14
D-04105 Leipzig
Germany
Fax: (+49) 341 563 44 15
Email: ihde@leipzig.ifag.de

Hilmar Ingensand
Institut für Geodäsie und Photogrammetrie
ETH
Hönggerberg HIL D 43.3
CH-8093 Zürich
Switzerland
Fax: +41 1 633 11 01
Email: ingensand@geod.ethz.ch

Casper Jepsen
National Survey and Cadastre
Rentemestervej 8
DK-2400 Copenhagen
Denmark
Tel: +45 35 87 50 52
Email. caj@kms.dk

Lotti Jivall
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 37 40
Fax: +46 26 61 06 76
Email: lotti.jivall@lm.se

Magnus Johansson
Student
Royal institute of Technology
Drottning Kristinas Väg 30
100 44 Stockholm
Sweden

Sven-Inge Johansson
Vägverket
Box 4202
171 04 Solna
Sweden
Fax: +46 8 28 37 07
Email: sven-inge.johansson@vv.se

Bo Jonsson
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 37 38
Fax: +46 26 61 06 76
Email: bo.jonsson@lm.se

Ulf-Göran Jonsson
Swedish Hydrographic Office
601 78 Norrköping
Sweden
Tel: (+46) 11 19 11 45
Fax: (+46) 11 13 39 03

Reiner Jäger
Fachhochschule Karlsruhe
Moltkestrasse 30
D-76133 Karlsruhe
Germany
Fax: +49 721 925 2591
Email: reiner.jaeger@fh-karlsruhe.de

Juhani Kakkuri
Finnish Geodetic Institute
Geodeetinrinne 2
02430 Masala
Finland
Fax:+358 9 29555 200
Email: Juhani.Kakkuri@fgi.fi

Janis Kaminskis
State Land Service of Latvia
National Surveying Centre
Geodesy Department
11 Novembra krastmala 31
Riga
Latvia LV1050
Fax: (+371) 722 7858
Tel: (+371) 721 3471
Email. kaminski@main.rsf.rtu.lv

Michel Kasser
Ecole Superieure des Geometres et Topographes
1 Bd Pythagore
72 000 Le Mans
France
Fax: +33 2 4343 3102
Email: kasser@esgt.cnam.fr

Oddgeir Kristiansen
StatensKartverk
Geodesidivisjonen
Kartverksveien 21
3500 Honefoss
Norway
Fax: +47 32 11 81 01
Email: oddgeir@gdiv.statkart.no

Jussi Kääriäinen
Finnish Geodetic Institute
Geodeetinrinne 2
02430 Masala
Finland
Fax:+358 9 29555 200
Email: jussi.kaariainen@fgi.fi

Michel Le Pape
Institut Geographique National
2, avenue Pasteur
94165 Saint-Mande Cedex
France
Fax: +33 1 43 98 84 50
Email: Michel.Le-Pape@ign.fr

Jiri Lechner
GEODIG
VUGTK
Czech Republic
Fax. +420 2 68 73 64

Martin Lidberg
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 37 42
Fax: +46 26 61 06 76
Email: martin.lidberg@lm.se

Mikael Lilje
National Land Survey of Sweden
801 82 Gävle
Sweden
Fax: +46 26 61 06 76
Email: mikael.lilje@lm.se

Åke Lindberg
KTH-Bygg
Box 88
801 02 Gävle
Sweden
Tel: (+46) 26 14 78 01
Email: lindberg@bmg.kth.se

Jan Lindström
MMPG
Metria
801 82 Gävle
Sweden
Fax: (+46) 26 65 41 67
Email: jan.lindstrom@lm.se

Thomas Lithén
National Land Survey
801 82 Gävle
Sweden
Email: thomas.lithen@lm.se

Sven-Åke Ljungberg
KTH-Bygg
Box 88
801 02 Gävle
Sweden
Tel: (+46) 26 14 78 01
Email: ljungberg@bmg.kth.se

Sten Lundén
Banverket
781 85 Borlänge
Sweden
Fax: +46 243 44 56 17
Email: sten.lunden@hk.banverket.se

Sören Lundqvist
Swedesurvey
801 82 Gävle
Sweden
Email: soren@swedesurvey.se

Robert Lundström
Student
Inst för tillämpad naturvetenskap
Mitthögskolan
871 88 Härnösand
Sweden
Fax: +46 611 861 60

Bo Malmström
University of Gävle
Institutionen för Teknik
801 76 Gävle
Sweden
Tel: (+46) 26 64 85 00
Fax: (+46) 26 64 88 28
Email: bo.malmstrom@hig.se

Björn Melin
Swedish Hydrographic Office
601 78 Norrköping
Sweden
Tel: (+46) 11 19 11 46
Fax: (+46) 11 13 39 03

Matthias Menzel
Carl Zeiss ena GmbH
Tatzendpromenade 1a
07740 Jena
Germany
Fax: (+49) 3641 644 201
Email: menzel@zeiss.de

Roy Morgan
Ordnance Survey Northern Ireland
Email:
MBRAND.OSNI@NICS.GOV.UK

Stig-Göran Mårtensson
Inst för tillämpad naturvetenskap
Mitthögskolan
871 88 Härnösand
Sweden
Fax: +46 611 861 60
Email: s-g.martensson@tnv.mh.se

Christina Nenzelius
MMPG
Metria
801 82 Gävle
Sweden
Fax: (+46) 26 65 41 67
Email: christina.nenzelius@lm.se

Andreas Nilsson
Student
Inst för tillämpad Naturvetenskap
Mitthögskolan
871 88 Härnösand
Sweden
Fax: +46 611 861 60

Kurt Nilsson
MMPG
Metria
801 82 Gävle
Sweden
Fax: (+46) 26 65 41 67
Email: kurt.nilsson@lm.se

Britta Nordin
Student
Inst för tillämpad naturvetenskap
Mitthögskolan
871 88 Härnösand
Sweden
Fax: +46 611 861 60

Anders Nordquist
MMPG
Metria
801 82 Gävle
Sweden
Fax: (+46) 26 65 41 67
Email: anders.nordquist@lm.se

Dan Norin
Stockholms Stadsbyggnadskontor
Box 8314
104 20 Stockholm
Sweden
Fax: +46 8 508 27 213
Email: Dan.Norin@SBK.stockholm.se

Ann-Katrin Nyqvist
Student
University of Gävle
Institutionen för Teknik
801 76 Gävle
Sweden
Email:
TK96ANT@STUDENT.HIG.SE

Poul Nörgård
The Danish Road Directorate
Thomas Helstedts Vej 11
8660 Skanderborg
Danmark
Fax. (+45) 86 52 17 01
Email: pon@vd.dk

Joakim Ollén
National Land Survey of Sweden
801 82 Gävle
Sweden
Email: joakim.ollen@lm.se

Maria Olsson
Luftfartsverket
ANS/ASP
601 79 Norrköping
Sweden
Fax: (+46) 11 19 22 46
Email: maria.olsson@ans.lfv.se

Georges Pauchard
Mesures et Systems
6, rue des Jardins
60500 Chantilly
France
Fax (+33) 344 57 4658

Christophe Pichot
Dassault-Sercel N.P.
16, Rue de Bel-Air
B.P. 433
44474 Carfquefou Cedex
France
Fax: (+33) 2 40 30 59 92
Tel: (+33) 2 40 30 59 00
Email:
christophe.pichot.dsnp@sercel.fr

Hans Peter Plag
StatensKartverk
Geodesidivisjonen
Kartverksveien 21
3500 Honefoss
Norway
Fax: +47 32 11 81 01
Email:
hans.peter.plag@gdiv.statkart.no

Markku Poutanen
Finnish Geodetic Institute
Geodeetinrinne 2
02430 Masala
Finland
Fax. +358 9 295 55 200
Email: Marku.Poutanen@fgi.fi

Bo-Gunnar Reit
National Land Survey
801 82 Gävle
Sweden
Tel: +46 26 63 37 35
Fax: +46 26 61 06 76
Email: bo-gunnar.reit@lm.se

Markus Rennen
Institute of Geodesy and
Photogrammetry
Technical University of Braunweig
Gausstr. 22
38106 Braunschweig
Germany
Fax (+49) 531 391 7499
Tel: (+49) 531 391 7485
Email: m.rennen@tu-bs.de

Gethin Roberts
IESSG
University Park
Nottingham
NG7 2RD
England
Fax: +44 115 95 13881
Email:
gethin.roberts@nottingham.ac.uk

Andres Rudja
Finnish Geodetic Institute
Geodeetinrinne 2
02430 Masala
Finland
Fax: +358 0 295 55 200
Email: andres.rudja@fgi.fi

Bengt Rystedt
National Land Survey
801 82 Gävle
Sweden
Email: bengt.rystedt@lm.se

Veikko Saaranen
Finnish Geodetic Institute
Geodeetinrinne 2
02430 Masala
Finland
Fax: + 358 9 295 55200
Email: veikko.saaranen@fgi.fi

Olle Samuelsson
Student
Inst för tillämpad Naturvetenskap
Mitthögskolan
871 88 Härnösand
Sweden
Fax: +46 611 861 60

Ulf Sandgren
National Land Survey
801 82 Gävle
Sweden
Email: ulf.sandgren@lm.se

Jaakko Santala
Institute of Geodesy and Cartography
University of Technology
Box 1200
02015 Helsinki
Finland
Fax: +358 9 465 077
Email: Jaakko.Santala@hut.fi

Klaus E Schmidt
National Survey and Cadastre
Rentemestervej 8
DK-2400 Copenhagen
Denmark
Tel: +45 35 87 50 52
Email. kes@kms.dk

Felix Schneider
Leica GeoSystems AG
CH-9435 Heerbrugg
Switzerland
Tel: +41 71 727 33 86
Tel: +41 71 727 31 31
Fax: +41 71 727 47 02
Email: Felix.Schneider@leica-geosystems.com

Klaus Peter Schwarz
Department of Geomatics Engineering
The University of Calgary
2500 University Dr. N.W.
Calgary, Alberta
Canada T2N 2N1
Tel: (+1) 403 220 73 77
Fax: (+1) 403 284 1980
Email: schwarz@ensu.ucalgary.ca

Takao Seto
SOKKIA Co
260-63 Yanagicho
Hase
Atsugi
Kanagawa
243-0036
Japan
Fax: +81 462 47 1731
Email: seto@sokkia.co.jp

Jaroslav Simek
GEODIG
VUGTK
Czech Republic
Fax. +420 2 68 73 64

Kjell Staflund
Täby Kommun
Tekniska Kontoret
183 80 Täby
Sweden
Fax: (+46) 8 758 08 96
Email: kjell.staflund@taby.se

Gershon Steinberg
Survey of Israel
1, Lincoln St.
Tel Aviv 652 20
Israel

John Sundsby
StatensKartverk
Geodesidivisjonen
Kartverksveien 21
3500 Honefoss
Norway
Fax: +47 32 11 81 01
Email: john.sundsby@gdiv.statkart.no

Runar Svensson
MMPG
Metria
801 82 Gävle
Sweden
Fax: (+46) 26 65 41 67
Email: runar.svensson@lm.se

Anders Söderberg
Student
Inst för tillämpad naturvetenskap
Mitthögskolan
871 88 Härnösand
Sweden
Fax: +46 611 861 60

Mikko Takalo
Finnish Geodetic Institute
P O Box 15
FIN-02431 Masala
Finland
Tel: +358 9 295 55 200
Email: mikko.takalo@fgi.fi

Henrik Tengbert
Swedish Hydrographic Office
601 78 Norrköping
Sweden
Tel: (+46) 11 19 11 27
Fax: (+46) 11 13 39 03

Goran Turk
University of Ljubljana
Faculty of Civil and Geodetic Engineering
Jamova 2
1000 Ljubljana
Slovenia
Fax +(386) 61 176 8629
Email: gturk@fagg.uni-lj.si

Wolffried Wehmann
Hochschule fur Technik und Wirtschaft Dresden
Faculty of Surveying and Cartography
PF 120701
D-01008 Dresden
Germany
Fax: +49 3 51 4 62 21 91
Email: wehmann@vermessung.htw-dresden.de

Stieg Vennström
National Land Survey
801 82 Gävle
Sweden
Fax: (+46) 26 61 06 76
Email: stieg.vennstrom@lm.se

Martin Vermeer
Finnish Geodetic Institute
Geodeetinrinne 2
02430 Masala
Finland
Fax: + 358 9 295 55200
Email: mv@fgi.fi

Olav Vestøl
StatensKartverk
Geodesidivisjonen
Kartverksveien 21
3500 Honefoss
Norway
Fax: +47 32 11 81 01
Email: olav.vestol@gdiv.statkart.no

Hans-Erik Wiberg
National Land Survey
801 82 Gävle
Sweden
Email: hans-erik.wiberg@lm.se

S Stampe Villadsen
National Survey and Cadastre
Rentemestervej 8
DK-2400 Copenhagen
Denmark
Tel: +45 35 87 50 52
Email. sv@kms.dk

Bertold Witte
Geodätisches Institut der Universität Bonn
Nussallee 17
D-53115 Bonn
Germany
Tel: (+49) 228 73 26 20
Fax: (+49) 228 73 29 88
Email: witte@uni-bonn.de

Technical Program

Sunday 14th of March 1999

Steering Committee meeting FIG Commission 5

Monday 15th of March 1999

Registration at NLS

Opening ceremony: Chair J-M Becker

J-M Becker, chair FIG Commission 5. Introduction

K. P. Schwarz, President IAG: The changing world of geodesy and surveying

J. Ollén: Director General National Land Survey: The National Land Survey of Sweden today and tomorrow.

J-M Becker: History and evolution of height determination techniques

Slideshow with pictures of Sweden

Technical Session 1: Advance in techniques and instrumentation for terrestrial height determination: Chair M. Kasser

H. Ingensand: The evolution of digital levelling techniques - limitations and new solutions

T. Seto: Results of test and experiments with SDL30 digital level

J. Kakkuri: The rapid precise levelling system - dream or reality

G. Pauchard: New automated real-time 3-D total station, possibilities and limitations

M. Menzel: The development of levels at Carl Zeiss during the past 25 years with special focus on the optical level NI 002 and the digital Level DiNi® 11

H.F. Wennström: Information about the exhibition at Karteum - NLS

Technical Session 2: Applications of modern terrestrial height determination techniques. Chair H. Ingensand

W. Wehmann: Experience with various digital levels in both motorised and conventional on foot levelling in east Germany

M. Lilje: The production line used in the third precise levelling in Sweden

T. Lithén: Motorised Trigonometric Levelling (MTL) for precise levelling – the Swedish tests and results

K.E. Schmidt: The use of Motorised Trigonometric Levelling (MTL) in Denmark

Technical Session 3a: Heighting using satellites or other techniques: Chair P. A. Cross

E.W. Grafarend: Sensitive high-speed railway track control

H. Duqenne: Levelling by GPS - the state of the art in France

M. Higgins: Heighting with GPS - overview of possibilities and limitations

R. Jäger: State of the art and present developments of a general approach for GPS-based height determination

Technical Session 3b: Heighting using satellites or other techniques: Chair M. Higgins

P. A. Cross: Accuracy achievable by GPS in practical engineering and surveying applications

Y. Gao: Multi-sensor systems for height determination

G. Roberts: Height control of construction plant by GPS and GLONASS

Opening ceremony of the technical exhibition at Karteum by Joakim Ollén (Director General of NLS)

Tuesday 16th of March 1999

Technical Session 4a: Quality Control: Chair H. Heister

S. Csepregi: Some questions on precise height measurements

M. Kasser: Error sources in high precision levelling - How to minimise their effect on the heights!

T. Fischer: Manufacturing of levelling rods - Methods, Calibration, Accuracy, etc

M. Takalo: On behaviour of invar rods with aluminium frame used in third precise levelling of Finland.

Technical Session 4b: Standards: Chair J. Kääriäinen

H. Heister: Checking, testing and calibration of geodetic instruments

J. Simek: Present state of standardisation in surveying Profession

J-M Becker: The new updated ISO standards concerning levelling instruments

Technical Session 5: Different aspects: Chair Y. Gao

A. Dodson: Ocean tide loading effects on heights

G. Turk: Approximation of a local geoid surface by artificial neural network

V. Saaranen: Computation of postglacial land uplift from the three precise levellings in Finland.

B-G Reit: The influence of heights on datum transformations

Technical Session 6: Height Networks: Chair K. P. Schwarz

J. Ihde: Status of the European height systems UELN and EUVN

J. Simek: Heights and vertical control in the Czech Republic: evolution and present state

M. Poutanen: Use of GPS in unification of vertical datums and detection of levelling network errors

G. Steinberg: The future of vertical geodetic control

Technical Session 7: How to fulfil the User needs: Chair M. Lilje

P.O. Eriksson: The requirements on the monumentation of height benchmarks

S. Villadsen: The implementation of a new common height datum in Denmark

M. Kasser: Best use of different height determination techniques

SPECIAL SESSION: Chair J-M Becker

K. P. Schwarz: Some Trends in Geomatics: Multi-sensor systems and global georeferencing

Wednesday 17th of March

Technical Session 8: How to meet the End-users needs? : Chair B Engen

A. Ellman: Different solutions adopted to modernise the height networks in the Baltic countries

P. Nörgård: High Precision GPS makes challenges to realisations of Reference Systems.

M. Le Pape: Direct access to the French digital Height Databank via Minitel

D. Norin: Requirements from urban users of heights, examples from the city of Stockholm

J. Kääriäinen: Precise levellings in Finland

Technical Session 9: National reports: Chair A. Ellman and S Villadsen

T. Österberg: Experiences of technical co-operation in Zambia and Mozambique

S. Villadsen: National report on the activities in Denmark

B. Engen: The Norwegian levelling activities today and in the future. Problems and expectations

L. E. Engberg: The Swedish geodetic networks today and in the future.

G. Busics: The past and the future of the levelling networks in Hungary

Panel Discussion and Summary of the seminar:

J-M Becker, K. P. Schwarz, M. Kasser, H. Ingensand, H Heister, S Villadsen, M Higgins, J Kääriäinen

Technical visits and demonstrations at NLS:

- Practical demonstration of Motorised Levelling
- Demonstration of the control centre of SWEPOS (the national network of permanent reference stations)
- The National Historical Map Centre - Swedish maps from several centuries
- The production of modern digital maps.
- Laser interferometer comparator in action - calibration of levelling rods

Introduction

by

Prof. Jean-Marie Becker

National Land Survey of Sweden
Chairman of FIG Commission 5:"Positioning and Measurement"
Chairman of Nordic Geodetic Commission Height Determination Working Group

Ladies and Gentlemen, dear Colleagues, welcome to Sweden to our Jubilee Seminar in Gävle at the National Land Survey of Sweden.

Mesdames & Messieurs, Chers Confrères, Bienvenue à notre Seminaire ici à Gävle.

Meine Damen und Herren, geehrten Kollegen, herzlich willkommen zu unserem Seminar hier in Gävle.

As you can see on the picture in front of you the title of this Seminar focused on three subjects namely:

- Geodesy and Surveying in the Future.
- The importance of Heights.
- 25 years of Motorised Levelling.

For **25 years** ago, in spring 1974 the first Swedish Motorised Levelling team started its activities. Each field season since 1979 between tree and six teams have been levelling step by step through Sweden from south to north in order to cover the country with a modern precise height network. When this project will be finished Sweden will have more than 50 000 benchmarks of highest quality that means with a precision of millimetre level.

The celebration of the 25 years anniversary of Motorised Levelling activities in Sweden give us the unique opportunity to focus and concentrate our attention during the coming three days on one of the most important geodetic component namely **"Heights"**.

A look in our national geodetic database at the National Land Survey shows that we have about fifteen time more monumented height reference points (100 000) than trigpoints. In the daily engineering survey operations the needs of heights are also the most predominant, this can be seen at each construction project.

Heights are of interest both for the **Geodesist**, the **Surveyor** and common people that mean both in **Geodesy and Surveying**.

A more practical illustration of the importance of heights can easily be found around us in the nature when looking in which direction water runs. The answer is a known fact for everybody namely **never upward**. Consequently in all engineering work this physical reality has to be respected if we want to eliminate disagreeable surprises (like pumpstations).

Often levelling work is considered as the simplest geodetic operation. I remember one of my early chiefs who's opinion about levelling was very simplified, for him it was only "a question of + and –".

During 25 years of work with levelling questions I have tried and still not succeeded to implement fully his simplified theory.

Today some of my colleagues dream about a "magic black box" solving all geodetic questions, some of them believe that GPS is already is this global problem solver. The question is " How near are they from the reality?" Perhaps this dream never will be reality!

Nevertheless Your response to our invitation, your presence here (more than 110 participants from 24 nations) and the high number of papers (more than 50 exclusive 19 refused for lack of time) show that we are many working with this subject. Heights are again on the top of interest for the people involved in Surveying and Geodetic activities.

The goal for this seminar is to give a complete overview and the state of the art concerning height determination questions. Each one of you (scientist or technician, teacher or practitioner, engineer or field assistant) will, contribute with its own knowledge and experience from scientific and/or an practical point of view, to make this seminar successful

The different technical sessions will take you through all the steps (scientific, technical and financial) from field operations to the final heights needed for the End-Users different applications.

The Panel discussion on Wednesday will be a good opportunity to discuss and hopefully to answer to you're remaining questions.

The Organising Committee has not forgotten that an important factor for optimal brainwork is that natural human needs also have to be satisfied. Therefore we offer you good "Swedish International" food at lunchtime followed each evening by a splendid dinner with beer and wine.

The wine will certainly stimulate you in the discussions and perhaps give you the faculty to see double. I hope that it will contribute to give us an answer about how near our dreams are from the reality.

We have several qualified keynotes speakers who will introduce us into our subject: the first keynote speaker is Prof. Klaus Peter Schwarz from Calgary, Canada and President of the International Association of Geodesy. He will be followed by Joakim Ollen, Directo General of the National Land Survey of Sweden and finally by myself.

May I invite Prof. Klaus Peter Schwarz to give us his lecture on the subject: The changing World of Geodesy and Surveying.

The Changing World of Geodesy and Surveying

K.P. Schwarz
Department of Geomatics Engineering
The University of Calgary
Calgary, Alberta, Canada T2N 1N4
E-mail: schwarz@ensu.ucalgary.ca

Summary

Change in the field of geodesy and surveying has been rapid during the past twenty-five years. It was driven by major advances in space geodesy resulting in new measurement systems. These systems have had a profound effect on the practice of geodesy and surveying and it is likely that their impact will become even broader and more pronounced during the next decade. Four major trends that developed over the past twenty-five years are briefly discussed in this paper and their impact on the major tasks of geodesy - the representation of the Earth's surface and its gravity field - are evaluated. The paper concludes with an outlook on a possible integration of geodetic techniques and data into an Earth Observing System that will more accurately describe the evolution of the Earth in time and space.

1. Twenty-five Years in Retrospective

Understanding change requires interpretation of the present in terms of the past. In this first chapter some developments of the past 25 years are interpreted as technological trends in the field of geodesy and surveying. Trends express continuity over a given time period. They offer therefore a limited amount of predictability which is dependent on the linearity of the observed phenomenon. Looking at the state of geodesy and surveying in 1974, few of the developments mentioned below were predictable in terms of the trends dominant during the period 1949-1974. The development was not linear and, only in hindsight, are we able to see some of the trends. This should be kept in mind when talking about 'The Changing World of Geodesy and Surveying'. Understanding change does not mean that the future can be predicted by extrapolating the past. Understanding change can, however, prepare us for the future. Golo Mann's remark that *"those who do not know the past will not get a handle on the future"* should serve as a reminder and an antidote against too much blue-eyed optimism.

What then are some of the major changes in our field at the end of this century? Disregarding general technological trends, such as the still rapid development of computer technology, the trend towards miniaturization of sensors and systems, and the rapid emergence of complex information systems, there are a number of specific developments which are changing the world of geodesy and surveying as we have known it. Four of them will be discussed here:

- The implementation of a time-varying reference frame of unprecedented accuracy which for the first time, allows the measurement of global and regional changes of the Earth and their modeling in space and time.

- The capability to operate the measurement systems directly in the reference frame using satellite orbits as the link and thus eliminating the need for networks and dense ground control point monumentation.
- The capability to solve the vertical datum problem by a combination of satellite and airborne gravimetry.
- The increasing trend towards integrated kinematic measurement systems with high data rates and the resulting changes in automated data acquisition, modeling and algorithm development.

Before discussing these trends in more detail, conventional approaches to solving the task of geodesy will be briefly reviewed in order to provide a framework for the rest of the paper.

2. Views on How to Solve the Task of Geodesy

More than 100 years ago, Helmert defined geodesy as the science of measuring and mapping the Earth's surface. Although methods have considerably changed since then, the definition is still useful if one adds the temporal variations of the Earth to the definition. Figure 1 illustrates the change in measurement systems and techniques that has taken place in the past 25 years and that is still in progress. Especially the development of kinematic techniques of mapping the Earth surface and gravity field are remarkable and will be discussed further in chapter 6. Helmert's seemingly simple definition has been interpreted in rather different ways by the groups involved in surveying and mapping. Part of this difference came about because of differences in the surface itself. Those who measured the ocean surface - by far the largest part of the Earth's surface - obviously faced different problems than those who measured the land surface. However, even among those who measured on land, there were vast differences in concept and approach. Figures 2 to 4 indicate some of these differences.

Figure 2 illustrates the view of the surveyor/geodesist who typically considers the measurement of the Earth's surface as a point positioning problem. The accurate determination and monumentation of points on the surface of the Earth is therefore seen as the major task. In order to express these points in a consistent coordinate system over larger parts of the Earth's surface, networks are established and the datum problem must be solved. Once this has been done, the network points can be used for point densification in local areas. The resulting representation of the surface by a more or less regular cluster of points is considered as sufficient. Mapping is done as pointwise mapping. In a way, the concept behind this approach is that the higher the point accuracy, the better the mapping. This is true for pointwise mapping, but obviously not for surface mapping. Simple interpolation between network points will for instance create large errors in a topographic map. Thus, the accuracy of the surface representation will not be uniform. In addition, although networks may stretch over a large part of the Earth's surface, they are globally disconnected when established by conventional procedures. This means that the datum problem cannot be solved without extraterrestrial measurements. This method has therefore to be supplemented by other techniques in order to solve the task of geodesy as defined by Helmert.

Figure 3 illustrates the view of the photogrammetrist who considers the measurement of the Earth's surface as an imaging problem. It is solved by deriving a model of the surface from digital or photographic images. In this case, patches of the Earth's surface are actually measured and mapped in accordance with Helmert's definition. The concept behind this method is that the surface of the Earth can be presented by pixels measured in projected images. The smaller the pixel size and the more uniform the geometry, the better the mapping. In this case, the accuracy is more or less uniform across the image and interpolation of specific image features is possible with high accuracy, once the image has been properly georeferenced. This is done by solving the datum problem using geodetic ground control in the survey area. Comparing the view of the surveyor/geodesist with that of the photogrammetrist shows that they are essentially complementary. The surveyor/geodesist provides highly accurate point positions in an adopted reference system which then can be used by the photogrammetrist to georeference measurements and solve the datum problem for the precise local maps derived from images.

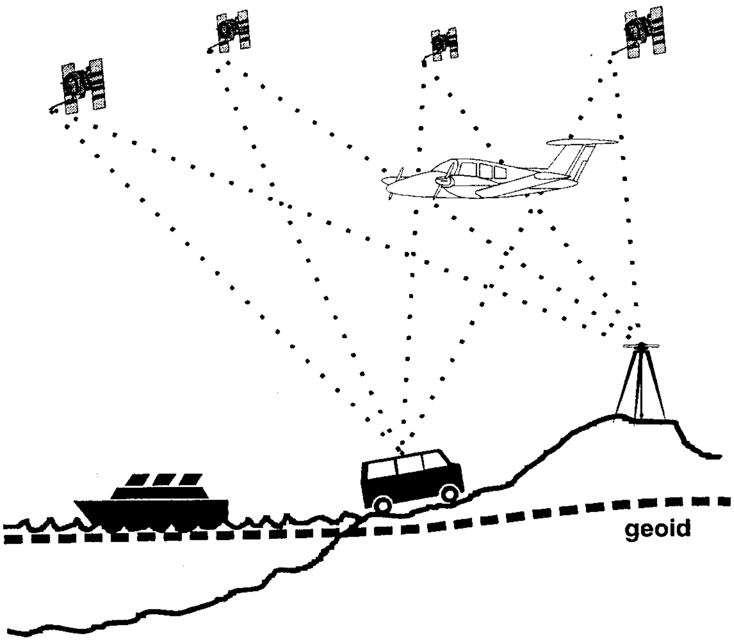
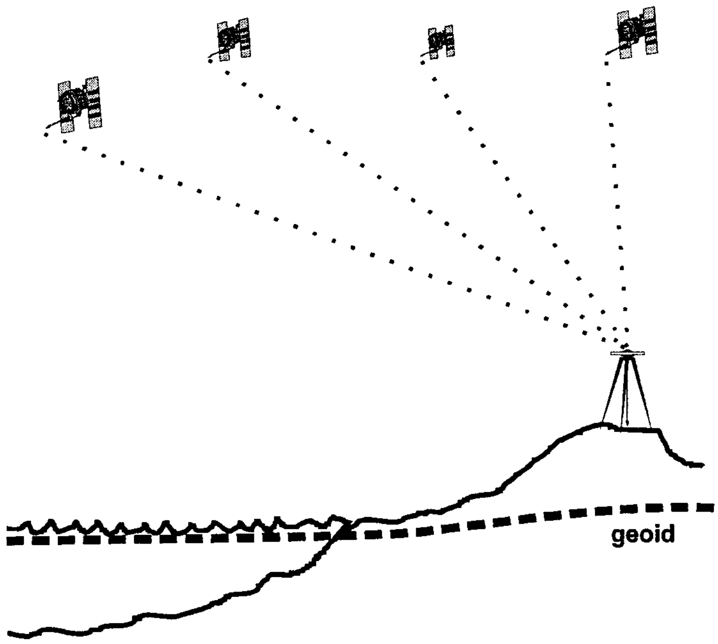


Figure 1: Measuring the Earth's Surface by Static and Kinematic Systems



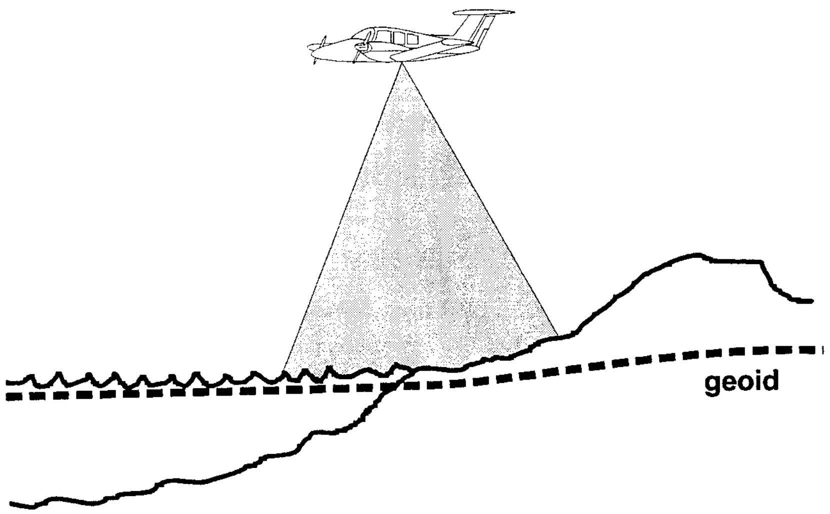
The surface of the Earth is defined by point measurements.

The higher the accuracy of point determination, the better the mapping.

Figure 2: Point Positioning – The Surveyor's View

Figure 4 illustrates the view of the geodesist who considers the surface of the Earth as a boundary surface to be determined by gravimetric measurements. This corresponds closely to the definition of geodesy given by Bruns in 1878 stating that "the task of geodesy is the determination of the potential function W(x,y,z)". The connection to the positioning problem is given by the fact the W is defined as a function of position (x,y,z). Thus, once W(x,y,z) is determined with sufficient accuracy, the Earth's surface can in principle be derived and the mapping problem solved. The practical problem in this approach is the determination of the potential function from discrete measurement (gravity anomalies, deflections of the vertical, etc). Data density and consistency will strongly influence the accuracy with which the surface can be determined. In other words, the denser the gravimetric data, the better the surface mapping. Currently, the measurement accuracy is still orders of magnitude better than the interpolation accuracy. In addition, the datum problem has to be solved. Thus, on a global scale the best models are still about two orders of magnitude away from the accuracy level that would make them consistent with the point positioning accuracy currently achieved by GPS and other satellite methods.

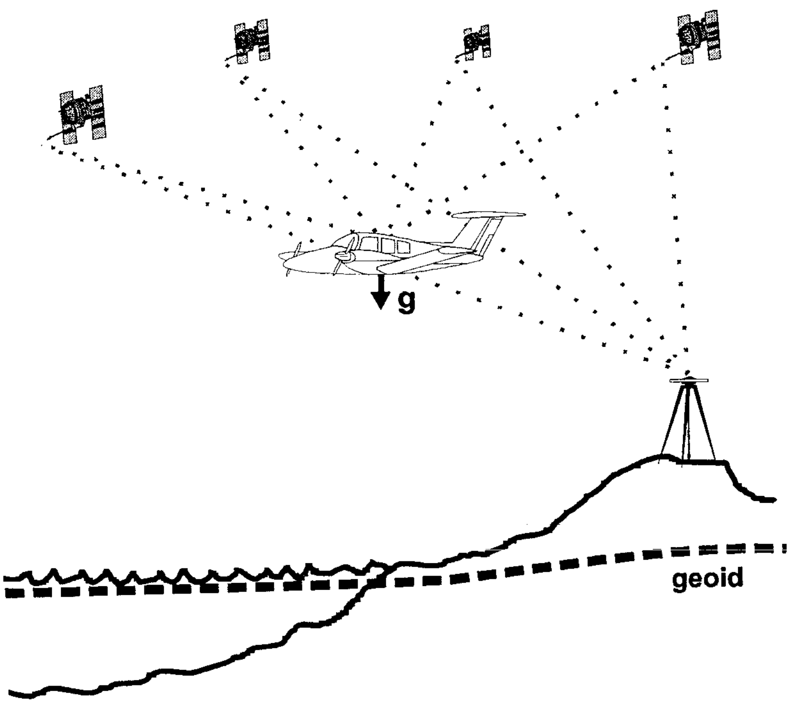
All three classical solution approaches have one drawback in common: they approximate the global situation by patching together those pieces of the Earth's surface which are covered by measurements. This leaves big gaps generated by ocean areas and by poorly surveyed parts on the continents. To improve the patchwork, a consistent global frame is needed and a methodology to transform isolated surface patches onto this frame. Looking at the current efforts in Europe to patch together the different reference systems, different DEMs, and diverse geoid patches, gives an appreciation of the size of the task for just one well-surveyed continent.



The surface of the Earth is defined by pixels measured in projected images.

The smaller the pixel size, the better the mapping.

Figure 3: Image Modelling – The Photogrammetrist's View



The surface of the Earth is defined by a gravimetric boundary value problem.

The denser the gravity measurements, the better the mapping.

Figure 4: Boundary Surface Determination – The Geodesist's View

3. Reference Frames and the Solution of the Datum Problem

One of the advantages of applying space methods to geodesy is the establishment of a highly accurate reference frame for positioning. The centre of mass of the Earth, as well as the direction of the axes of the conventional terrestrial frame can be established with an accuracy that, in a relative sense, is at the part per billion level and is thus superior to most practically applied positioning techniques. Comparing this to the best available global frame 25 years ago shows that reference frame implementation has been improved by more than two orders of magnitude. For much of the following discussion, the detailed technical background can be found in the proceedings of the recent IGGOS (1998) symposium.

A Conventional Terrestrial Reference Frame (CTRF) is implemented by tying the frame definition to the positions of fundamental observing stations which are continuously measured. The measurements are either made with respect to satellites or with respect to extraterrestrial sources. If only one technique is used for the determination of the coordinates, small biases may remain in the frame definition. Comparing independently determined conventional reference frames offers therefore a means to detect and eliminate such biases. Such comparisons have been made by the International Earth Rotation Service (IERS), established by the IAG, and have shown that the origins of these different reference frames agree at the level of a few centimeters and that the directions of the axes agree at the level of a few milliarcseconds. Thus the stability of current global reference frames is such that time changes in the coordinates of the fundamental stations have to be taken into account. The IERS has therefore added a plate tectonic motion model to its ITRF94 reference frame, making it a four-dimensional frame. It is planned to extend this model to include regional motions, once estimates of sufficient accuracy are available; for an overview see Blewitt et al (1997). Twenty-five years ago that would have been impossible because the existing reference frames did not have the accuracy nor the stability to reliably determine such small motions by measurement.

Besides the ITRF94 which uses a combination of observational techniques to determine the reference frame, there are a number of reference frames which make use of one observational technique only. The best known is the WGS84 which is based on the GPS tracking station network and uses observations to GPS satellites only. The IGS network operated by the International Geodesy and Geodynamics Service, established by the IAG, is another GPS-based reference frame with a much larger tracking station network, almost 200 stations by now. It will be used to measure and model regional motions. Similar reference systems exist for satellite laser techniques and VLBI. Each of these techniques has its own set of tracking stations to define the reference frame. Each of these reference frames can be considered as an implementation of the underlying reference system. Figure 5 illustrates in a schematic way the weakness of implementing a reference frame by only one type of measurements using GPS as an example. The GPS tracking network is indicated by three widely spaced stations on the surface of the Earth (triangle). Each of these stations can be determined by range measurements to at least four satellites, indicated by the heavy lines between one of the network stations and five of the satellites. The underlying assumption of this approach is that the satellite positions at the time of measurement are precisely known. This is not the case and therefore the range measurements are also used to improve the satellite orbits. This can be done reliably if the positions of the tracking stations are accurately known. Thus, one ends up with a typical bootstrap procedure: To improve orbits, accurate positions of tracking stations are needed - to get tracking station coordinates, precise orbits are needed. This problem is solved in an iterative way by matching the accuracy of tracking station coordinates with the accuracy of orbit determination. The results are excellent because of the measurement precision and the continuous observation schedule. However, because of the bootstrapping operation, small systematic errors in scale and orientation may remain in a reference frame derived in this way. It is therefore important to compare and improve reference frames derived from only one technique.

To do that, fundamental stations with more than one observational technique are included in the network to derive transformation parameters from the specific network to the ITRF94. Figure 6 shows in a schematic way how the reference frame derived from GPS observations could be improved in its orientation accuracy by VLBI measurements. The tracking network is again indicated by the triangle of fundamental stations on the Earth which are now simultaneously observed by VLBI and GPS. The dotted lines indicate VLBI measurements between the tracking stations and the quasar sources. They provide precise orientation of the CTRF within an inertial frame of reference. This technique has been used for the WGS84 for instance and the results are shown in Table 1 which gives the translations in cm and the rotations in milliarcseconds (mas); for details see Slater and (1997). Both the transformation parameters and their standard deviations indicate that the differences between the two systems is at the level of a few parts per billion.

Table 1: How Good is the GPS Reference?
Transformation of WGS 84 (G873) on ITRF94

Origin	Orientation of axes
$\Delta x = -0.1 \pm 2.9$ cm	$\varepsilon = 0.0 \pm 0.3$ mas
$\Delta y = -0.2 \pm 2.3$ cm	$\psi = 0.4 \pm 0.2$ mas
$\Delta z = 0.1 \pm 1.4$ cm	$\omega = 0.6 \pm 0.4$ mas
Scale Factor Accurate to:	
$s = -0.5 \pm 0.2$ parts per billion	

With a reference system of this accuracy and stability, the datum problem for positioning can be considered as solved for all practical requirements. The only remaining problem is the transformation of the existing network information onto this global reference frame. As the ongoing EUREF and REUN campaigns in Europe show, this is not a trivial task. While the global reference frame is a consistent three-dimensional coordinate system, this cannot be said for the reference systems used in the conventional network approach. Horizontal and vertical networks are essentially disconnected. They have few or no overlapping points and are based on different datums and are therefore not consistent. To transform the vertical network information to the global reference frame, the geoid is needed with high global accuracy. This will be further discussed in chapter 5. To transform the horizontal network information to the global reference frame, network distortions have to be eliminated first, before the relatively simple geometric transformation can be applied. Network distortions are due to a variety of causes, such as the observational procedure, the insufficient knowledge of the geoid used for reductions, and geodynamic changes of the Earth's surface during the long time periods over which networks were established. Whether these transformations can be determined with an accuracy sufficient to reliably transform existing networks into the global reference, will be answered by the ongoing investigations. If the answer is positive, an enormous amount of valuable observational material will be preserved for scientific investigations. Even in this case, however, their practical value as ground control will be very limited due to reasons discussed in chapter 4.

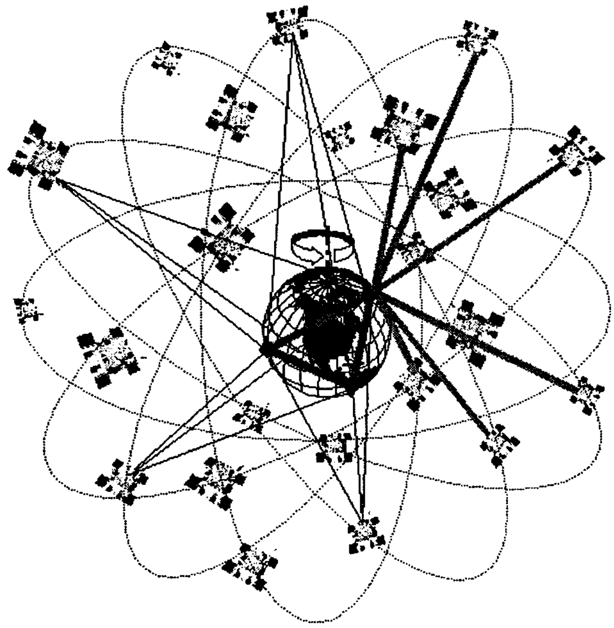


Figure 5: The GPS Bootstrap Operation: Tracking Stations vs. Orbits

4. Positioning in the Age of GPS - Satellites Replace Ground Control

Although GPS is now extensively used for a broad spectrum of survey tasks, it is widely seen as a highly accurate relative positioning method. Interstation vectors are the output of differential GPS methods and, in this sense, GPS is viewed as a sophisticated replacement of a total station for longer distances. In this scenario, a dense network of ground control points is still needed to tie the output of the receiver to the existing network. What is lost in this view of GPS positioning is the fact that the receiver output is directly connected to the global reference system by way of satellites. In principle, it should therefore be possible to determine globally referenced positions without access to networks or dense ground control.

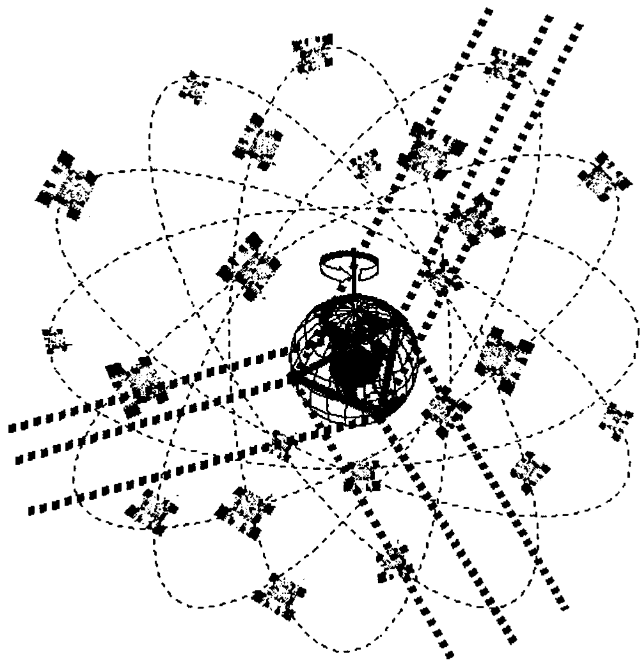


Figure 6: Connecting GPS to VLBI

This means that in the long run networks and monumented control will lose their importance because it will be possible to establish accurate global positions within a relatively short observation period. Part of this future is already with us in both static and kinematic positioning. Currently, the accuracy of the results is not good enough for all applications. To make it the standard method for most applications, it will be necessary to improve the availability of precise orbits, to better model or eliminate atmospheric effects, to improve the clock technology, and to further advance real-time algorithm development. Many of these improvements are discussed in the NAPA/NRC (1995) report where specific recommendations are given. Further details can be found in the technical literature. As an example likely developments in the area of orbital modeling will be briefly discussed in the following.

A major difference between GPS and traditional positioning methods is the replacement of ground control by sky control. Instead of tying into monumented control points one links into satellites which, in their orbital positions, carry accurate reference system information with them. This is possible because satellites are tied by measurement to the ground tracking stations which define the reference system. The accuracy of the orbital information depends on its age and on the density of the tracking network. The age of the orbit information is important because the broadcast ephemeris is predicted for a 36 hour period, computed from previous satellite observations. Their accuracy gets poorer with time which means that the accuracy of the reference information stored in the satellites deteriorates with time. This will affect real-time results, but not post-mission processing which can make use of orbital information that was derived from measurements during the observation period. While broadcast ephemeris may contain errors of up to 2m, post-mission orbits are typically better than 0.2m. The accuracy of broadcast ephemeris could be considerably improved by shortening the prediction period. Studies have shown that this is not a computational problem any more. The information could be available with relatively short time lags. The remaining problem is efficient data distribution. It may be possible to upload the orbital information at a higher rate than the current 12 hour rate. Otherwise, some way of automatically updating the receivers would be needed.

Another way of improving the long-term prediction accuracy is the use of GPS crosslinks, i.e. of direct measurements between GPS satellites. Figure 7 shows this concept in a schematical way. Instead of using only measurements from the Earth to the satellites for orbit determination, measurements between satellites could be used to create a kinematic network on the GPS-satellite envelope. While Earth-satellite observations are optimal in fixing the radial orbit component, between-satellite observations would strengthen the along-track and across-track components. Thus, the ground tracking network would be supplemented by a sky tracking network. Technically, the capability for crosslink measurements is available in the Block IIR GPS satellites and can be activated, once enough of these satellites are in orbit. It is interesting to note that in such an approach the separate orbital planes, resulting from the gravity field model employed, are tied together by geometric measurements, essentially defining a potential surface at satellite altitude.

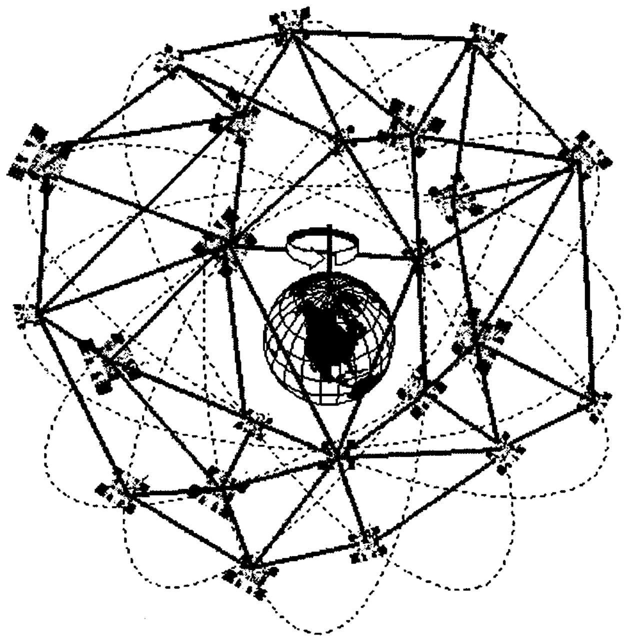


Figure 7: Skynet from GPS Crosslinks

The current trend towards the development of wide-area networks or active control networks is an intermediate step between relative positioning which requires dense ground control point information (DGPS), and absolute positioning which is based on satellite orbits only and does not require ground control for the measurement process. Compared to conventional control networks, the station distribution of wide-area networks is much sparser. These stations transmit orbit information and atmospheric corrections for the area covered by the network to improve real-time static and kinematic positioning. All active stations are at the same time permanent tracking stations and are tied into a global reference frame, such as the IGS. They can therefore be considered as a high-accuracy regional representation of the global reference frame. To which extent this accuracy can be transferred to the receivers operating within such a wide-area network depends largely on the station spacing, the accuracy of the transmitted information, the measurement mode (static or dynamic) and the operational procedures applied. It is likely that such networks will be operated for a considerable time to come. They will also prepare the way for precise absolute GPS positioning by improving orbital and atmospheric modeling techniques and pioneering data transmission to large numbers of users.

The emphasis in this chapter has been on possible developments in GPS positioning. It should not be misunderstood as an advertisement for GPS as a panacea for all positioning ills. GPS, as all other positioning methods, has advantages and drawbacks. Some of the advantages have been discussed above. Limitations are 'line-of-sight' problems between satellite and receiver which will be especially serious in urban centres, forested areas, and in steep mountainous terrain. Thus, other methods will not only continue to exist, but will be more economical and more effective in numerous situations. It will be the task of the practitioner to select the right positioning tools for a given task.

5. Towards a Solution of the Vertical Datum Problem - The Decade of Gravity Satellites

As outlined in chapters 2 and 3, the reference surfaces in the conventional approach are not consistent. Horizontal coordinates refer to the ellipsoid, while height coordinates refer to the geoid. This is somewhat surprising because the measurement systems, theodolites and levels, both refer to the local astronomic frame and, thus, the geoid should be the surface of choice. It was not used as a reference for horizontal coordinates, however, because measurements could be reduced to the ellipsoid by deriving deflections of the vertical from astronomic observations. Since computations were much simpler on the ellipsoid, the methodology was not changed, even when a global representation of the geoid became available. On the other hand, in leveling the line of sight is essentially parallel to the equipotential surface and thus almost parallel to the geoid. Therefore, the height differences are very close to orthometric height differences which are defined with respect to the surface of the geoid. To transform such height differences into ellipsoidal height differences, the geoidal undulations along the leveling line must be known. This is usually not the case and it is the main reason why two different reference surfaces came about.

When GPS was introduced as a three-dimensional positioning system, all three coordinates became available in a consistent reference frame which could be either Cartesian or curvilinear. Usually an ellipsoid was chosen as the curvilinear reference surface and, thus, a direct comparison between the GPS-derived coordinates and the conventional horizontal coordinates was possible. It was not possible, however for ellipsoidal heights and orthometric heights. To transform one height system into the other, an accurate geoid representation was needed in the measurement area. The situation is shown in Figure 8 where, in first approximation, the orthometric height H is the difference of the ellipsoidal height h and the geoidal undulation N. To transform the GPS-derived height into an orthometric height of equal accuracy, the geoid representation had to be accurate to a few centimeters. This is still not the case in many parts of the world. On a global scale the height transformation problem remains therefore an unsolved problem. The best global geoid models are not better than 1-2 m in areas with poor gravity coverage and between 0.3 an 0.5m in areas with good gravity coverage. Thus, the CTRF can be defined with an accuracy of a few centimeters by GPS, it cannot be transformed, however, with the same accuracy into a global reference frame with an orthometric height system. To solve the vertical datum problem, the geoid must be globally known with an accuracy of a few centimeters. In that case, the CTRF will be consistent independent of the height system used.

A number of different techniques are currently used to determine the global geoid model. They are shown in conceptual form in Figure 9. Each technique contributes to a specific part of the gravity spectrum. Because of the attenuation of gravity with height, the spectral range resolved by every technique is dependent on the height of the sensor above the attracting masses. Therefore, measurements on the surface of the Earth or airborne measurements typically give better short-wavelength resolution than satellite measurements. The only exception is satellite altimetry which determines the geoid from direct measurements to the sea surface. Its wavelength resolution mainly depends on the size of the footprint. To resolve the whole spectrum, all techniques have to be combined. For the long-wavelengths the analysis of satellite orbit perturbations is still the most important method. Satellite altimetry resolves long and medium wavelengths over the oceans if a good model for sea surface topography is available. Mean gravity values cover the medium wavelength range on land. Finally, densely spaced point gravity measurements on land allow the resolution of short wavelengths. Absolute gravimetry is used on selected points to guarantee measurement consistency. Data from all these techniques are used for current global geoid models.

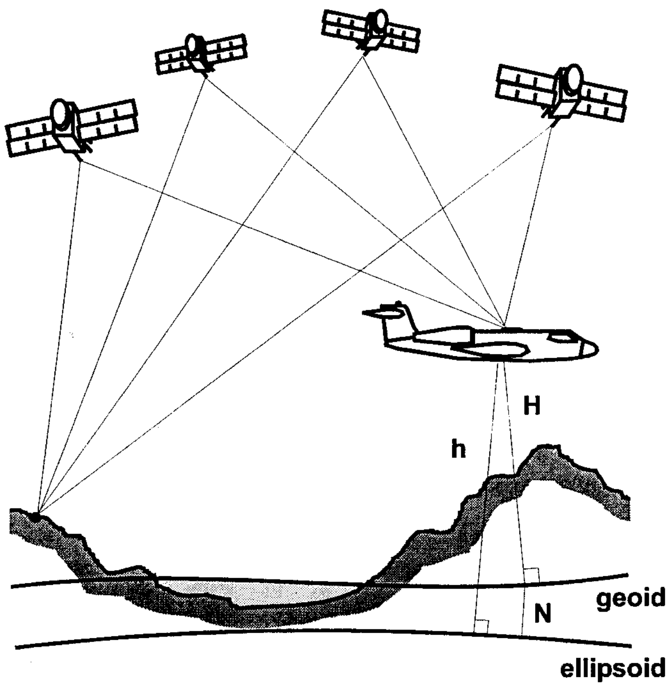


Figure 8: Heights in the Age of GPS – The Datum Problem (N)

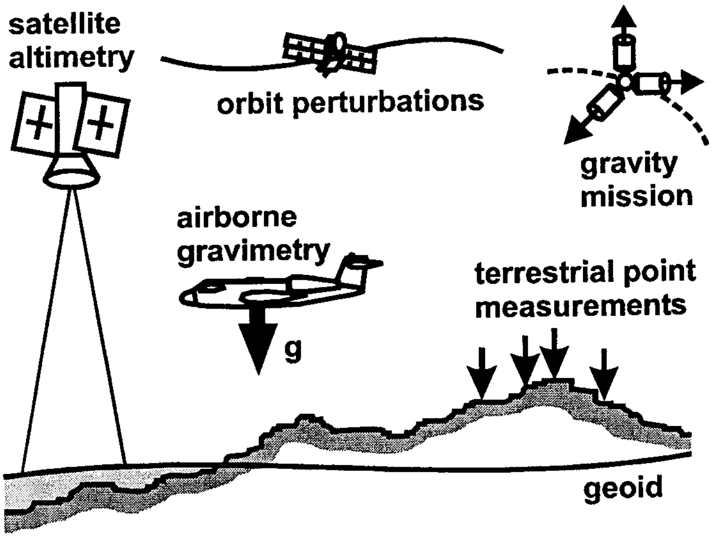


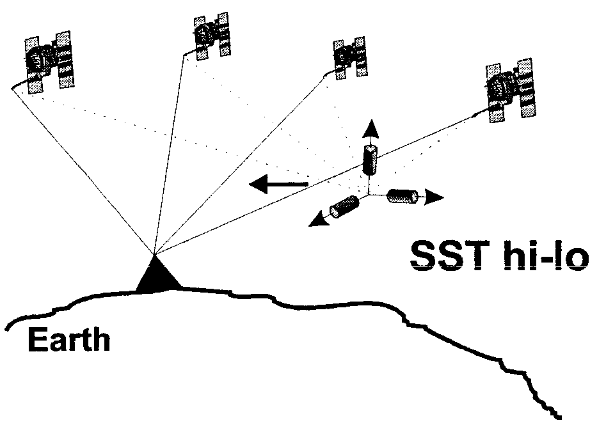
Figure 9: How is N Determined – Current and Future Gravity Methods

To improve current geoid models, new measurement techniques are needed. The most promising ones are airborne gravimetry and gravity satellite missions. The first is a local or regional technique, the second a global technique. In airborne gravimetry the acceleration output of DGPS and INS is differenced, resulting in filtered gravity along flight profiles. This method covers areas of up to 1000 km by 1000 km with gravity profiles and thus resolves half wavelengths between 8 km and 500 km. Dedicated gravity satellite missions use low-orbiting satellites to resolve the gravity spectrum to half wavelengths of about 80 km in the best case and about 300 km in the worst case. The two methods are therefore complementary with airborne methods covering the high frequency spectrum which cannot be resolved by satellite methods and part of the medium frequency spectrum where satellite methods are weak, and satellite methods covering the long and medium spectral ranges.

Currently three specific gravity satellite missions have been proposed, two of which are in an advanced stage, see Ilk (1998) for details. They are shown in schematic form in Figure 10. The first is the microsatellite CHAMP which will be launched this year by Germany and which will operate in a high-low mode. This means that the low-orbiting CHAMP satellite will be tracked by GPS satellites, thus eliminating one major error source, namely atmospheric effects. The perturbation analysis of the CHAMP satellite orbit will be supported by the output of an accelerometer triad on the satellite which will allow a better separation of non-gravitational forces. It is expected that this mission will improve the current global solutions by better decorrelating the medium wavelengths. It will not add decisively, however, in terms of minimum wavelength resolution. The second planned mission is GRACE, which will be launched in 2002 by the USA and will use a satellite-to-satellite tracking technique to resolve the gravity field spectrum. The distance between two low-orbiting satellites will be monitored by an interferometric microwave link. Variations in the measured range will be used to detect temporal variations in the gravity field spectrum and to improve its minimum resolution to half wavelengths of about 150 km. The third mission GOCE is planned by ESA and is scheduled to be launched in 2005. It will use satellite gradiometry to directly measure gravity gradients over a very short base in the satellite. The minimum wavelength resolution could be as low as 80 km. If all three missions go ahead, the complete gravity spectrum to half wavelengths of about 80 km and its major temporal variations will be determined. The combined solution would provide a much better global resolution of the gravity field and especially of the geoid than is currently available. The next decade would then rightfully be called the decade of gravity satellites.

Figure 11 shows the impact of the two new measurement techniques, airborne gravimetry and satellite gravimetry, on the accuracy of global geoid determination. Figure 11a compares four possible scenarios where the dark column indicates the worst case in each scenario and the white column the best case in each scenario. Starting with the currently best global model, the EGM 96, global standard deviations range from about 0.4 m to 1.5 m. As mentioned before these differences are mainly due to the differences in gravity coverage in different parts of the world. The second scenario shows the impact of the gravity satellite missions only. The range of values is much smaller now, between 0.35 m and 0.5 m, and is mainly due to the difference between the optimistic and the more guarded predictions. The accuracy in this case is more or less uniform over the globe. It would not be sufficient, however, to give the geoid transformation with centimeter accuracy. The third scenario shows the combination of the current global model with airborne gravimetry. It gives slightly better results than the previous scenario and has the advantage that it could be implemented right now. The difference between the best and the worst scenario is again due to the difference in EGM 96 accuracy in different parts of the world. The final scenario is the combination of airborne and satellite gravimetry which clearly gives the best results and achieves the accuracy required for height transformation. This means that the required accuracy in the geoid representation will most likely be reached in the next five to seven years, but only in areas where airborne gravity has been obtained or consistent ground gravity coverage is available.

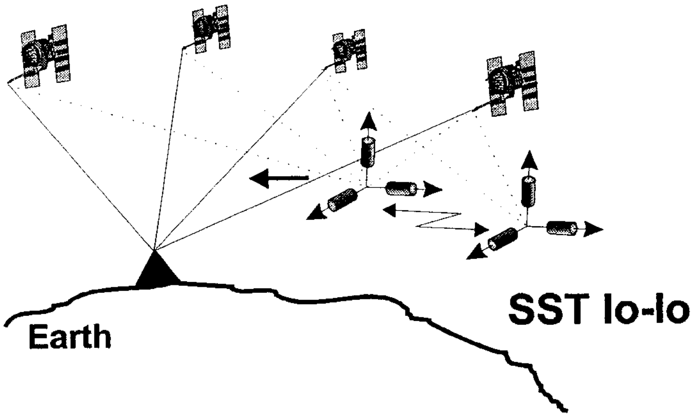
Figure 11b gives best (dotted) and worst (solid) accuracy projections for the next six years assuming that the planned satellite missions are on schedule. Some improvements of the current global models can be expected around the middle of 2000 when the CHAMP data are integrated into the global solution. After that, improvements will be mainly due to the maturing of airborne gravimetry, until GRACE data come on line in about 2003. This will result in major improvements in geoid accuracy because of the better wavelength resolution. GOCE data will add to the high and medium frequency spectrum and, together with airborne gravity data, finally provide the accuracy required in the height transformation.



Orbit perturbation analysis

GPS tracking of satellite in high-low mode

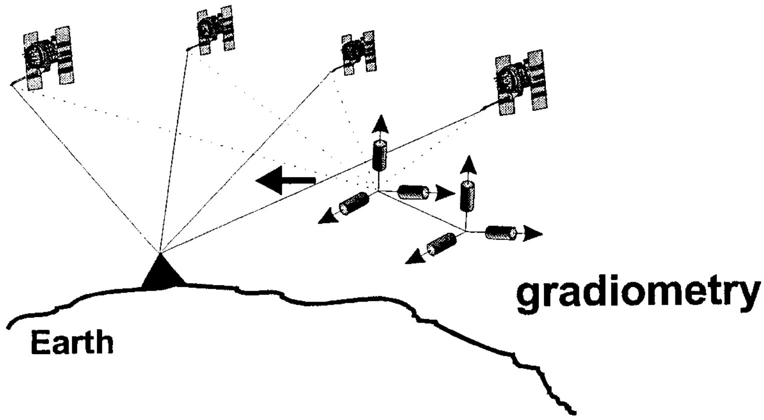
CHAMP (GER) - 1999



Satellite-to-satellite tracking

Interferometric microwave link between two low satellites

GRACE (US) - 2002

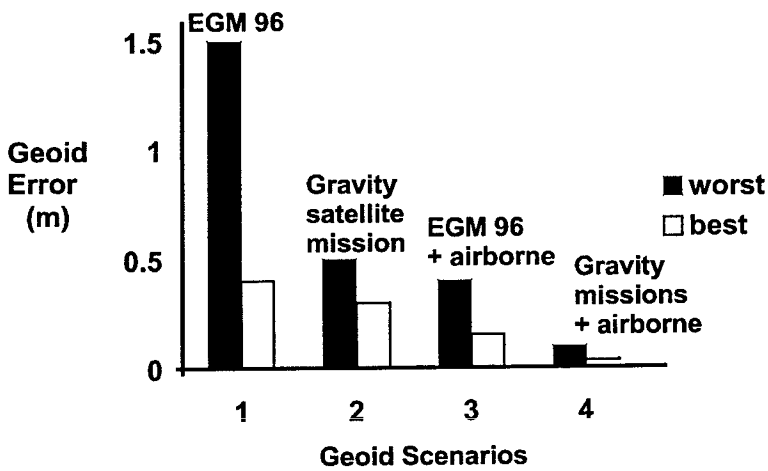


Satellite gradiometry

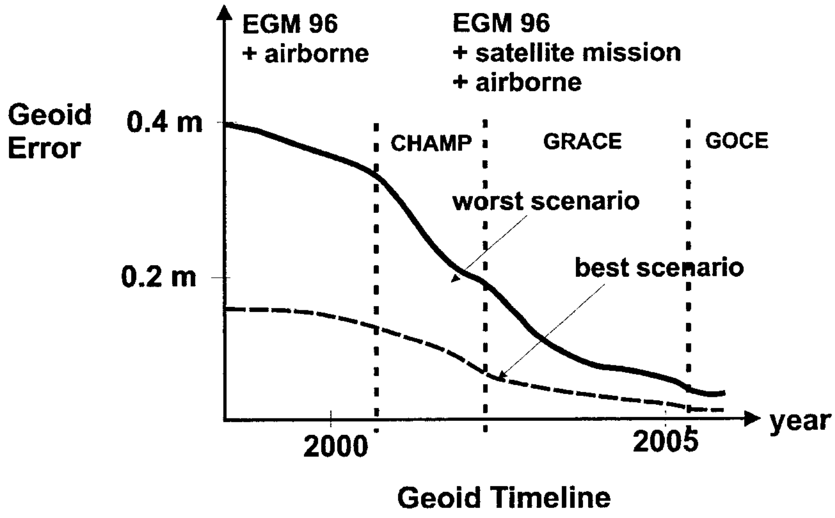
Direct measurement of gravity gradients

GOCE (EU) - 2005

Figure 10: The Decade of Gravity Satellites



(a) Near-Future Scenarios



(b) A Timeline for Geoid Accuracy

Figure 11: The Future of Geoid Determination

6. A Systems Approach to Helmert's Definition - Integrated Kinematic Mapping Systems

In chapter 2, three different views of interpreting Helmert's definition of geodesy have been outlined and some of their advantages and shortcomings have been pointed out. The resulting measurement and processing techniques, i.e. point positioning, photogrammetric mapping, and geoid determination, have been considered as essentially independent, even if their results were often combined in post mission. By combining the three methods, it is possible to come up with an integrated system to measure and map the Earth's surface that maximizes the advantages that each method offers without being affected by their drawbacks. It thus solves the problem contained in Helmert's definition. Such an integrated system can be designed in a number of different ways. The following conceptual discussion of an integrated airborne imaging system should therefore be seen as only one of a number of possible realizations. For more details and some results, reference is made to Schwarz (1998).

There are a number of theoretical and practical reasons why such an integration is advantageous. First of all, a highly accurate global reference frame now exists which can be accessed everywhere by using a GPS receiver as the measurement tool. Since GPS receivers work in kinematic mode, there is no reason to separate the positioning process from the imaging process. By operating in DGPS kinematic mode, with one receiver on the aircraft and one on the ground, there is no need to first establish control positions on the ground which then have to be identified in the images.

Instead, the perspective centre of the photogrammetric camera is determined by DGPS at the moment of exposure. This provides the first three parameters of exterior orientation in an accurate global reference frame (WGS 84). The other three parameters describing the orientation of the camera at the moment of exposure can be obtained by integrating an Inertial Measuring Unit (IMU) with DGPS and the camera. This has two major advantages. First, it is now possible to give each individual image its full set of exterior orientation parameters which means that any two images with overlapping image content can be directly used for mapping part of the Earth 's surface in a consistent coordinate frame. Thus, there is in principle no need for designing photogrammetric blocks and corresponding adjustment procedures to solve the problem. Second, such a system could also solve the vertical datum problem in an elegant way without the need for additional instrumentation. By differencing the output of the IMU and the DGPS, gravity at flight level can be determined from which a relative local geoid can be derived at ground level. By combining it with global information, as described in chapter 5, the transformation problem between ellipsoidal and orthometric heights can be solved. Thus, all measurements that are needed to map the Earth's surface in a consistent global frame can be taken from the same airborne platform. This will not only result in a much more homogeneous data acquisition process, but will also produce a much more efficient data processing procedure.

It has been mentioned already that an integrated kinematic mapping system is not restricted to photogrammetric techniques, nor to an airborne platform. Digital cameras have been used with a land vehicle-based system, and effective use of airborne geoid determination has been made in deriving orthometric DEMs by interferometric SAR. Figure 12 illustrates the latter application. Other systems use laser scanners, sometimes in conjunction with digital cameras, to solve the surface mapping problem.

In all these systems, the data acquisition and processing procedures are remarkably different from conventional methods. Because of the emphasis on point positioning and the type of equipment available for implementing it, conventional survey methods have always been sparse data techniques requiring considerable observational skill and attention to procedure. Since sparse data problems are best solved by least squares adjustment, this became the dominant, and in many cases the only, estimation method used in geodesy. All of this hardly applies to the new measurement systems. Instead of sparse data, redundancies in static positioning are enormous and data compression techniques are much more important in imaging than sparse data techniques. Bandpass filtering, wavelet methods and multi-scale estimation seem to offer much better solutions to these problems than least squares. Because of the large number of redundancies, not all of the data will be stored in the future and efficient and reliable methods of real-time data processing will replace current procedures. Similarly, observational skills have already now been largely eliminated from the measurement process and given way to automated procedures of real-time data checking. Because of the limited amount of automation currently implemented, much still depends on the knowledge of the system operator about the measurement process. It can be expected, however, that more of this know-how will be built into the software and human decision making and expertise will more and more shift to the planning and managing aspects of the problem. Since kinematic mappings systems either are or will be fully digital in the future, the pressure to produce results as fast as possible will result in much more emphasis on real-time data processing. It is conceivable, therefore, that real-time mapping systems for specific applications, such as forest fires, oilspill monitoring, etc, are a distinct possibility in the not too distant future.

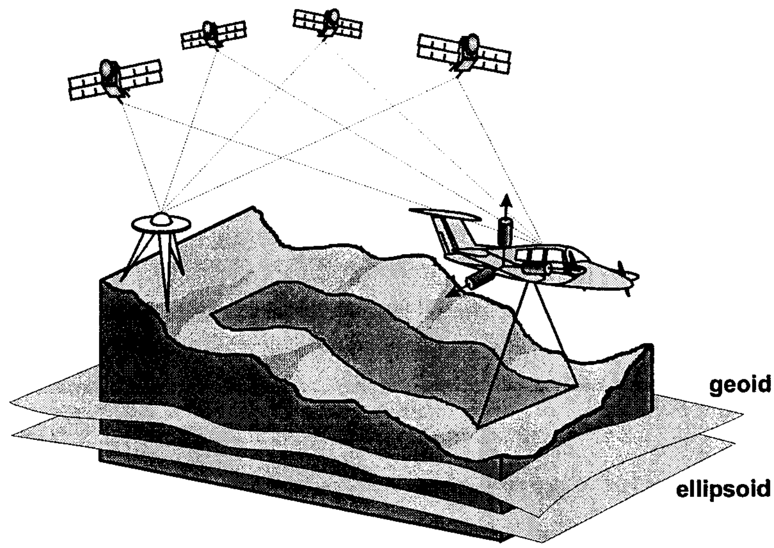


Figure 12: Interferometric SAR with a Geoid Reference

7. Towards an Integrated Geodetic and Geodynamic Observing System

In chapter 3, the establishment of an accurate Conventional Terrestrial Reference Frame (CTRF) and a corresponding motion model have been presented as one of the major contributions of space techniques to geodesy. The CTRF provides a framework in which spatial and temporal variations of the Earth can be precisely measured. How can these measurements be used in an enlarged concept of geodesy?

A couple of years ago, Rummel (1998) published a paper in which he proposed an integration of all geodetic data and techniques, conventional as well as space based, into a Global Integrated Geodetic and Geodynamic Observing System (GIGGOS). Such a system was meant to focus all current geodetic activities in such a way that they would become identifiable as geodesy's contribution to international science. The diagram presented as Figure 13 shows the major components of such a program and indicates the interactions that define it as one system. The following summary of some of the main characteristics of such a system is based on Rummel's original paper.

The four components, indicated as Frame, Earth rotation, Geometry and Kinematics, and Gravitational Field, will be briefly discussed. At the centre of this system is a well-defined and reproducible global terrestrial frame which provides the reference for the observing systems and a framework for modeling Earth processes. Its accuracy and stability affects the accuracy with which the other three components can be modelled. The establishment and maintenance of such a reference frame will be done by a combination of space techniques, such as VLBI, SLR, LLR, GPS, DORIS, PRARE. Closely related to the frame definition is the determination of Earth rotation as the integrated effect of all angular momentum exchange inside the Earth, between land, ice, hydrosphere and atmosphere, and between Sun, Moon, and planets. The measurement systems are the same as for the frame determination, but will be augmented by geodetic astronomy and emerging accurate 'super-gyros'. The geometry of the Earth and its temporal variations would include models for the solid Earth, ice sheets, and the ocean surface and their change in time and space whether secular, periodical or instantaneous. All conventional and space point positioning techniques will contribute to this modelling process as well as surface measurement techniques, such as satellite altimetry, interferometric satellite techniques, and remote sensing. Finally, the gravity field of the Earth and its temporal variations will require models for mass balance, fluxes, and circulation patterns which put constraints on the geokinematic models. The required measurement systems have already been discussed in chapter 5. The largest future contribution to the global gravity field representation is expected from the proposed gravity satellite missions. For the numerous interactions between the components of GIGGOS indicated by arrows, the paper by Rummel (1998) should be consulted.

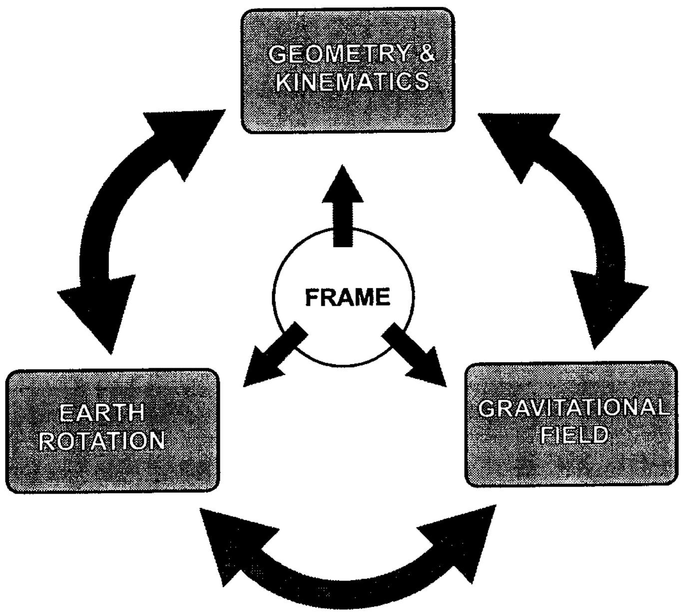


Figure 13: Towards a Global Integrated Geodetic and Geodynamic Observing System Adapted from Rummel (1998)

The idea of coordinating and focusing geodetic activities under such a concept have generated a lively discussion inside the International Association of Geodesy. Some of this discussion is captured in Beutler et al (1998) and in some other papers of a recent IAG/Section II symposium in Munich (see IGGOS, 1998). Such a system is attractive to many researchers because it

- could become the focal point for research activities within the IAG, including much of the current research, and would accelerate the integration of classical and space measurement techniques.
- would more clearly identify the IAG contribution to Earth system science and show that the interaction of IAG with other Earth sciences goes well beyond data delivery.
- would recognize that the contribution of geodesy goes beyond solid Earth research.
- would, on the one hand, use the metrology tradition and strengths of geodesy and, on the other hand, open new vistas and challenges for young geodesists.

Such a program would emphasize the science tradition of geodesy which has been a strong component of geodetic activities since the Internationale Erdmessung was founded about one hundred years ago. With time, it would considerably extend the impact of geodesy on other branches of the Earth sciences and accelerate the cooperation between national agencies contributing to such an enlarged concept of geodesy. The engineering tradition of geodesy which also has strong roots in IAG would not be enhanced in the same way. This does not mean that its influence would dwindle. As indicated in the previous section, there are many challenging tasks in accurately representing the Earth's surface and its temporal change for local applications. These applications will continue and will profit from a better understanding of the processes that are at the root of change. In the long term, they will be needed to describe the fine structure of the Earth's temporal variations.

Acknowledgements

The author would like to thank Jean-Marie Becker for the invitation to present this talk, proposing a topic that provided room to roam and ramble. Thanks are also due to A. Bruton, M. Mostafa, and J. Skaloud for actively participating in the design of the figures.

References

Beutler, G. , H. Drewes, C. Reigber, R. Rummel (1998): Space Techniques and Their Coordination within IAG at Present and in Future. Int. Symp. of IAG-Section II "Towards an Integrated Global Geodetic Observing System (IGGOS)", October 5-9, 1998, Munich, Germany. Proceedings to be published by Springer Verlag.

Blewitt, G., C. Boucher, P.B.H. Davis, M.B. Heflin, T.A. Herring, J. Kouba (1997): ITRF densification and Continuous Realization by the IGS. Proc. IAG Symposia Volume 118: Advances in Positioning and Reference Frames, F.K. Brunner (editor), September 3-9, 1997, Rio de Janeiro, Brazil; published by Springer in 1998.

IGGOS (1998): Int. Symp. of IAG-Section II "Towards an Integrated Global Geodetic Observing System (IGGOS)", October 5-9, 1998, Munich, Germany. Proceedings to be published by Springer Verlag.

Ilk, K.H. (1998): Envisaging a New Era of Gravity Field Research. Int. Symp. of IAG-Section II "Towards an Integrated Global Geodetic Observing System (IGGOS)", October 5-9, 1998, Munich, Germany. Proceedings to be published by Springer Verlag.

NAPA/NRC (1995): The Global Positioning System- Charting the Future. The National Academy of Sciences, Washington, DC.

Rummel, R. (1998): A Global Integrated Geodetic and Geodynamic Observing System.

CSTG Bulletin, Munich 1998.

Schwarz, K.P. (1998): Mobile Multi-Sensor Systems - Modelling and Estimation. Proc. of the Int. Symp. on "Geodesy for Geotechnical and Structural Engineering", April 20-22, 1998, Eisenstadt, Austria.

Slater, J.A. and S. Malys (1997): WGS 84 - Past, Present and Future. Proc. IAG Symposia Volume 118: Advances in Positioning and Reference Frames, F.K. Brunner (editor), September 3-9, 1997, Rio de Janeiro, Brazil; published by Springer in 1998.

The National Land Survey of Sweden today and tomorrow

Joakim Ollén, Director General

The National Land Survey of Sweden, originating from 1628, is a Government agency under the Ministry of the Interior. The mission is to give support for creating an efficient and sustainable use of Sweden's real property, land and water. The combination of geographic information, land information, property formation and geographic information technology gives us unique possibilities to meet the users' needs.

Change in society is rapid and affects us all. To continue to provide high levels of service, we need to anticipate and plan for change in the needs of those who depend upon our data, information and services. We do this in the context of a transformation in the very nature of the public service: over the last years, there has been enormously increased emphasis upon efficiency and measuring value for money in these services. The operation accounts for the last years show that we have had problems in the adoption of the organisation to new circumstances, but that is now history. In 1998 we started to earn money again, and we plan to do still better. Not primarily for our own sake, but for our customers. To meet the users' needs it is essential to have resources for development of new products and services as well as staff competence.

In the introduction of the seminar on Geodesy & Surveying in the future I will summarise a vision of where we intend be in five years time. I will also give examples of the strategies and plans which will translate the vision into reality.

History and evolution of height determination techniques especially in Sweden

Prof. Jean-Marie Becker

The surveying profession has been subject to many important changes during the last decades. We have seen a rapid technological evolution especially concerning the surveying techniques and instrumentation used for different applications.

The time of surveying with purely optical and mechanical instruments (steel meter, levels, etc) has rapidly been replaced with more sophisticated surveying techniques and equipment like motorised total stations, GPS, etc. However this was mainly for the purpose of positioning in 2D, plan co-ordinates.

Attempts to automate and make the levelling process more efficient have also been going on for a considerable time through both method and instrumentation developments. Today we use a new kind of survey systems based on "Black box", "Push Bottom" fully automate and producing digital height data in real time like the Swedish Motorised Levelling technique (ML). In the following report the author will present some of the most important development steps during his live time and illustrate this with examples from Sweden like the different motorised height determination techniques.

1-Introduction

The today Surveyors are familiar with a new generation of surveying equipment and techniques who can bee characterised as "Black box technologies" giving results in real time and in digital form. Many of the surveying activities including the field operation have been reduced to simple "Push of Button" operations with limited use of the knowledge and experiences of the professionals.

In our obstinate research after a "universal geodetic surveying system", it is easy to be blind. Often the reality does not correspond to our dreams and expectations. Sometimes it can be very useful to take a look backward and around the corner on other technical solutions to gain a better knowledge and understanding, as Plato told it will help:" in tenebris lumen rectis!". (And light came in the darkness), this can be done through a short historical summary.

This technical evolution in our profession has also influenced the instrumentation and techniques used for height determination especially for large projects as for the establishment of the national height network.

In the following report I want to present how the evolution has been during the last decades and how Sweden was involved in the development and use of modern techniques like Motorised Levelling (ML), Motorised Trigonometric Levelling (MTL) or Motorised 3-D technique M.XYZ.

I will also shortly describe some of these techniques and present their results seen from different point of view like the production capacity, the quality in term of accuracy, the working conditions and the economical aspects.

Finally to focus more on the theme of this Jubilee Seminar "25 years of Motorised Levelling" I will show some pictures from different countries around the world where the ML and MTL techniques has been adopted and implemented.

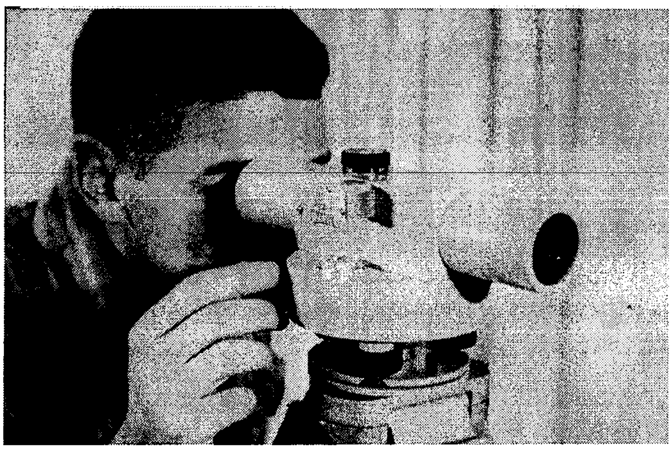
2- Milestones in the technical development

The technical development of the height determination techniques has been done in different ways, fields and times. We can see at least two major fields of development- firstly concerning the measuring instruments and secondly the measuring procedures/ techniques.

2.1- Instrumental development

The advancement concerning equipment does not realised everyday, especially with regard to the instruments themselves. However we can distinguished some important milestones marking the development of instruments.

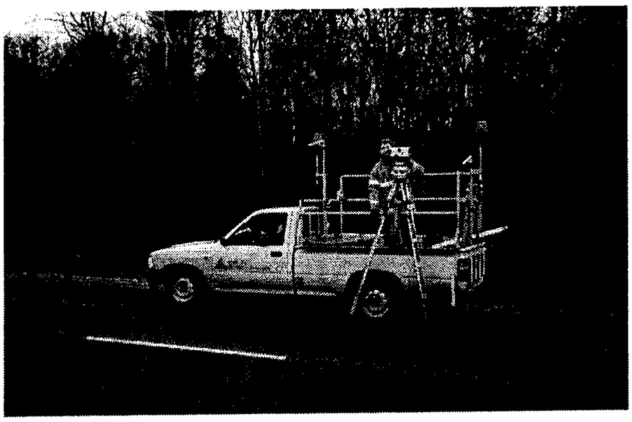
- 1951: Introduction on the market of the Ni2 by Carl Zeiss Oberkochen. It was the first so called "*self-horizonting*" instrument with an automatic pendulum instead for spirit level for the determination of the horizon. This simplified significantly the work and increased the production rates.



- 1972: Introduction of the Ni002 from Carl Zeiss Jena, the first self-levelling pendulum instrument giving a quasiabsolute horizon (through two symmetrical pendulum positions) as well as a rotating eyepiece allowing to shut 360 degree around and with the haircross placed in the objective. This instrument made possible the motorization of the levelling procedure thanks its technical features.

- 1985: The use of electronic total station I combination with 3-D measurements gives height results of very high accuracy in concordance with the specifications for precise levelling

- 1989: NA 2000 from LEICA/Wild was the first digital level on the market. This was an important step forward in the digital capture of field data. Several other digital levels produced by Leica (NA 3000), Zeiss (DiNi10-11), Topcon, Sokkia are now on the market.



Thereafter the increasing use of satellite techniques (like GPS) for large scale projects especially in rural areas where the researched accuracy is lower (some cm/dm) and for studies of long term deformations.

Before 1950 all levelling where practically produced with so called "*spirit levels*". After 1950 a new generation of instruments *("self-horizonting")* takes over first for low and medium precise levelling work and thereafter also for precise levelling. Some old fashion spirit levels like Wild N3 survived up to the nineties.

The introduction of the digital levels 1989 exclude definitively the old generation of "spirit levels" from all kind of levelling works.

Today digital levels and electronic total stations are predominant for all kind of height determination work.

The use of GPS for precise height determination is progressing in urban areas but steel not competitive for many purpose especially urban areas regarding the accuracy and the costs.

2.2- The development of the measuring techniques

The evolution of the measuring techniques is mostly the result of the efforts made by national agencies responsible for the establishment of large national projects of high precision. These organisations have to face two problems: firstly an increasing demand for densified networks with high accuracy and secondly the lack of financial support to fulfil this demand. To reach these goals it was necessary to increase the production, to modernise and optimise the production techniques making the best use of the new instrumentation on the market.

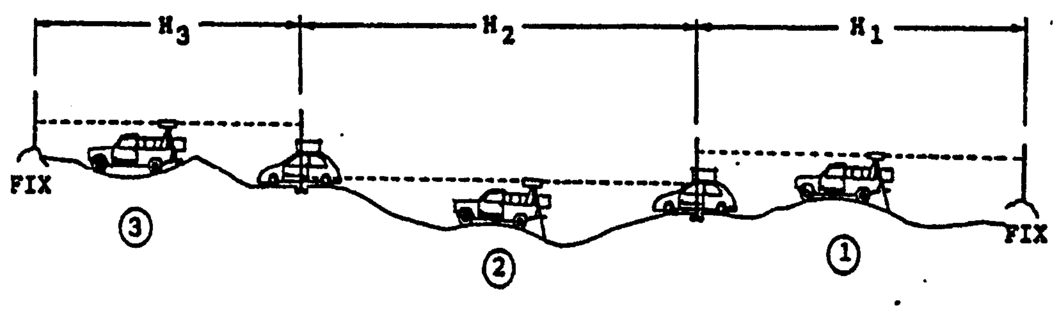
At least two different measuring technologies have been in use for the production/establishment of national height network namely Levelling by Foots (FL, TL) and Motorised Levelling (ML, MTL).

In many countries all levelling works are steel done as for 50 years ago using classical foot levelling (FL) procedures, the only changes are related to the use of new instruments as for example digital levels and electronic fieldbooks. To insure high quality (accuracy) the sighting lengths are limited to 30 max 35 meter, the measures are only done by favourable meteorological conditions (negative temperaturgradient), and many believes in the healthy effect of several corrections made in the office, etc.

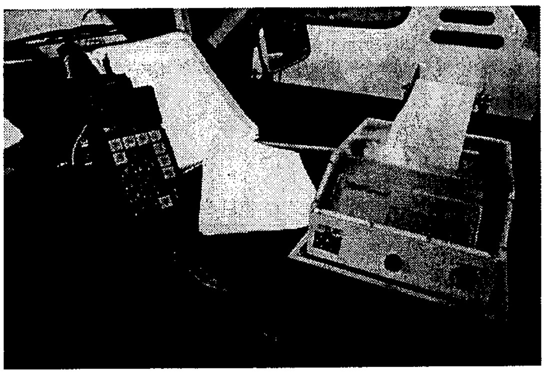
For the National Land Survey of Sweden as early as 1970 it was apparent that the national heights networks did not meet the user-needs and requirements, regarding both quality and density, especially from the first order height network. To remedy to this situation NLS started a lot of investigations, studies and tests to found or develop an appropriate levelling technique. Up to that datum NLS was operating like all other countries using the FL technique for precise levelling and it clearly appears that this technique was not the appropriated solution: to slow and to costly.

Therefore other possible solutions were tested to increase the levelling speed using bicycles or cars for the moves from one instrumental station to the next. Such as to allow good levelling results during the hall working day, to minimise the numbers of readings and to assure a permanent quality control of the field operations by as example the use of electronic datalog, etc. All these efforts result in the following development steps:

- 1973/74: Construction and field tests of the first Swedish Motorised Levelling (ML) equipment. (Se details chapter 4). ML use tree cars (one instrument car and two rod cars) for all levelling operations. This technique made it possible to work through all weather conditions and seasons increasing twice the production rates and also the quality of the results. (Se next chapter). A special tripod with footplates was developed and used with success.



- 1981: Construction and use of the first datalog (fieldcomputer) together with MICRONICS (Sweden) allowing the storage and automated field control of all observation data. This was the first step in the digital production line.



- 1981: Construction and use of an automated rodcomparator with laserinteferometer (designed in collaboration with H.Schlemmer). This comparator made it possible to calibrate and calculate corrections for each graduation from the invarrod, which increased the quality of the results. The whole calibration process takes only two hours for both scales again one week for the same operation by classical optical procedures. Furthermore all data are produced directly in digital form.
- 1982: Construction and use of the first 3,5 meter invar rods increasing the production rates with 15% because they allowed to increase the mean average observation sightlength from 33 to 37 meters.

- 1985: Development of two new motorised height determination techniques: MTL (Motorised Trigonometric Levelling only for heights) and M.XYZ (Motorised 3-D technique). Both are using modern electronic totalstations instead for the classical levels Ni002. Each car is a modified instrument car from ML. The achieved performances where astonishing both in quality and quantity. Precise levelling accuracy (< 1mm/ V km) was performed. The purpose was there use in mountainous areas where classical ML has limited sightlengths.

MTL: The observer in action on instrument car

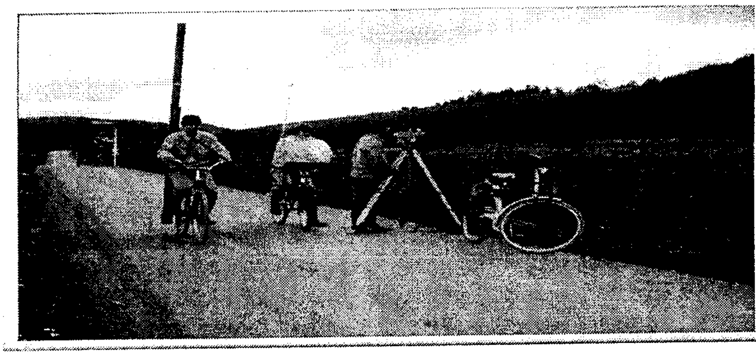
- 1988: TL or Trigonometric Levelling using motorcycle. Four wheel motorcycle (type Honda) were used for the transport of equipment and personal outside the roads where car cannot be used, mostly in swampy and hilly areas on the border to Norway (se picture). The measurements were made in a classical way with set-up outside the vehicles.
- 1994: Use of scanners for the digitising and storage of the BM (benchmark) descriptions (protocol) into a digital database.

Note: The GPS technique has not been used up to now in such kind of project because this technique is steel not competitive with ML concerning the high accuracy and the costs.

3- Description of some levelling techniques: CL, RL, ML

3.1- Levelling with the help of bicycles: CL

The apparition on the market of the self-horizonting levels (small, light, less sensitive and easy to transport) made it possible to use bicycle for their transport from one instrumental station to the next. All observations are made as by FL. The essential benefit was higher production rate. In Sweden CL was implemented in second and third order levelling from the sixties until 1980.



The use of CL for precise levelling was not enough efficient to compete with ML and therefore Sweden never adopts CL for precise levelling. However our Finnish colleagues made their own variant of CL and use it for their 3rd precise levelling (se pictures)

3.2- Levelling along railways using trolleys: RL

In many countries the levelling network established the last century follows the railway lines mostly because it was a easy way to perform good levelling with perfect concordance between forward and backward sightlengts during the hall project (often 30 meter) thanks a constant and low inclination of the trajectory. In Sweden and Finland the majority of the first order levelling network lines (from the first and second first order levelling networks) were along railway lines. The Finnish colleagues developed for their own purpose one sc. RL technique using handdriven inspection trolleys for the transfer of the equipment. Because it was impossible to pass each others from set-up to set-up (all on the same rail), they had to apply a kind of *"leapfrogging"* moves consisting in the simultaneous jumps forward for all operators of the team (rod Backward, instrument and rod Forward). Each levelling rod being all-time the Backward or Forward rod between two BM.

The increasing, dense and heavy railway traffic at least in Sweden made it difficult and dangerous to work in the railway area. The stability and access to the benchmarks along railway lines is not satisfying for the users needs. For these reasons among others the new (third) precise levelling network of Sweden was planned to be located along the roads.

3.3- Motorised Levelling: ML

Several attempts were made in different countries to use motorcycles, cars or other vehicles to speed up the production rates in order to reduce the expenses without reducing the accuracy of the results. The existing technical solutions fall into two distinct groups: *semimotorized* (1/2ML) and fully motorised (ML) techniques. In the first group the vehicles are only used to transport personal and equipment between set-ups; all observations are carried out in a classical way as with FL.

With the *fully motorised technique*, all work is performed directly from the vehicles. The operators do not leave their car, the only exception to this rule is when connecting to benchmarks.

The first successful motorised levelling unit could be developed 1972/73 in Germany (former DDR) by Prof. Peschel/Dresden through the special designed level Ni002 from Zeiss Jena. DDR use it then for the remeasurements of their network (around 5000 km).

Influenced by the promising results from DDR, the National Land Survey of Sweden decided to built a Swedish ML team, to acquire experience by tests under field production conditions, to verify and evaluate the results and if possible to improve this ML technique.

During 1973-1985 technical improvements were made step by step as result from the experience gained through many tests and measurements made with ML both in Sweden and in other countries:

- the construction and use of a special tripod with long adjustable legs and special foot plates to reduce settlement effects; vibrations, wind and temperature influence
- the construction and use of the MICRONICS data log as electronic field handbook/computer with printer
- The use of 3,5 meters Inver rods with tree bull's eye levels for permanent plumbing control and electronic temperature sensors at 0,5 and 3,0 m over the ground to apply correction for temperature influence on the rod scale.
- radio communication system
- electronic precision trip meter "Digitrip" for the determination of sightlengths and guidance of the car moves

The today ML equipment is illustrated in the following pictures. It is perhaps interesting to note that several different car types have been successfully used for ML purposes.

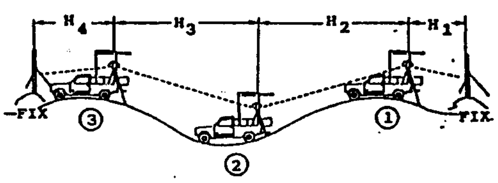
The field operations themselves where also subject to some improvements like:

- Use of "*non equal sightlengths*" for Back & Forward observation at each set-up. Differences up to <10% at each set up, where accepted
- only for readings (two for each pendulum position I & II) , one on each scale of the invarrod graduations

- automatic registration and control check in accordance with the accuracy/tolerance specifications at each set-up
- *change of observer* between the Forward and the Backward of each benchmark interval, section
- the separate Forward and Backwards measurements of a levelling line/section are never carried out immediately after each other
- field operations under *all weather conditions* preferably as different as possible between Forward and Backward levelling of the same sections
- once the week *instrument check* using two instruments together
- all invarrods are calibrated twice the year/field season at meteorological conditions (temperature- humidity, etc) corresponding to their *field use conditions*:- not unique and specific only for laboratory conditions

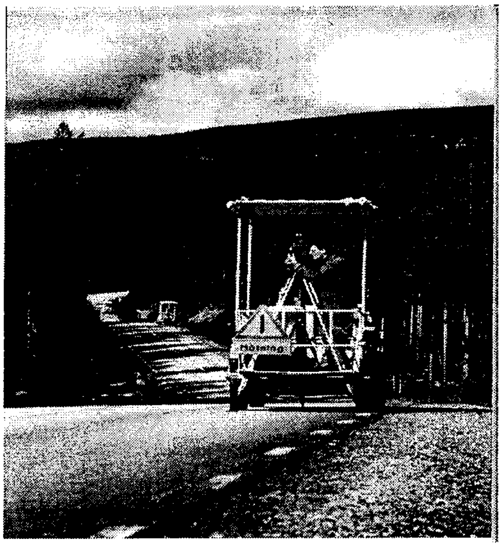
3.4- MTL or Motorised Trigonometric Levelling

The MTL developed in Sweden uses electronic total stations instead for levels. Each team consists in minimum two or preferably tree identical instrument-cars that are modified instrument ML-cars with a more central position for the tripod. The observer is steel forced to go around the instrument for the pointing Backwards and Forwards, which is not necessary by ML.



All observations are made simultaneously and reciprocally between two instruments set-ups and the calculated height differences continuously checked before any move. The procedure is fully automated and computerised and needs only perfect human co-ordination for the simultaneous reciprocal pointing.

All an MTL result shows high quality (accuracy) corresponding to first order precise levelling requirements if well defined operating procedures are strictly followed. However this technique was economically not competitive with ML in Sweden, as it was the case in Denmark. The reasons are to short average sightlengths because of obstacles in the pointing trajectories.



3.5- M.XYZ. Or Motorised 3-D positioning.

This technique is a sophisticated variant of MTL where all the data (horizontal, vertical angles and distances) from the electronic total stations are used and combined to perform long traverses measurements along the roads. To produce co-ordinates (2D + 1D) simultaneously require good connections to known reference points. This method achieved cm–level accuracy in x, y and 0,5 cm in altitude. M.XYZ was only used under very short time in Sweden for the positioning of opto-cables for Telecommunication Company needs and replaced by the INS (Inertial Survey) technique that was more powerful for the same needs.

4- Comparison of the results of different techniques: advantages – limitations

To better illustrate the results yielded by the motorised levelling technique ML, a comparison with the other techniques is made below concerning efficiency, quality, and working conditions.

4.1- Efficiency.

The efficiency of a height determination technique can be illustrated on the one hand by the average daily production (net new production) and on the other hand by the cost per produced kilometre.

The main average daily production for different techniques is given in the following table. These results comply with identical quality specifications and are obtained during a normal 8 hours working day. Statistics shows that the effective measuring time by ML is about 5,5 hours per day, the resttime is used for transfers, breaks, etc.

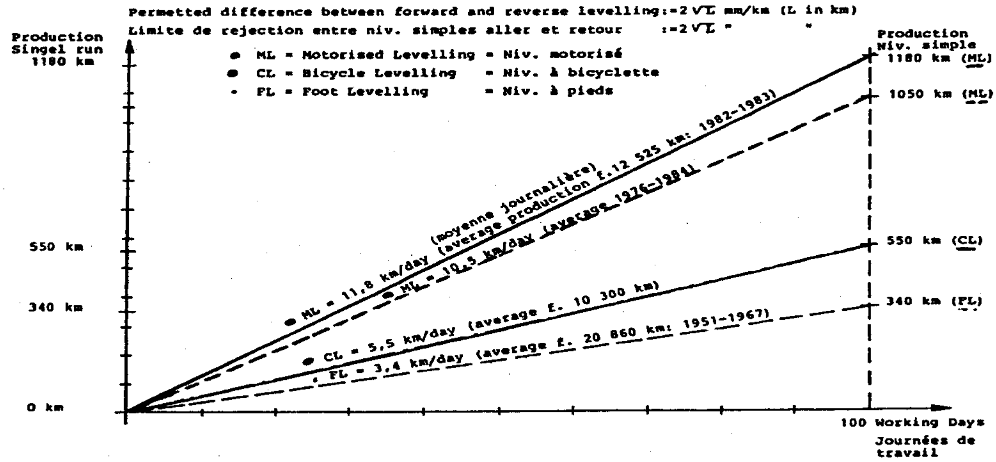
The average hourly progression for precise levelling by ML is around 2, 2 km with average sightlengths of 35 meters (maximum allowed 50 m). The total time used for each height difference at each set-up including the moving time, varied between 1,6 and 2,4 minutes depending on the sightlengths, etc.

The brut average daily production between 1978-1998 is 12,0 km including 7% relevelling that means about 11km daily net production.

These production rates are the result of an optimal work distribution between all actors in the ML team to catch all interesting data at each set-up.

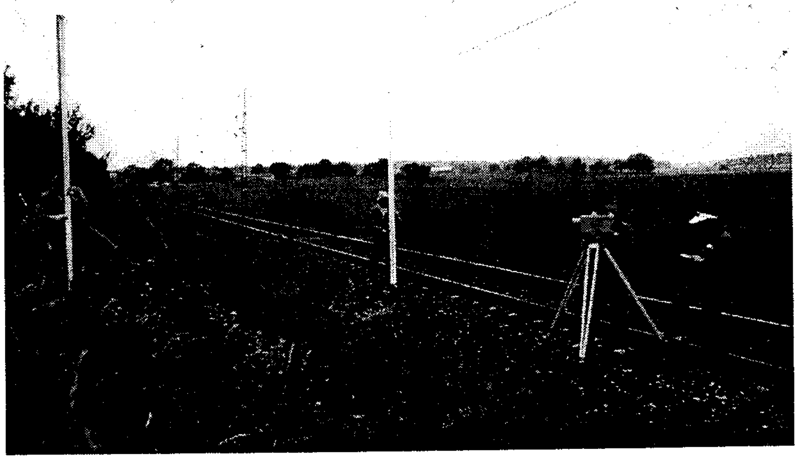
FIGUR 3 COMPARAISON DES RENDEMENTS OBTENUS EN SUEDE L'AIDE DE 3 TECHNIQUES DIFFERENTES DE NIVELLEMENT

COMPARISON OF PRODUCTION FIGURES FOR 3 TYPES OF LEVELLING TECHNIQUES IN SWEDEN



Note: Tests made with digital levels (Leica or Zeiss) where all information recording and storing has to be made through the instrument itself shows that more time is needed per set-up and the optimal worktime distribution model is disturbed. Our tests give 15% decreasing production rates, which is important for the production costs.

The same statistics concerning other techniques shows for FL= about 3,5 km single run (with classical levels like Wild N3) and 5 km using Digital Levels; for CL and RL about 6 km and for MTL around 10 km.



The most significant advantage compared to FL, CL, RL is that ML can perform good results during the *whole fieldseason*, the *whole working day* independently the weather and surrounding conditions (se figure above). These results have been confirmed in other countries with quiet different working conditions like Zambia, Malaysia, France, USA, etc.

From an economic point of view, this possibility to perform *non-stop measurements* under the normal working conditions existing in Sweden, has reduced the production price by more than 50% compared to FL or CL.

4.2- Quality of the results

The field specifications state that the rejection limite between separate For-and Backward measurements of a section is: < 2 x D.km1/2 expressed in mm. The rejection limite at each set-up between the two scale height differences is 0,4 mm. The statistics from more than 49 000-km double run shows an average percentage of relevelling of 7% independently the working conditions (meteorological, surroundings, operators, etc). This is about the same for FL or CL but limited to the reduced working time when FL and CL can perform acceptable results. (se figure above).

The mean standard error per km calculated from either the discordance's between separate For-and-Backward measurements of levelling lines/sections or from the closure errors of levelling loops through different computations/adjustments are better than < 1mm / D.km1/2 which correspond to the international norms for precise levelling.

Here it is perhaps worth to mention the factors which make it possible to carried out without interruption these good results with ML under the most varying working conditions compared to the difficulties with FL and CL techniques

- less refraction and flimmer effects because of the higher sighting line for the instrument above the ground = 2,1 m (se Kukkamäkki table nr.3). No pointing on the Invar staff below 0,6 m.
- perfect stability of the instrument and Invar staffs: no operator movements around them and permanent check of their verticality through the 3 bull's eye levels
- reduced sinking effects thanks the special tripod foot plates
- less errors do to fatigue because of the regular change of observers at each BM and transport by cars
- better homogeneity because of identical & symmetrical refraction effects over the same ground surfaces below the sightlines
- less settlement linked to the time factors because of higher operating speed.

The amount of data collected will also allow us to analyse the effects of different registrated factors and hopefully help to better understand the systematic error budget affecting all levelling operations. Corrections for minimising the bad influence of systematic error sources can thereafter be made by postprocessing.

4.3- Working conditions and field use

To perform levelling in a classical way like FL or also CL has all-time been considered as a hard-working monotonous task because of the weight of the equipment (invar staff, turtle, tripod, etc). Few surveyors were attracted or interested to work, day in - day out, a whole field season with levelling. The introduction of motorised techniques like ML or MTL has greatly improved the field working conditions since nearly all operations are performed from the vehicles. The physical part has been lightened:

- no more tiring walks with heavy and cumberlone equipment because transportation by vehicles
- no manually holding of the Invalids in a vertical position thanks to the special suspension device
- all work from a seating position for the staffmen and the booker
- protection against rain, wind, etc
- no limitation in weight, volume and quantity for equipment better suited for survey and data registration (meteorological, etc)
- greater flexibility in the execution of field activities

The introduction of motorised techniques made it possible for female to participate to the field activities. Nearly half of the personnel are today women that were never the case with FL, CL or TL, the explanation came from the fact mentioned above.

Another important aspect concerning the field activities is the safety during the work for the personnel. Our experience shows that this has been strongly improved, the operators have better protection through firstly their own position inside the cars and secondly because of the numerous warning signs on and around the vehicles.

The following table show the production rates during some fieldseasons

Year	Total Km	Relevelled Km	Relevelled %	Netto Km	Total km/ Day	Netto km/ Day	Eff. Hrs Total	Eff. Hrs Day
1987	5326	300	5,6	5026	11,3	10,6	2459	5,2
1988	6082	529	8,7	5553	12,6	11,5	2903	6,0
1989	5063	451	8,9	4612	12,1	11,0	2395	5,7
1990	5505	585	10,7	4919	11,9	10,6	2667	5,8
1991	4249	278	6,5	3971	12,4	11,5	1961	5,7
1992	3989	257	6,4	3732	12,9	12,1	1817	5,9
1993	4180	258	6,2	3938	12,6	11,9	1921	5,8
1994	4077	447	11,0	3630	13,4	11,9	1825	6,0
1995	3964	425	10,7	3539	13,8	12,3	1648	5,7
1996	3815	445	11,7	3369	12,3	11,0	1702	5,5
1997	3470	291	8,4	3178	13,5	12,4	1472	5,7
1998	3376	285	8,4	3091	13,4	12,2	1395	5,5

5- Conclusions

We are now on the end of our historical retrospect travel through 50 years of technical evolution of the height determination technologies. We have 25 years of experience with motorised levelling techniques from Sweden and other countries. Finally we have to summarise our impressions and to conclude.

The digital levels are the most preferment type of levelling instruments for use in foot (FL) or bicycle (CL) levelling. Their use for high precise levelling is subject to some limitations (sightlengths < 35 meter) and restrictions (influence of temperature variations and light) that reduce their performances (quality and production).

For motorised levelling they are not competitive with the Zeiss Jena Ni002 for several reasons as example:

- decreasing production rates of about 15% non-optimal worktime distribution inside the levelling team =. because the concentration of all work to the observer
- to sensitive to non equal sightlenghts and temperature variations.(no quasihorizon by double pendulum positions)
- Their ocular is not rotating around the horizon which complicate the observations
- Some of them needs to see a to big part of the invar staff which is limiting the sightlengts

More than 100 000 km motorised levelling shows that the ML technique steel after 25 years is outstanding and the most efficient technique for high precise large scale levelling work compared to FL, CL or TL.

The results can be summarised as follow:

- accuracy in terms of mean standard error < 1,0 x D.km1/2 mm even under unfavourable conditions
- production under all weather and surrounding conditions during normal daily working hours: 08.00-17.00
- uninterrupted working throughout the normal field season also in countries like Africa, Asia
- improved working environment make it easy also for female operators
- increased production rates with > 50% compared to FL and # 15% to MTL
- production cost per levelled km reduced by about 30%
- increased measuring capacity by 40% per year in Sweden
- increased security for the operating crow

The MTL technique is more appropriated and competitive for second order levelling projects (mean standard error between 3 and 6 mm) in countries/areas where long range sightings (> 300m) are possible as example Denmark. MTL is also successful outside the road network especially in hilly terrain.

The new technologies like GPS have not yet shown that they are fully able to take over the role of the more "classical" height determination techniques. Also in the future FL, CL, TL, ML and MTL will be used and compete on the levelling market. We believe that all these different techniques steel can be improved and have a place for specific applications.

REFERENCES

Becker J-M: Den Motoriserade Avvägningen. RAK- D20, Stockholm 1973.

Becker J-M: Uppbyggandet av Sveriges Nya Riksnät i Höjd, LMV-Rapport 1984:01, ISSN 0280-5731

Becker J-M: Comparisons between different height determination techniques: Workshop on Heights, Hannover October 1990

Becker J-M et al: Experience with Motorised Trigonometric Levelling, LMV-Rapport 1988:23

Becker J-M: Riksavvägningsarbeten i Sverige under perioden 1974-1995, LMV-Rapport 1997:2, ISSN 0280-5731

Kasser, M and Becker J-M: Error sources in High Precise Levelling- How to minimise their effects on the heights, Presented at the seminar "Geodesi and Surveying in the future, The Importance of Heigths", Gävle, March 15-17th 1999.

The Evolution of digital levellling techniques - limitations and new solutions

Hilmar Ingensand, ETH Zürich

Forerunners

The development of Prof. Zetsche at Bonn [1966] can be considered as a forerunner of all digital levels. The image of the rod pattern was compared with a scaled-down rod pattern on a ruler at the focal plane. Due to the lack of adequate electronics there was no further progress in this direction until the appearance of CCD-sensors in the early eighties, and an improvement of the microprocessor performance opened the way to powerful image processing.

Besides the development made by Leica, it is also necessary to refer to the research work of the Technical University Dresden in collaboration with Carl Zeiss, Jena (Germany). A development instrumentally based on the Zeiss Ni002 and using a linear CCD-array with 1024 elements (pixels), had been started in 1982 and stopped in 1988 in benefit of other projects. These experiences are certainly the base for the actual development of Zeiss digital levels. In 1987 the development of a digital level at the Neues Technikum Buchs (NTB) (Switzerland) was published. Despite of a sophisticated zoom optic its measuring range was restricted to distances between 20 and 30 meters.

The Fundamental Information and Image Technologies in Digital Levelling

Most of the geodetic measurement methods as electronic terrestrial or spatial distance determination (EDM, GPS, GLONASS) can be characterised as an information transfer between two positions. Transferred to the levelling process it is the determination of a position at a vertical scale represented by a coded staff.

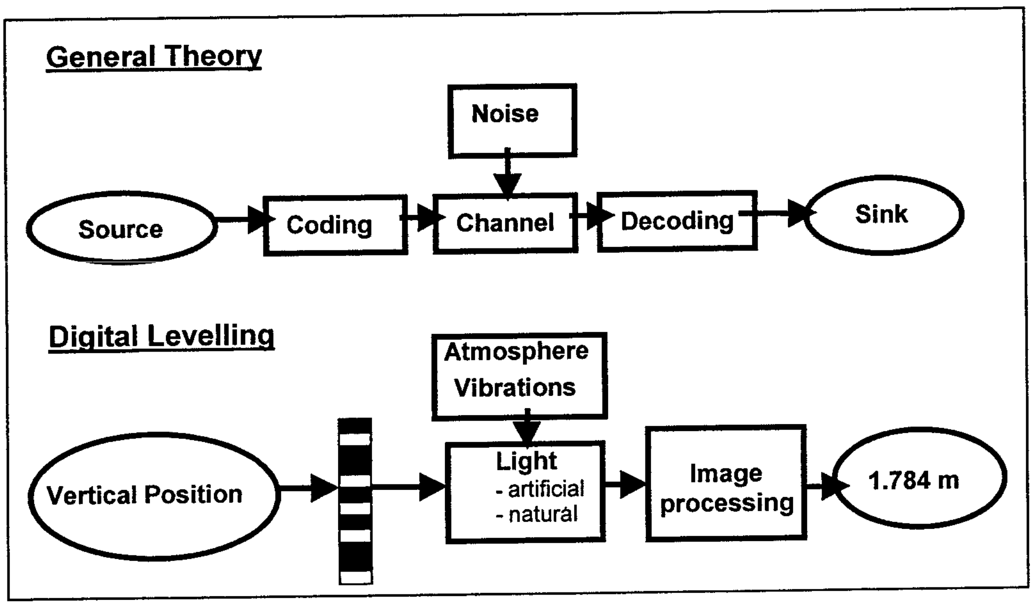


Fig. 1: The information transfer of digital levelling

According to other geodetic measurement techniques the same principles in coding and demodulation can be found in EDM- and GPS-technologies. As all digital levels normally operate with natural light, the sun light, various influences – in the language of information-transfer it is called NOISE- have to be covered and suppressed by the digital levelling process. These influences are listed in the following table 1.

Illumination	**Atmospheric influences**	**Mechanical influences**	**Instrumental behaviour**
Various light intensity of natural light (SNR)	Turbulences (blurred image, higher SNR)	Vibrations (deviation of the line of sight)	Thermal effects (deviation of the line of sight)
Inhomogeneous light intensity by shadows at the staff	Refraction (deviation of the line of sight)	Settlement of the instrument and staff	Interference of code-element size and pixels (wrong results at certain distances)
Spectrum of the light source		Staff centring and inclination of the staff	Compensator function (eigenfrequency)

Table 1: The different noise effects in the digital-levelling process

The Various Codes in Digital Levelling

The efficiency and the reliability of the information transfer is mainly a question of the appropriate code (modulation) of the information. In addition to the basic requirements of the information transfer, several geometric requirements of a code as pseudo-random characteristic, good contrast at the edges and unambiguity within a staff length of four meters and the distance range up to 100 m have to be covered. For patent reasons every manufacturer developed his own code and processing method. The Bonner and the NTB developments have shown that normally tuning of the scale by a zoom optic is required. The manufacturers were forced to develop special codes for the digital-levelling system, giving a clear projection without extensive optics. The code of all the manufacturers is set up in a way to convert the image via a linear CCD-array into a digital intensity- and position-information. Therefore, all codes use the black-white transition at the edge of the code-bars.

The **Leica** code represents an aperiodic pseudostochastic binary code. The complete bar code over the full 4050 mm length of the staff has 2000 elements, i.e., each code element has a 2.025 mm dimension. The so-called **Zeiss** bi-phase-code or more exactly the modulation of the code is based on an alternation of brightness within each bit of 2 cm width, that means whether a certain code element is either black or white, or consists of a black and a white bar. This pattern has an optimum distribution over the whole visual field so that at least 15 black-white-transitions can be detected within a field of view (FOV) of 30 cm. This grants sufficient oversampling and hence a high accuracy for the fine measurement. Only at very close range, below 6 m , additional 2 mm wide black or white lines are required.

The **Topcon** staff carries a code with three overlapping single patterns. A constant bar-triplet R as reference pattern and two further bars A and B coded in the adjoining bars. The bar width of the A and B pattern changes according to a sinus function from 2 to 10 mm and a wave length of 600 mm and 570 mm, respectively. The distance p between the bar-centres is 10 mm and constant. The two sinus signals have a phase shift of $\pm \pi/2$ at the beginning of the rod, so that there is always an unambiguous phase difference of the two signals A and B within the height range of 4 m. Distance and height are derived from the frequency and the phase position of the image of the three related codes using FFT.

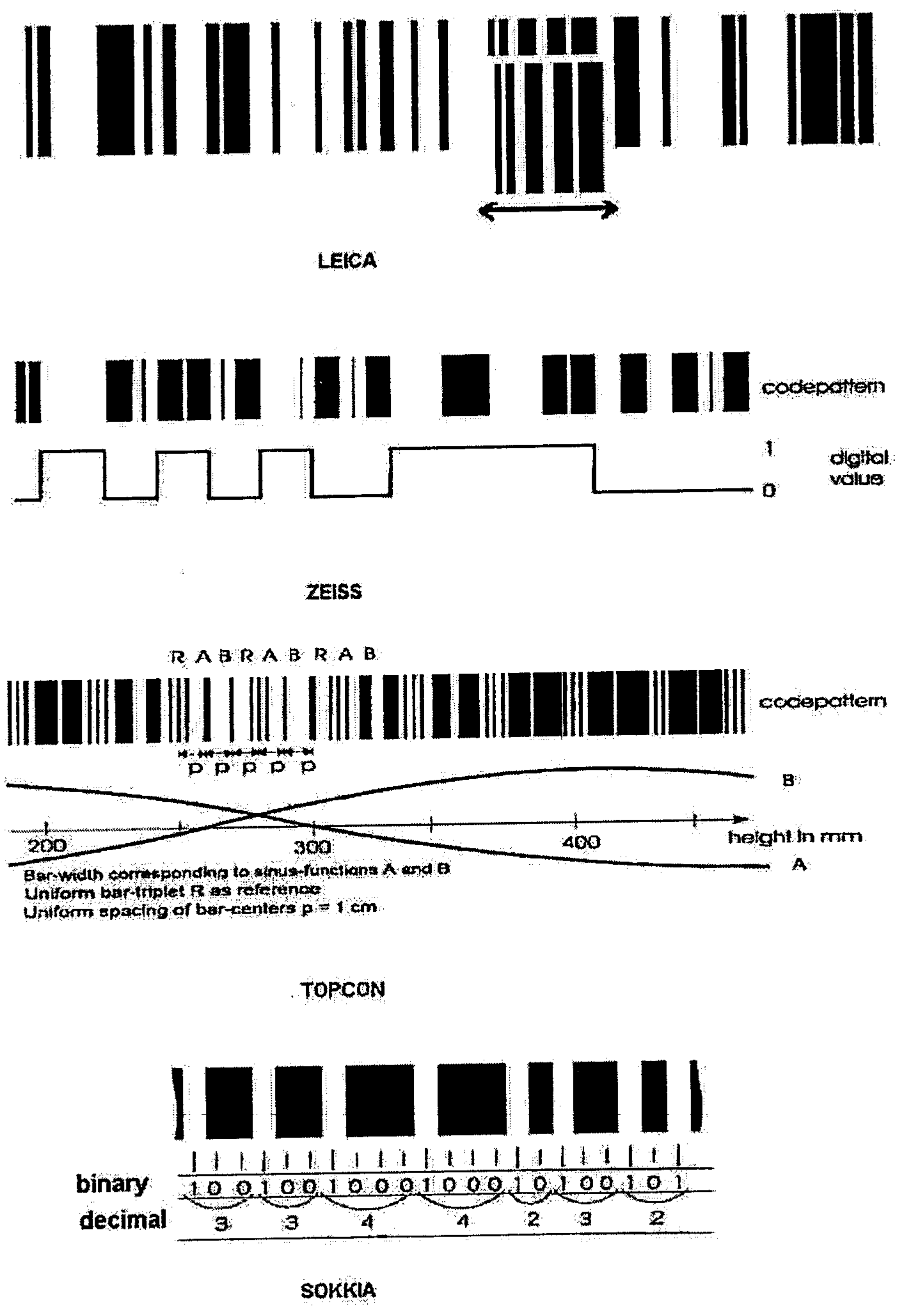


Fig. 2: The various codes of the actual digital levels

The **Sokkia** Random Bidirectional Code (RAB) covers a width variation of 6 codes, where each code is defined in relation to the basic code-element dimension of 16 mm. Each code can be recognised by the following relations: 1 = 4:12, 2 = 6:10, 3 = 8:8, 4 = 10:6 and 5 =12:4. The 0 code is required for short range measurements and is integrated in the form of white lines in the black bars [Nagao, Kanagawa].

Manufacturer	**Properties**	**Near-farfield code**	**Distance/ Scale required**	**Dimension of one code element**
Leica	Pseudostochastic	Yes	Yes	2.025 mm
Sokkia	Random bidirectional Digital width relation	Yes	No	16 mm
Topcon	Analog width variation	No	No	10 mm
Zeiss	Biphase	Yes	No	20 mm

Table 2: Code properties

General Features of the Receiving Units

Optical layout

Digital levels can be regarded as a fusion of a digital camera and automatic level. It has a telescope with upright image and a compensator to stabilise the line-of-sight. Additionally a position sensor coupled with the focus lens supplies a rough distance information. This refers to the Leica instruments only, the others operate without information of the focus-position. A tilt-sensor observes the compensator position and a beam-splitter guides part of the light to the CCD-sensor.

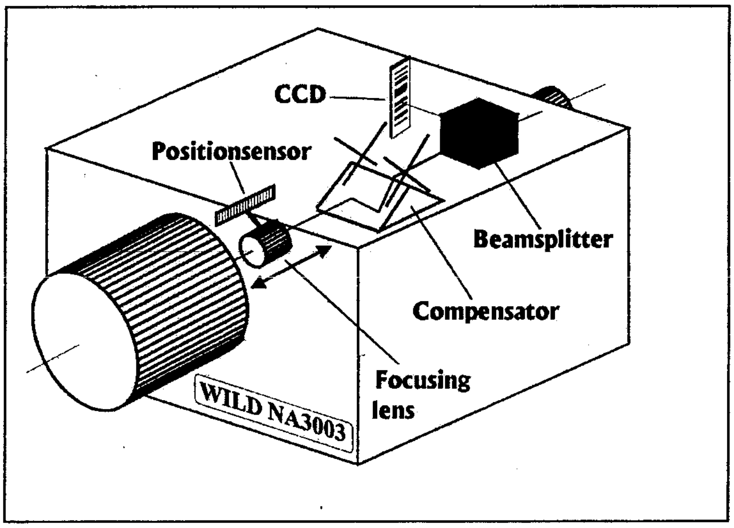


Fig. 3: Basic optical design of today's digital levels Leica NA3003

Electronics

The processor system is based on a microprocessor. For the complex computations needed for the correlation and reference functions, it is supported by a gate array (LEICA). The detector-diode array converts the bar-code image into an analog video signal of 256 intensity values.

Signal Analysis and Image Processing Methods of Digital Levels

The determination of the position by image processing is a combination of a radiometric processing and the detection of the edges, i.e., the black-white transition of the code elements.

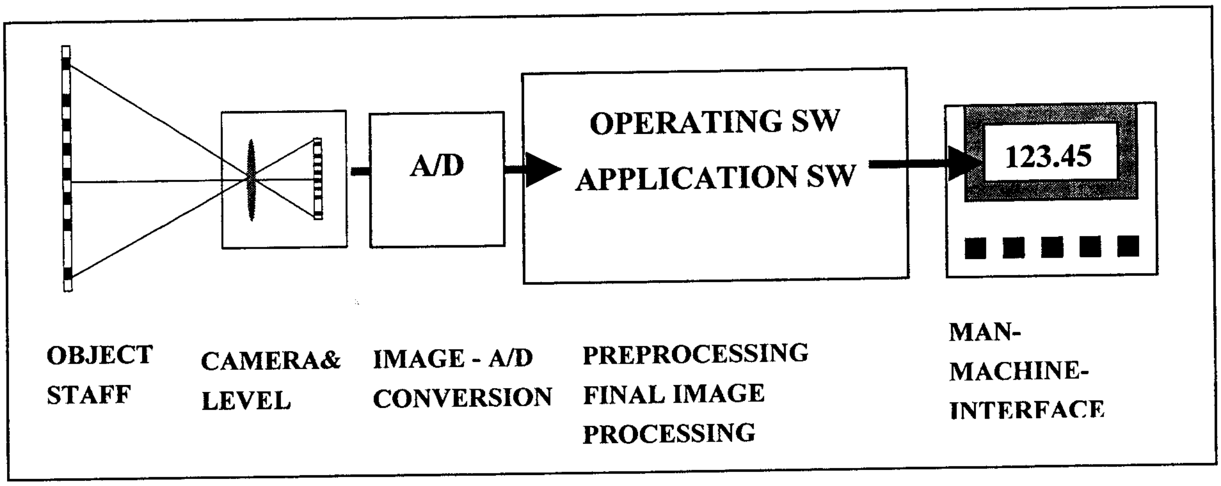


Fig. 4: Data capture and processing

Radiometric aspects of the image capture

The radiometric process must also take into account that each CCD-pixel exhibits a Gaussian sensitivity. This can be compensated by the convolution of the image with a trapezoidal detector-sensitivity function. In addition to the aforementioned detector sensitivity an inhomogeneous intensity variation (see Fig.5) of a partly shaded staff has to be covered by the image analysing and compensating process.

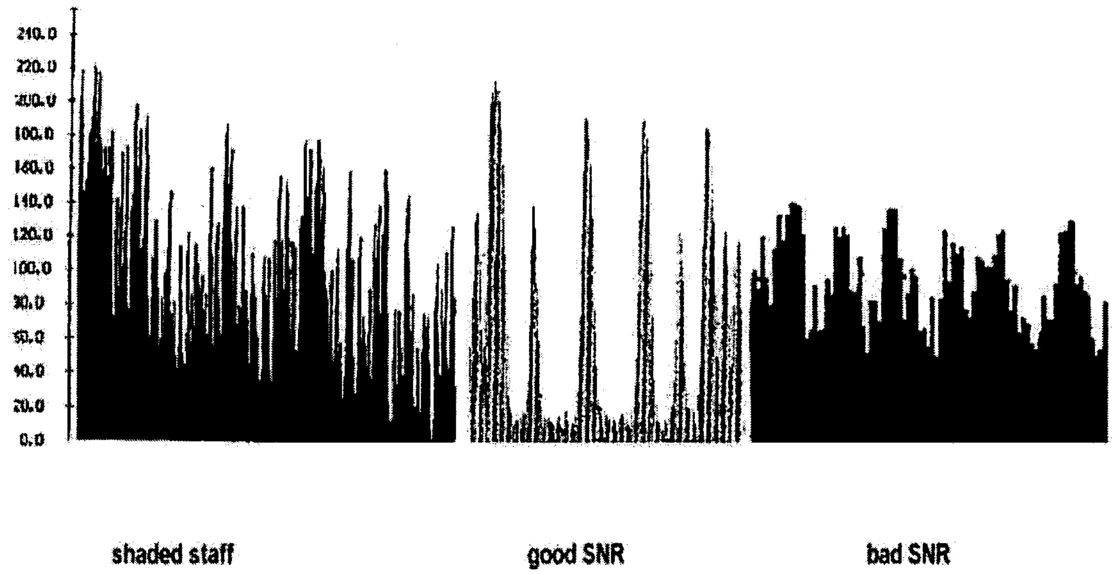


Fig. 5: Radiometry of rod images

Except the Leica system all other digital levels have at 100 m distance an oversampling of a minimum of 4 pixles with an average focal length of 250 mm and a pixel size of 10 μ and a code element of 1 - 2 cm. With Leica instruments having the aforementioned focal length at about 25 m staff distance there may occur interferences between the code element size (2.025 mm) and the pixel size [Schauerte]. In the mean time this effect is reduced by new image processing software.

Position Determination by Image Processing

The **Leica** digital level applies a two step and two-dimensional correlation method similar to GPS pseudo-range determination. The code is correlated with the image of the rod projected to the CCD-array. As the image varies not only in height but also in scale, according to the distance of the rod, it is necessary to optimise both parameters. With a range of 0 to 4.05 m in height and 1.8 to 100 m in distance and the respective increments of some millimetres in height and some meters in distance this would lead to 50 000 correlation coefficients to be computed with 8-bit accuracy. To speed up the evaluation process it is divided into three steps.

In a first step a rough distance information is derived from the position of the focus lens beforehand. The second step is coarse correlation. Using a threshold value according to the average intensity of the signal, its 8-bit dynamic is reduced to 1-bit. This permits the use of an EXNOR logic function instead of multiplication. The last step is a fine correlation with full 8-bit intensity-signal-accuracy in a very restricted area around the solution of the coarse correlation. Typical computation time is about 2 seconds.

The procedure of **Zeiss** can be described as geometric positioning method. A minimum FOV only 30 cm is sufficient to derive height and distance in the whole range up to 100 m. It uses the code for coarse positioning only and performs fine positioning by detecting and averaging several dark-light-transitions of the code-elements.

Topcon uses a phase-measuring method quite similar to the method known from electronic range-finders. The frequency and the phase position of the three signals can be gained by fast Fourier transformation (FFT). Besides linear combinations of the 3 codes A, B and R for accuracy augmentation are imaginable.

The **Sokkia** image processing method is quite similar to the Zeiss method. Intensive investigations at the Institute for Geodesy and Photogrammetry of the ETHZ have shown that this method is able to work with a minimum of 8 cm code.

Instrument	**TOPCON**	**WILD**	**SOKKIA**	**ZEISS**
Feature	**DL102**	**NA3003**	**SDL30**	**DiNi10**
Accuracy mm/km Double levelling	0.4 mm Invarstaff	0.4 mm Invarstaff	1.0 mm (0.7 mm ETHZ) Fibreglass Staff	0.3 mm Invarstaff
Distance (Resolution)	1 cm	1 cm	0.1% x D	1 cm
Compensator - Type - Accuracy - working range	 Pendulum 0.3" ± 15'	 Pendulum 0.3" ± 15'	 Pendulum - > ± 15'	 Pendulum 0.2" ± 15'
Measurement time	4 s	4 s	> 3 s	4 s
Range	2-60 m Invarstaff	1.5 - 60 m Invarstaff	1.6 – 100 m Standard-staff	1.5- 100 m Invarstaff
Man-Machine-Interface (MMI)	Menu	Menu with function keys	Menu	Menu with function keys
Display	2 lines	2 lines	2 lines	4 lines
Operation time/ battery	10 hours	8 hours	> 7 hours	1 day
Weight including battery	2.8 kg	2.5 kg	2.4 kg	3.0 kg
Field of view (FOV)	No Information	2°	1°20'	Minimum 30 cm
Data storage capacity	2400	500	-	2000

Table 3: Performances of the actual digital levels

Technical Improvements of Digital Levels

Although it is one of the most frequently asked questions in connection with digital levels, implementation of an autofocus has not been realised up to now in order to keep cost low.

Modern digital levels realise a height precision of some 1/100 mm. For typical levelling distances of about 30 m this is well below the influence of the atmospheric refraction. If we want to improve

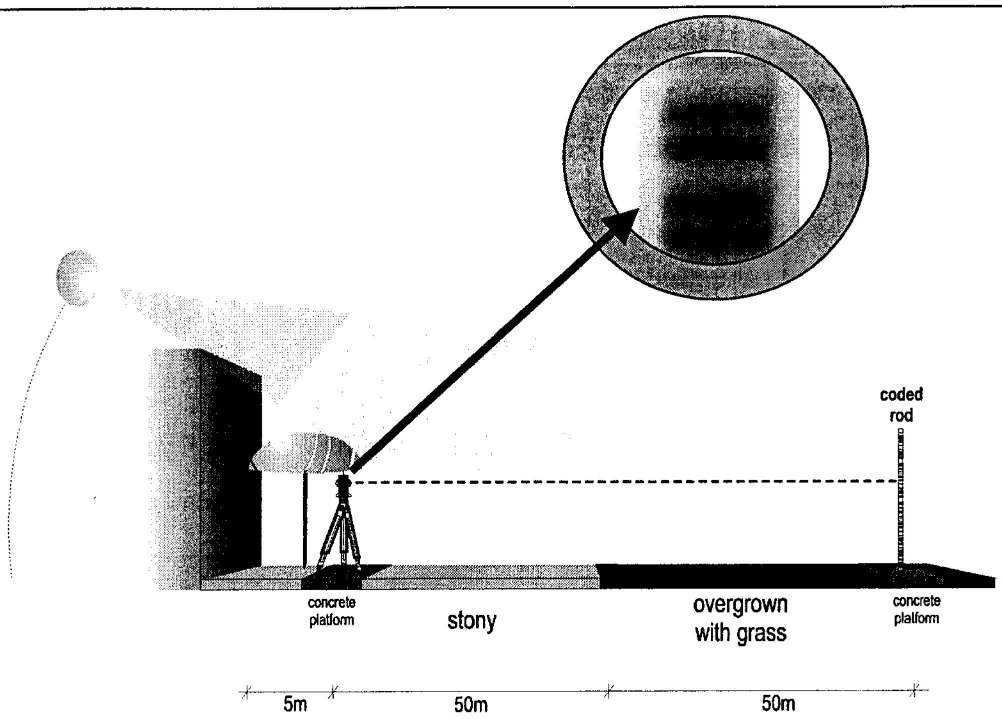


Fig. 6: Evaluation of refraction by image distortion [Flach, Hennes]

the reliability of height measurements, we must manage to determine the systematic influence of the atmosphere.

At the ETH Zurich there are promising results to evaluate refraction from image distortion [Flach, Hennes]. The realisation is asking for higher resolution, a rather large FOV, and a higher computational power. Another step in the development of digital levels can be the implementation of inclination sensors to enable inclined trigonometric observations with digital levels.

New Applications with Digital Levels

Besides the standard levelling application, digital levels have opened a new dimension in geometric levelling. But they have a potential to solve further problems that is just starting to being used:

- permanent monitoring with motorization
- improved motorised levelling

- plumbing
- tracking of construction machines

The frequent need for observation of buildings under construction has been leading to the idea of a digital level with motorised focus- and azimuth-drive to observe several targets permanently and automatically. Including reference-targets into the measuring cycle confirms the stability of the station. This system has been developed by Solexperts AG, Schwerzenbach (Switzerland), together with the Institute of Geodesy and Photogrammetry of the ETH Zurich. The first system, based on the NA3003, operated 1995/1996 at a building sites at Zurich. Meanwhile other monitoring systems, based on the Zeiss instrument, had been installed successfully [Keppler et al]. Beside the levelling task zenith and nadir plumbing can be realised with a pentaprism mounted to the front lens tubus and allow a monitoring of horizontal movements.

Outlook

Nowadays digital levels are used in all levelling procedures including motorised levelling. With the integrated horizontal circle the Zeiss DiNi has the capability of a levelling tacheometer. An implementation of area CCD's could give the performance of x-y-positioning and height tracking for construction machines and will improve the efficiency of motorised levelling in the future. It can be foreseen that high resolution CCD's and the adequate processing power will enable a correction of influences of refraction and turbulences. Basing on this the distance range of precise levelling and machine navigation with digital levels can be extended.

References

Feist, W., Gürtler, K., Marold, T., Rosenkranz, H. — Die neuen Digitalnivelliere DiNi10 und DiNi20. Vermessungswesen und Raumordnung, Heft 2, 1995, pp.65-78.

Becker, J.-M., Andersson, B. — Utvärdering AV NA 2000. Lantmäteriet-Rapport 7, Gävle 1991.

Becker, J.-M., Lithen, T., Nordqvist, A. — Experience of Motorized Trigonometric Levelling (MTL) – a comparison with other techniques. Lantmäteriet-Rapport 23, Gävle 1988.

Flach, P., Hennes, M. — Die optische Turbulenz – Wirklich nur ein limitierender Faktor für geodätische Messungen? Progress in Geodetic Sciences at Geodätische Woche '98, Kaiserslautern (1998), S.65-72.

Ingensand, H. — Neue digitale Nivelliertechniken und ihre Anwendungen. Vermessung Photogrammetrie Kulturtechnik, Nr.4, 1995, pp.221-226.

Ingensand, H., Meissl, A. — Digital Levels on the Way to 3-D Measurements. Conference on Optical 3-D Measurement Techniques, Technische Universität Wien, October 1995.

Ingensand. H. — Das erste digitale Nivellier der Welt. AVN, Heft 9, 1990, S. 201 ff.

Keppler, A., Meissl, A., Naterop, D. — Automatisierte Bauwerksüberwachung mit motorisierten Digitalnivellieren. XII Kurs für Ingenieurvermessung, A9, Graz 1996.

Nagao, T., Kanagawa, J.P.	Elektronisches Nivelliergerät und Nivellierlatte zur Verwendung bei dem Nivelliergerät. Offenlegungsschrift DE 19723654 A1 29.1.1998.
Schauerte, W.	Erste Untersuchungsergebnisse zum neuen DiNi 10. Vermessungswesen und Raumordnung, Heft 2, 1995, pp.78-88.
Schmid, R., Fürer, T.	Automatisierung bei Nivellierinstrumenten. Diplomarbeit am Neuen Technikum Buchs, Oktober 1987.
Zetsche, H.	Beiträge zur Konstruktion von Geodätischen Feldinstrumenten mit digitaler Datenausgabe. DGK Reihe C. Nr.88, München 1966.

Carl Zeiss AG	Prospekt Zeiss DiNi 10/20.
Topcon	Prospekt DL 101/102.
Solexperts AG	Brochure Motorisiertes Digitalnivelliergerät 1995.
Sokkia	Brochure SDL 30.

RESULTS OF TEST AND EXPERIMENTS WITH SDL30 DIGITAL LEVEL

Takao Seto, Minoru Chiba, Takashi Nagao, Masaru Muraki

Abstract

Electrification of surveying systems rapidly changed the situation of the level system. Several advanced manufacturers have already introduced new models of digital levels to the surveying market this decade. Sokkia has recently introduced a new model digital level, the SDL30, that combines accuracy and ease of operation at a very reasonable price.

The system employs a finer pitch (8 micron/cell) CCD sensor, high performance microprocessor (32bit RISC), and new bar code system called RAB-Code that utilizes a digital sensing method for highly reliable and wide ranging height measurements. The graduation patterns on the staff are read by the CCD sensor and classified into 3 kinds of character for long distance measurement to cover the lack of optical resolution, and 6 kinds of character for short distance measurements to make up for the insufficiency of position information limited by the narrow field of view of the telescope.

The new system was tested in severe field conditions for brightness, partial obstruction, ground vibration, and air scintillation. Also, a general performance accuracy test was carried out using ISO DIS 12857-1. Sokkia has created the unique and versatile digital level SDL30 that can be used not only for leveling, but also for measuring and monitoring applications.

1. PREFACE

The introduction of the GPS system into the surveying market has changed survey procedures fundamentally. However, leveling surveys carried out using levels and staffs remains an important surveying method because it enables accurate and simple surveying at a low cost, and produces necessary data for solving geoid undulation problems. For this reason, the level continues to maintain its status as a fundamental tool among surveying instruments. Recent progress in electronic components has led to rapid advancement in the performance and accuracy of surveying instruments and has given the level a chance for metamorphosis.

Sokkia has combined those new components with new ideas, and has succeeded in developing a new digital level system, the POWERLEVEL SDL30.

2 DEVELOPING CONCEPT

Our main objective was to produce a level that would provide high-accuracy measurement even under severe environmental conditions. To obtain our objective, we strove to achieve the following items.

(1) To obtain better resolution than that of the human eye.

(2) To be able to carry out measurement in a wide range of lighting conditions.

(3) To eliminate the effects of irregular brightness caused by shading of trees or buildings.

(4) To eliminate the fluctuating effect caused by ground vibration or air scintillation.

(5) To shorten the observing time.

(6) To avoid added costs even when countermeasures are applied.

We proceeded to employ the following features.

(1) Adoption of the finest pitch CCD sensor (8 micron pitch).

(2) Automatic control of charging time of CCD sensor and amplifier gain to match the input brightness.

(3) Digital readout of analog images.

(4) Increase the sampling rates, and adopt mean values.

(5) Adopt a high speed RISC microprocessor.

(6) Use the same optical and mechanical components of the current model.

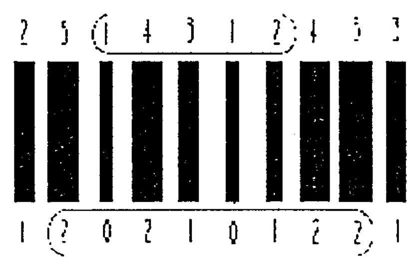


Fig. 1 A part of bar code.

3. PRINCIPLE

The SDL30 works in much the same way as other companies systems, by electro-optically reading the code pattern on the face of staffs to measure height and distance . However, Sokkia employs a unique code pattern on its staffs to improve measurement capabilities.

The following explanation outlines the characteristics and features of the bar code and includes some of the original ideas that went into making the digital level SDL30.

3.1 Bar code of SDL30

Pattern width (mm)	0-5 code Short dist.)	0-2 code (Long dist.)	Image size at 100m	Image size at 10m
3	0	0	1.05 μm/ 1 cell	84 μm/ 10.5 cell
4	1		1.40 μm/ 1 cell	112 μm/ 14.0 cell
7	2	1	2.45 μm/ 3 cell	196 μm/ 24.5 cell
8	3		2.80 μm/ 3 cell	224 μm/ 28.0 cell
11	4	2	3.85 μm/ 4 cell	308 μm/ 38.5 cell
12	5		4.20 μm/ 4 cell	336 μm/ 42.0 cell

Table 1. Relations between pattern width, code and distance

When designing the SDL30 we intend to be able to create a bar code to eliminate the effects of shading and automatically to identify and to recover partly missing code patterns. To achieve this goal we decided to adopt a system whereby analogue data output by the CCD line sensor is converted into digital data.

We required that the bar code be able to secure the solution necessary for measurement in a wide range of operating environments and that position information be acquired by reading a minimum amount of code with an error free algorithm.
Part of the bar code is illustrated in Fig.1. The composition of the code and image size on the CCD sensor is shown in Table 1.
We employed code patterns consisting of six different-width bars representing six levels of digital code incorporating 5-bit or 8-bit pseudo random noise code. For short distances, the image of the patterns are digitized with enough resolution to distinguish six levels clearly, but for long distances, it becomes difficult to recognize the patterns because the size of the image width and CCD unit cell size are closer, so 6 digit code is reclassified into 3 groups. In other words, the same pattern has two different meanings according to the distance. The newly obtained code series also becomes the unique address code. A minimum of 5-bits are required for 6-digit code and 8 bits for 3 digit code to cover the 10m length including direction discrimination. Using this method, the staff can be held upside-down to measure the height from ceilings. This method also enables the instrument to detect and correct errors automatically if redundant information is obtained. This feature is useful for compensating the effects of irradiation. We named this RAB-Code (RAndom Bi-directional Code).
The intervals between neibouring patterns have a constant 16mm pitch. This uniformity means that distance observation can be carried out simply and efficiently.

3.2 Features of RAB-code

(1) Digital image conversion

There are two procedures for reading bar code image signals: analogue and digital. Both procedures have their merits and demerits. Leica and Topcon use analog methods and Zeiss uses digital methods. Using the analog or direct comparison method between the image density pattern output by the CCD sensor and reference pattern in the instrument makes it difficult to separate level changes in the received image and irregularities of illumination that cause reading errors or malfunction. The digital method converts the pattern image directly to code, allowing the received code to be composed with the reference code. Even though bit errors may occur, if a large enough number of redundant marks are obtained, they can be checked and corrected. The digital method is more effective in compensating for irregularities in illumination and more reliably obtains accurate measurement results.

(2) Information density and recognition rate

Generally speaking, the total information is estimated by power-multiplying the visible number of bits by the number of coded digits consisting of bit data. However, by reducing the pattern size or increasing the types of coded digit causes a decrease in pattern recognition rate for longer distances, and limits the applicable range of the instrument. After many experiments, we decided to use a high density code that would maintain recognition rate at any working distance: 1.6cm constant pitch, 6 different width patterns 6 digit 5-bit code to 3 digit 8-bit convertible codes.

3.3 Observing sequences

Fig. 2 is a flow chart showing the processing procedures of the digital data.

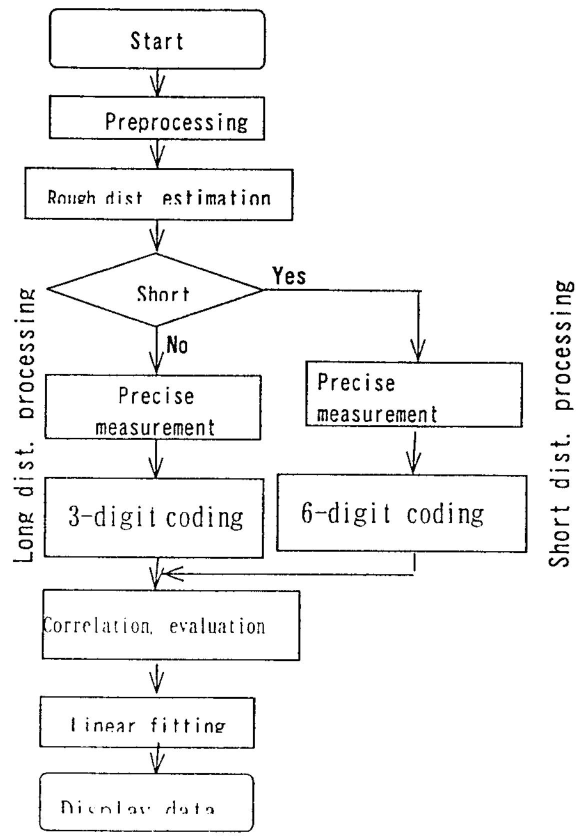


Fig. 2 Flow chart of data processing

(1) Signal treatment
Normalizing the received signal level, and noise elimination.

(2) Calculation of coarse distance.
By using the property of equal spacing of the graduation, estimated distance is calculated from mean separation of the images on the CCD sensor.

(3) Position, width of pattern
Center position and width of images of each graduation bit are calculated.

(4) Coding
Data of pattern widths are classified into categories of 5-or 8-bit codes depending on the estimated distances, and are converted to a series of digital figures.

(5) Correlation
The obtained series of figures and standard pattern of the scale in the computer are correlated. The correlation factors are evaluated by using summation of absolute differences of obtained bits. Judgement is based on the length of compared bits and degree of their discrepancy.

(6) Calculation
After the relationship between the position of the center of each bar on the CCD unit cells and absolute length of CCD are obtained, linear regression fit is operated. As a result, the distance is calculated using the incline and the exact height is derived from the intercept as the center position.

4. COMPOSITION OF MECHANICAL PARTS

Dividing prism for CCD sensor, electronic receiver unit, microprocessor of 32-bit RISC system, and outer case are all newly designed. Also, the staff is made of a newly designed glass fiber resin that intensifies the contrast between the coded bar pattern and white background of the staff for more reliable measurement results.

4.1 Optical part

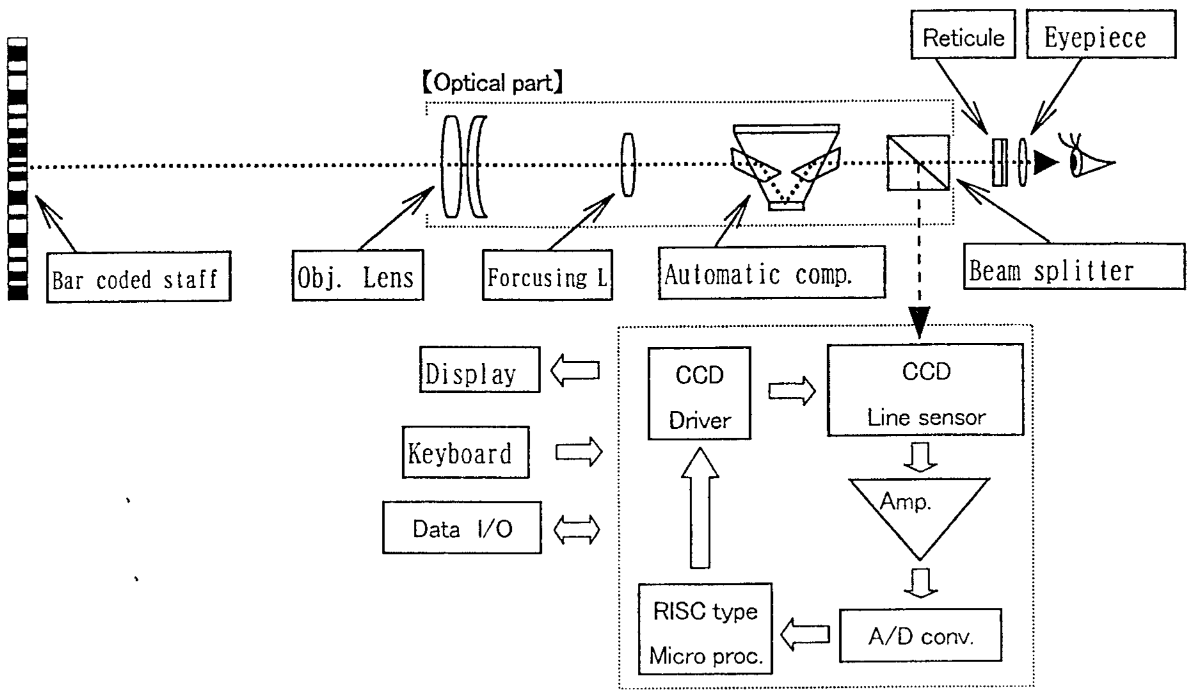


Fig. 3 Hardware composition of SDL30 【Data processing part】

To reduce R&D costs, many of the components used to construct the optical and mechanical parts are the same components used in the current visual model series. The hardware composition of the digital level is shown in Fig. 3. It consists of two main parts: optical devices for imaging and electronic circuits for data processing.

(1) Telescope

The fundamental structure of the SDL30 is very similar to the visual type auto levels. However, the optical system has been improved to project a better resolution in a wider field of view to obtain undistorted images of the staffs.

(2) Beam splitter

The digital level is equipped with a beam splitter that divides the image of the staff from the objective lens through an automatic inclination compensating device to the visual eyepiece and electronic receiving devices. We chose a prism that has high intensity, good characteristics of wavelength separation and minimum disturbance from backside stray light.

4.2 Image data processing part

(1) CCD line sensor

The CCD line sensor utilized in SDL30 is not a special one; it is the same product used in facsimiles or image scanning devices. Light and shade of the graduation image on the CCD sensors are converted to electrical signals and interpreted as digital characters to find width differences.

(2) RISC type micro processor

To shorten the observation time, we implemented a RISC type microprocessor with high speed operation using a pipeline procedure. Recently, the price has dramatically dropped for microcomputers with multiple functions and high performance because they are used for many application in consumer products.

5. SUITED TO VARIOUS ENVIRONMENTS

To achieve the greatest surveying accuracy under various observation conditions, we implemented the following features.

5.1 Brightness

To obtain a linear continuum between input light intensity and electrical signals so that accurate results can be collected, we chose CCD sensors that can work under evening light and in the shadows and under direct sunlight even when the reticle in the eyepiece is difficult to see. The sensitivity of the CCD sensor can be controlled by changing the charging times, and altering the gain of the amplifier. This is done automatically by the microprocessor. As a result, a wide range of input brightness, from 20 to 160,000 lux, can be processed.

Light source	Brightness (lux)
Sunlight	10
Fluorescent L.	15
Sodium lamp	10

Table 2 Minimum observable brightness of different light source

5.2 Emitted energy from light sources

Leveling survey works are carried out under various light conditions, not only sunlight but also artificial light such as fluorescent, Mercury, and Sodium lamps used for night and tunnel work. As seen in Fig.4, the wave length range of energy distribution of sunlight is rather wide, however the main spectra of artificial lights exist within the visible wave length range, so it is necessary to take the different characteristics among light sources into consideration. In the case of SLD30, several types of beam splitters were tested for refractivity and filtering characteristics under various operating environments. Finally, we adopted a beam splitter with a reflectivity of 30% that can distribute enough light to the CCD sensor for accurate measurement under a wide variety of light conditions. It also provides

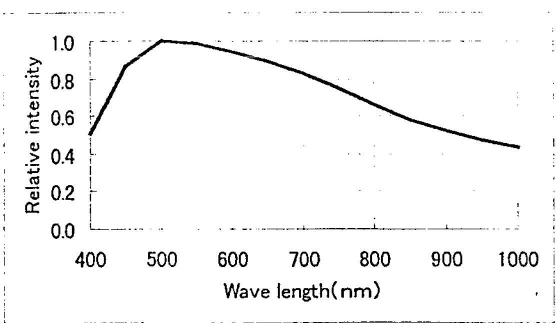


Fig. 4 Spectrum distribution of sunlight.

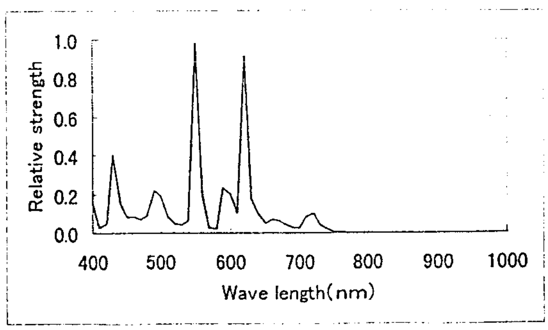


Fig. 5 Spectrum distribution of a fluorescent lamp.

bright and natural images on the visual eyepiece. However, we were still faced with the problem of how to reduce the back light to the CCD from the eyepiece.

5.3 Coping with ground vibration and air scintillation

The accuracy of leveling surveys is greatly affected by ground vibration and air scintillation, which although having quite different causes, have similar effects on the observed image and make it difficult to distinguish data from noise. For this purpose, the following subjects were examined, and among several tested methods, the method containing the shortest processing time was selected.

(1) Investigation of ground vibration

As the fundamental construction of SDL30 is the same as the automatic level, it is important to reduce the effects of vibration on the automatic tilt compensator. To achieve this we investigated the nature of ground vibrations at various different locations, to check the effectiveness of the compensator system.

We made sure that the main frequency of the ground vibration is about 3 Hz., and this is 5 times higher than highest limit of compensating system. ..

(2) Data sampling rate

Vibration noise overlapped data obtained from investigation (1) were resampled by changing the rate. The effective minimum sampling rate necessary to reshape the original waveform was confirmed and the difference between the minimum sampling rate and a quiet condition were evaluated.

(3) Data processing time

The processing method we used for the sampled data is to take the sample mean of a series of data and apply harmonic analysis to find the component. Ground vibration had little effect on measurement results, so the simple mean method with a 0.1 second sampling rate was applied. For smoothing a total processing time of 3 Sec. was achieved.

6. EVALUATIONS

As described above, various countermeasures were applied to SDL30, to confirm those results, evaluation tests were carried out. The performances were as follows.

61 Brightness

Changes in brightness under maximum and minimum test condition were evaluated.

(1) Minimum light intensity

Table 2. shows the minimum amount of light required when working under sunlight, fluorescent light, and Sodium light. The sunlight test was done in the evening, and the instrument was confirmed to work under illumination levels of 20 lux. The artificial light test was carried out by changing the distances from the light sources to the staffs, and the measurement results were confirmed.

(2) Maximum light intensity

We were not able to achieve illumination levels over 150, 000 lux in mid summer under direct sunlight.We were not able to confirm results under maximum conditions. We estimate that maximum illumination levels could be extended to 160,000 lux.

6.2 Shading effect

We carried out tests by periodically repeating light and darkness on the staff, and limiting the visible area of the bar code. Both cases were tested at a distance of 30m. We carried out the same test on another company's similar model as a reference, because there is no standard testing method for shading.

		Reference	SDL30
Grating test (Staffs were illuminated by striped pattern)	12cm pitch	×	○
	7cm pitch	×	○
	6cm pitch	×	×
Masking test (Collimating part was limited)	25cm	○	○
	20cm	×	○
	15cm	×	×

○ : Able :Disable

Table 3 Results of shading tests

(1) Grating test using OHP projection

The staffs were illuminated by OHP light through shading patterns with pitches of 12, 7 and 6 cm, and readability was confirmed. The test configuration is shown in Fig. 6 and results with comparative data are shown in Table 3. The results were obtained from tests of equal pitches of bright and dark. However, no readability was obtained with pitches of 6 cm or less, due to a lack of enough number of coded images.

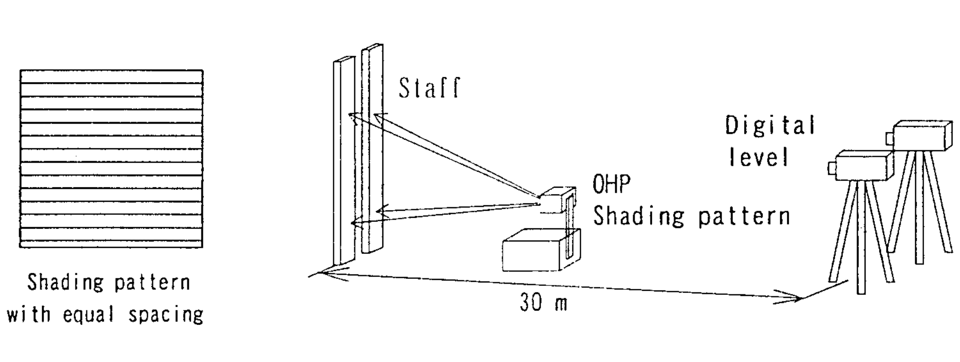


Fig. 6 Seting up for masking test(1)

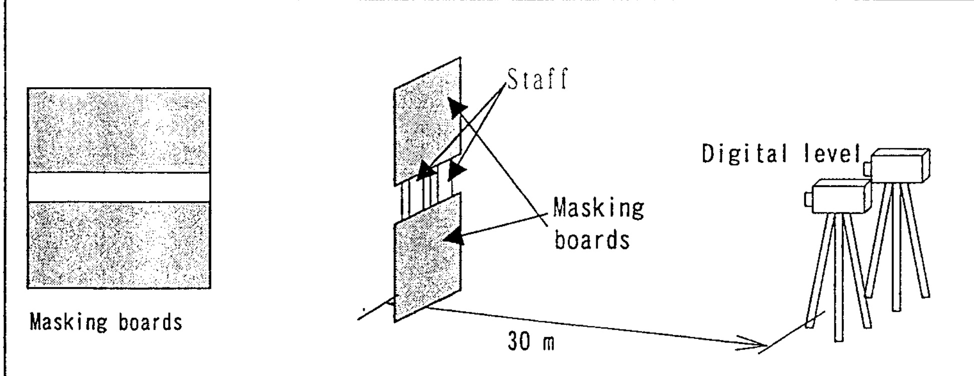


Fig. 7 Setting up for masking test(2)

(2) Masking test

The graduations near the collimation line were masked symmetrically by two sheets of black paper and their separation was changed. The test configuration is illustrated in Fig.7 and the results are tabulated in Table 3. Those tests evaluated the code performance and recognition rate when obstructed. The digital level used for reference showed good performance during the masking tests, but SDL30 performs just as well under even stricter conditions.

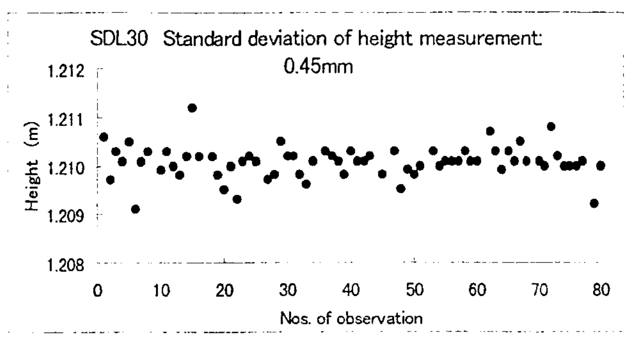


Fig. 8 Example of strong ground vibration.

6.3 Ground vibrations

A roadside, a place constantly affected by strong ground vibrations was selected as the test location. Continuous readings of three sets of digital levels situated in the same location were obtained. Each level was set in the same place, at the same height, and used the same wooden tripods. Distances of 50m were observed in cloudy conditions with low air scintillation.

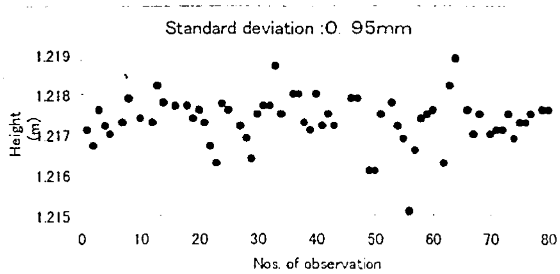


Fig. 9 Reference data of strong vibration. (reference A)

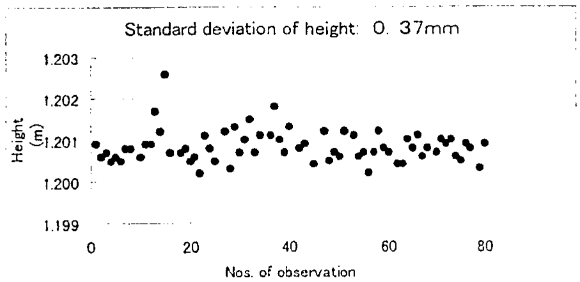


Fig. 10 Reference data of strong vibration (reference B)

Fig. 8 shows the height observation results of SDL30, and Figs. 9 and 10 show the other companies instruments. Looking at the results of SDL30, although the standard deviation is somewhat larger than the reference results; concentration seems to be better. It is clear that the common specification of the digital levels or automatic levels contain some robust data caused by impulse noise which does not abide by the gaussian rule.

6.4 Air scintillation

We chose an asphalt paved road in mid summer under direct sunlight as the test site for

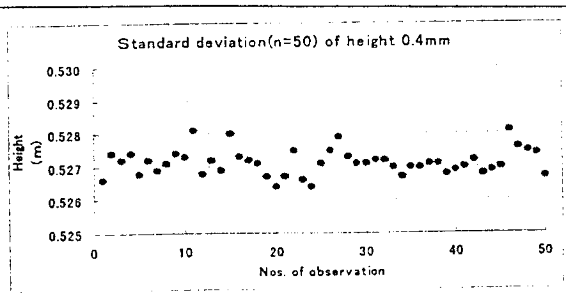


Fig. 11 Example under severe scintillation (SDL30).

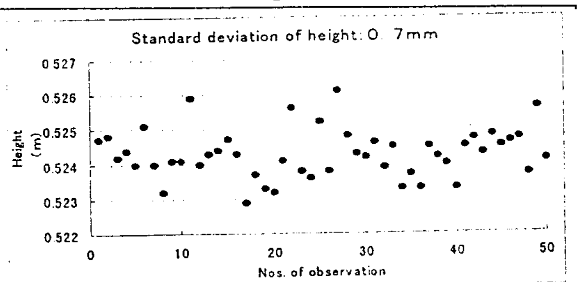


Fig. 12 Reference data of severe scintillation.

air scintillation. Continuous height observation was done on a fine day with no wind, and no traffic. The instrument height was 50cm, and the measuring distance was 90m. As with the tests for shading and masking, the reference instrument was tested alongside the SDL30. Fig.11 shows the results of the SDL30 , and Fig.12 shows the results of the reference level. The results suggest that the multiple reading and simple meaning method is effective in overcoming the scintillation problem.

6.5 ISO test

To evaluate the total performance of the SDL30, ISO DIS12857-1 test method is applied. Many results of different distances showed 0.3-0.5mm per 1 km double run equivalent.

7. CONCLUSION

Test results confirm significantly improved performance of the SDL30, especially in the following two areas.

(1) Wider observable illumination range of 10 to 150,000 lux is achieved. The maximum limit is estimated by calculation.

(2) Observation time is shortened even when using the multiple sampling procedure.

It is very difficult to express numerically the improvements made to the SDL30. Moreover, test conditions do not represent all possible situations in which the instrument will be used. However, from test results we have proved that the SLD30 is more accurate, simpler to operate and more cost effective.

We have succeeded in producing an improved level using field data and user opinion that is easy to use and has a very short processing time. Not only that, as well as leveling it can also be used in the field of industrial measuring and construction monitoring applications.

New automated real-time 3D total station, possibilities and limitations

Method, and test experiments' results with Geodimeter 650 S

Georges Pauchard

Among the total stations on the market, Spectra Precision from Sweden, manufactures 3 different high effective product ranges of total stations :

• Mechanical total stations : Geodimeter 600 M

Total stations equipped with conventional locking and fine adjustment screws for the pointing.

• Servomotorized total stations : Geodimeter 600S

There are no locking screws for the horizontal (Hz) and vertical (V) movements. The operator actions 2 screws (Hz, V) which are endless. This means of course : no loss of time for locking-unlocking, and especially the possibility to introduce a special unit : the TRACKER which allows automatic pointing and follow up on a special active target RMT. Furthermore, the Geodimeter 600S equipped with the TRACKER/RMT by adjunction of radios and a special program, can be robotized.

• Automatic Tracking Station : ATS

This family of products also motorized, is especially designed for the tracking of machinery on sites, hydrographical vessels, deformation measurements, ...

All the Geodimeter total stations are equipped with :

- For the angles : on-line correction of circle graduations and excentricity errors, collimations, axis verticality alterations by help of a dual-axis compensator, horizontal tilt axis error. The read or recorded angles are corrected.

- For the distances : accurate D measurement building continuously the arithmetic average values of the distance measurements, and data computed from the distances. D measurement forms also an average value for the angles.

Geodimeter 600S/650S

In the product family of 600S, the Geodimeter 650S is the most accurate :

- Angles accuracy specified : 3cc (0.0003 gon)

- Distances : 1 mm + 1 ppm.

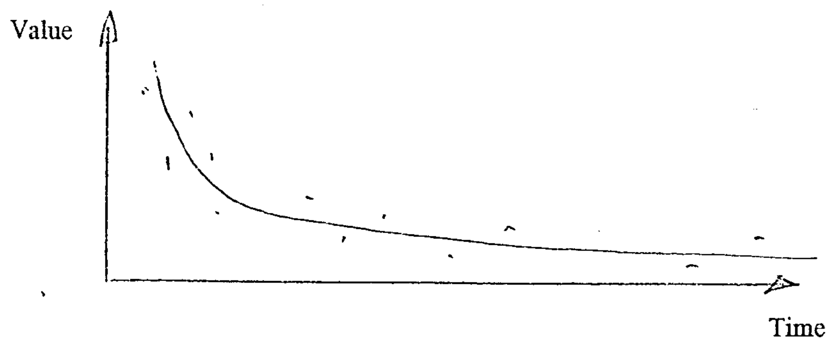
Readings on angles Hz, V : 1cc; and 0,1cc on Hz when using a special high accuracy mode D. Reading on distances : 1mm - and 0,1 mm when using the mode D.

We had the possibility to make some tests with the Geodimeter 650S. They are related here in this paper.

Measuring in D mode

D mode is a continuous measuring mode for the distances. In this mode, the instrument computes and displays (every second) the arithmetic average value of all the measurements made.

The measuring time is at the appreciation of the operator. After generally 7-8 seconds, one reaches the perfect stability of the result.



The last decimals of the distance is pending, for example : 425m,469 - 468, - 467, - 467,

The curve of the results like above shows some kind of "asymptotic" trend, after integrating small variations due to refraction, electronics, ...

Specially concerning the Geodimeter 650 M/S, this instrument has a higher resolution mode and stability on the measurement : 0.1 mm

Typical results in this case :

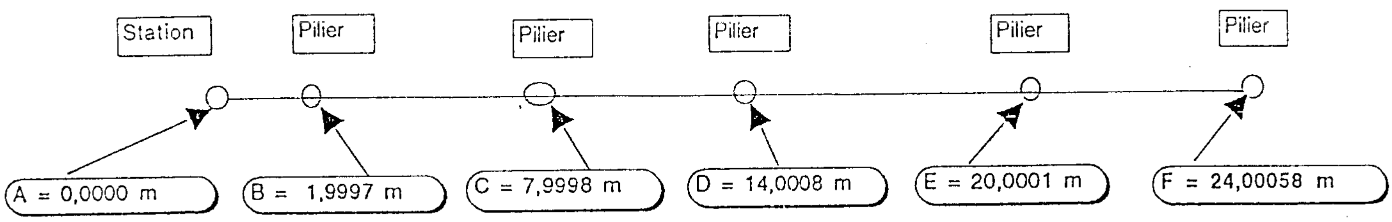
$175^{m},215._{2}$ - $215._{3}$ - $15._{4}$ - $15._{3}$ - $15._{3}$

After remeasuring, the stability/repetitivity of the results on such a distance are normally within 0,1 - 0,2 mm.

The same repetitivity of the results is also reached on the horizontal (reduced) distances and on height differences or directly Z, if we take into account the instrument height and signal height.

This higher resolution combined with differential measuring techniques allow to reach a very good accuracy. Temperature and pressure corrections have to be made carefully since an error of 1°C on temperature gives an error of 1mm/km = 1 ppm.

Checked measurements of horizontal distances were made on a basis measured with invar wires :



Compared Geodimeter 650S results :

C = 7,999.8 m, D = 14.0009 m, E = 20,0005 m, F = 24,0004 m

Repeated measurements gave the same results at 0,1 mm.

Measurements with Tracker and RMT target

The so called "active prism" Remote Target : RMT, sends on IR emission at a special frequency. In Autolock mode, the operator starts the positioning towards the RMT, and when it is in the Tracker's field of view : 2,5 m at 100 m, the Geodimeter automatically locks on the RMT and terminates the pointing. If the RMT is moving, automatically the Geodimeter follows up the target. The Geodimeter locks onto the RMT and nothing else, and is constantly directed towards the RMT.

We can imagine the interest of this device :

- No more pointing. No more focusing and no more screws to turn. The quality of the pointing is not dependant of the operator's eye or skill.

Furthermore the system is able to work with the same accuracy in tunnels, darkness, night, or bad light conditions. Namely some jobs have to be performed during the night, due to heavy traffic conditions.

The Tracker/RMT can be used in 3 distance measurement modes and accuracies :

- Tracking mode for quick continuous measurement (6mm)
- Standard mode for polygonal/network measurements (2 mm)
- D mode for high accuracy measurements (1 mm or better)

Measuring the height differences

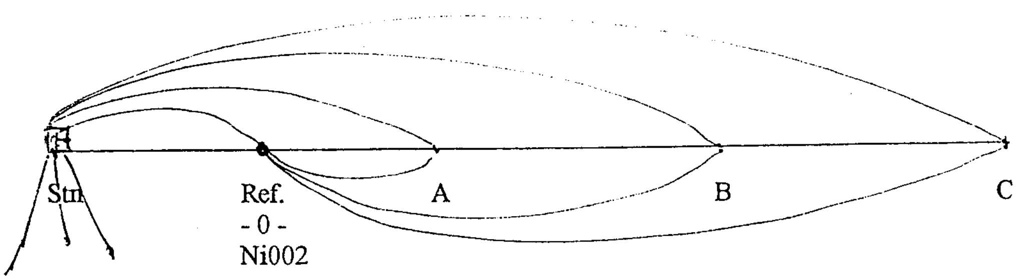
The Geodimeter 600 gives the height differences through trigonometric functions using the slope distance and vertical angle elevation.

In the same way as for distances, the height differences can be obtained in : quick Tracking, standard or high accurate D mode.

We had the opportunity to test the 650S with and without Tracker/RMT in D mode in France in February 1999.

- Outside temperature : 0° - 5°C. Some wind.
- After carefully checking the collimations of the instrument and of the Tracker.

Station



We used the D mode giving 4 decimals on the results : distances slope and horizontal, x, y, z, height differences, ... We took for reference and comparison the very accurate Zeiss Ni 002 level and invar levelling rods

Differences with	_ h ref.	A (73m)	B (138 m)	C (215 m)
Measurements with the level Ni002 : ref. value	0	0	0	0
Classic measurements with the operator's eyes	-	0.2 mm (0.1)	0.3 (0.5)	1.0 (0.5)
Measurements with Autolock/RMT	-	0.1 mm (0.1)	0.4 (0.3)	1.0 (0.5)

(xx) = max. dispersion between the measurements.

As we see above, agreements of 0.1 mm (at 73 meters), 0.4 mm (at138 meters), 1.0 mm (at 215 meters) were obtained compared with the obtained values with the Zeiss Ni002. They correspond well to the manufacturer's specifications , and are even better.

Specified angles 3cc (0.0003 gon), distances : 1 mm + 1 ppm accuracy.

Furthermore, the technique of continuous automatic locking on special RMT targets sending out modulated IR light, together with accurate and/or quick angles and distance measurements on prism reflectors can be used for Position Monitoring, Machine Control, Position Tracking. This family of equipments developped by Spectra Precision called ATS : Automatic Tracking System, is also a useful contribution to field measurements.

Active infrared RMT and motorized levelling

The RMT active target exists en 2 versions :

- a plane RMT emitting IR in an angle of 30 grades and a 360° RMT working all around.

These are especially interesting for the trigonometric height measurement with 2 total stations looking at each other simultaneously. One RMT, plane or 360°, is mounted on top of each Geodimeter 600 facing each other.

The advantages of the use of Geodimeter 600 Autolock (or Robotic) with RMT, compared to the use of ordinary total stations are :

• The pointing on the RMT is automatic; in this way not depending on the operator, thus of a more constant quality.

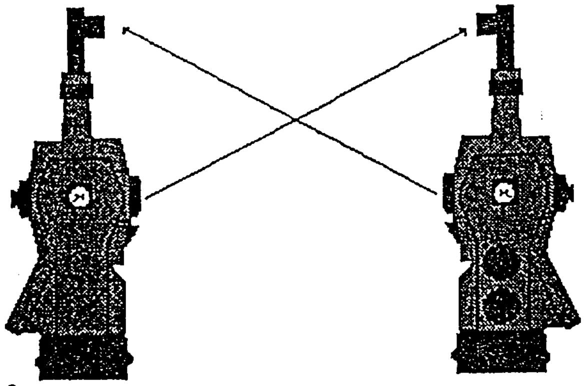
• The RMT leads the pointing for the vertical angle, by means of the IR diode, emitting at a special frequency, eliminating the risks for bad pointing or errors pointing on other prisms, reflective targets or car cat's eye .....

• Furthermore, if we give information for automatic search of the RMT for the Geodimeter 600 with Autolock, and all the more for Geodimeter 600 Robotic, there is no need for coarse pointing, the Geodimeter 600 will find iself the RMT.

By means of the built-in radio link, and its detachable control unit, the Geodimeter 600 Robotic can be controlled by the car driver himself.

The car driver can also read and check directly on the keyboard the height differences, and type codes, point numbers, ...

There is a genuine speed up of the whole process, and with less needed people. Thus allowing more rentable and quicker operations of motorized levelling.



GPS and Geodimeter 600

Another very promising and interesting development in progress made by Spectra Precision is the adaptation of a GPS together with a total station Geodimeter 600. It was presented at the Intergeo exhibition in September 1998 in Wiesbaden (Germany).

One can easily imagine that the whole equipment - total station + GPS - giving trigonometric height determination + GPS height determination, will be a very useful tool for the user.

Limitations and cautions

Of course, we have to be cautious with some facts like :

→ Do not mix the resolution/reading : ex. 4 decimals in x, y, z, with real accuracy. But the clever surveyor knows that by mean of differential measurements, one can improve the accuracy of the results.

→ Be cautious with the refraction. The manufacturer Spectra Precision uses in the system an average value for refraction, K = 0,142. Another K value can be used and through a calculation ppm can be entered into the system.

The disturbances generated by refraction are of the same type as for the direct levelling instruments.

→ Some difficulties are created by the interruption of the line of sight during the Autolock mode and D mode. The measurement has to be restarted.

→ For accurate measurements, do not use the instrument height and signal prism height. The Geodimeter horizontal axis height/trunnion axis can be determined by measuring on a prism located on a reference fix mark point known in z, via a special program called IZ/Z.

→ Check regularly the collimation (Hz, V) of the instrument tilt axis error, and the Tracker system. The errors are stored. After that, the instrument correct the values read or recorded for these errors.

→ Do not use D mode for measurement on moving target. The average value has no sense in this case.

The Development of levels during the past 25 years, with special emphasis on the NI002 optical geodetic level and the DiNi® 11 digital level

(By Matthias Menzel)

1. The History

The history of Germany after World War II is manifest in the history of a number of German companies, including Carl Zeiss. The development of levels is a case in point. After the war, the political situation led to the existence of two companies bearing the name of Carl Zeiss - one in Jena, East Germany, the other in Oberkochen, West Germany. Each of them designed, manufactured and sold surveying instruments levels independently of one another, and not exactly in an atmosphere of mutual friendship. Things changed after the German reunification in 1990. The reunited Carl Zeiss took advantage of the chance to concentrate efforts and develop technically advanced instruments such as digital levels.

My brief historical outline will cover the past 25 years of level design at Carl Zeiss. For better understanding we will occasionally cast a glance further back [6],[7].

From 1973, Oberkochen developed such instruments as the Ni 22 (1966), the Ni 20 (1972) and the Ni 3 (1980). They are all simplified versions of the Ni 2 of 1950, the first engineering level to have a compensator pendulum which automatically keeps the line of sight horizontal. Until today, in 1999, customers in need of a level providing geodetic accuracy have been specifically asking for a Ni 2 - a request that could be satisfied for many years.

With the attached parallel-plate micrometer, the Ni 2 allowed the mean error per kilometre to as low as 0.3 to 0.4 mm. An improvement on this was achieved with the Ni 1, another automatic geodetic level launched in 1967. The difference was in the pendulum suspension: it had crossed steel ligaments instead of the traditional four-bar linkage. Telescope power was increased to 50x, and the micrometer was integrated into the instrument.

In parallel, Oberkochen developed the Ni 4, an automatic construction level launched in 1980. A successor of the Ni 42 of 1971, the Ni 4 now had a conventional levelling base with footscrews. Not to forget, 1976 saw the launching of a non-automatic "bubble" level Ni 52 for the building industry. The routine levels Ni 30, 40 and 50, with medium and lower accuracies, were added to the line in 1991 and are still part of it.

In the period from 1973, quite a number of levels were designed in Jena too. First and foremost, I should mention the NI 002 of 1973, an automatic geodetic level of superior precision. I will deal with it in detail later in my talk. In the years after, a number of further automatic levels were created, such as the NI 020A (1982), the NI 005 A (1983) - actually a NI 020 A with integrated micrometer - and the low-end NI 040 A (1983). Also in 1983, the NI 021 bubble level was launched. The series A levels, from NI 005A to NI 040 A, were accuracy-graded to match different user requirements in the medium-to-low accuracy range. Flexibility in application was ensured by a wide range of accessories. Manufacture of the same series, with a slightly changed styling and a different name, is continued by a company that is a legal successor to the former state-owned "Carl Zeiss Jena" enterprise.

The NI 002 soon came into widespread use. Encouraged by the favourable response, Jena presented two further levels in 1988 forming a design series: the NI 002 A and the RENI 002A. The NI 002 A was a strictly optical level with almost all functions identical to the NI 002. The RENI 002 A, having the same accuracy, was the first step towards a semi-automatic level, with optical staff reading entered into an on-board computer, and digital micrometer reading - features which made operation much more convenient. What remained for the operator to do was to manually align the telescope crosshairs with the staff graduation. This semi-automatic technique was the state of the art until 1990, when Leica presented the world's first digital level, the Na 2000.

That was the time when not only the two Germanys but also the two Carl Zeiss companies in Jena and Oberkochen were reunited.

The reunited Zeiss made an important decision: to start a development effort for a digital level in 1992. The Jena R&D team had already gained some experience in the years between 1983 and 1985, when the Dresden University of Technology did research into digital levelling for them under a contract. The digital levels DiNi®10 and DiNi®20 were launched at the 1994 Intergeo. A year later, the first digital levelling total station, the DiNi®10T was presented to the surveying community in Dortmund. An upgraded series of digital levels comprising DiNi®11, DiNi®21 and DiNi®11 T were added to the range in1996. Essentially, the upgrading consisted in the use of a PC card as a memory medium for the DiNi®11 and DiNi®11T models, and an increased speed of measurement.

For the sake of completeness, I should mention the development the Ni 10 in 1994, an optical level which has always remained in the shadow of the Ni 2.

2. The NI 002 (NI 002 A / RENI 002 A) Geodetic Level

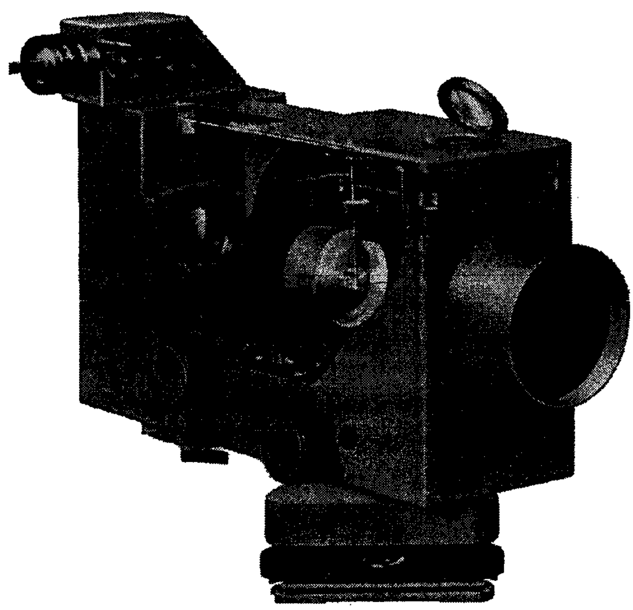


Figure 1: NI 002 A

When the NI 002 was presented to the surveying community in 1973, nobody foresaw that this opened a new chapter in geodetic levelling. Despite probable conjecture to the contrary, the instrument had not been designed with suitability for *motorized levelling* in mind, although tests with motorized levelling, using other levels, had been made already in the late sixties, by a team headed by Prof. Peschel of the Dresden University of Technology. The NI 002 was designed to satisfy the most exacting demands of height transfer in general. The fact that it also met the requirements of motorized levelling was merely, and almost incidentally, an added advantage of the concept. A detailed description of the NI 002 family and their performance parameters can be found in the literature [4, 5 and 1]. Let us focus here on those features of the instrument that bear upon its scope of applications.

The instrument's well-proven accuracy of $\pm$ 0.2 mm/km is achieved by its design concept, which includes the unique reversing mirror compensator (with the pendulum mirror suspended at half the focal distance, and measurements made with the mirror in an initial and a reversed position), the designed-in accuracy of this compensator, system focusing by means of shifting the pendulum mirror, and an objective micrometer. These elements provide what is called a "quasi-absolute horizon". The mean of the two readings is nearly independent of the distance between instrument and levelling rod. The features described eliminated the need to keep the backsight and foresight distances exactly equal and raised horizon stability to a new level of quality. With the NI 002 it is thus possible to carry out precise lines of levels without equalizing backsight and foresight distances to within 10 cm. The greater freedom of instrument stationing is an advantage also in industrial applications. Before, precise area levelling was only possible with relocating the instrument several times, while the NI 002 and its successors can remain at a single station, from which sightings can be taken to targets at different distances.

3. The DiNi®11, DiNi®21 and DiNi®11T Digital Levels

3.1 Instrument features

Comprehensive descriptions of the Zeiss digital levels, their mode of operation and accuracies are given elsewhere [2 and 3].

In this context I want to point to some particular features. For measurement, the DiNi® needs a staff segment of only 30 cm, which is frequently an important advantage under practical surveying conditions. This type of staff reading is very close to the common classical method. The user interface with its four-line graphic capability display and the extensive key panel leave nothing to be desired. The on-board software includes all surveying methods you can think of, and different language versions are a matter of course. Users of instruments purchased some time ago can order updates so as to keep their software at the latest state. The DiNi®11 and DiNi®11T models have PC cards as memory media, allowing the user to work on any number of projects (i.e. with any number of data files).These features have meanwhile earned the DiNi® a high reputation worldwide.

Figure 2: DiNi®11 T Digital Level with horizontal circle

Frequently, questions are raised on the accuracy of height and distance measurements as a function of the sighting distance, also in connection with the use of the DiNi®11T (DiNi®10T) with expanded staff segment. The DiNi®11T (DiNi®10T) is used with a staff segment of 100 cm for greater distance accuracy. Height reading, however, still uses the 30 cm segment only.

Questions about the obtainable accuracy arise where digital levels are to be used not only for lines of levels but also for structure monitoring and special measurements. It is difficult to make generally valid statements on accuracy, because it depends on various parameters. The next graph illustrates the accuracy of a single height measurement, determined from repeated sightings, as a function of distance.

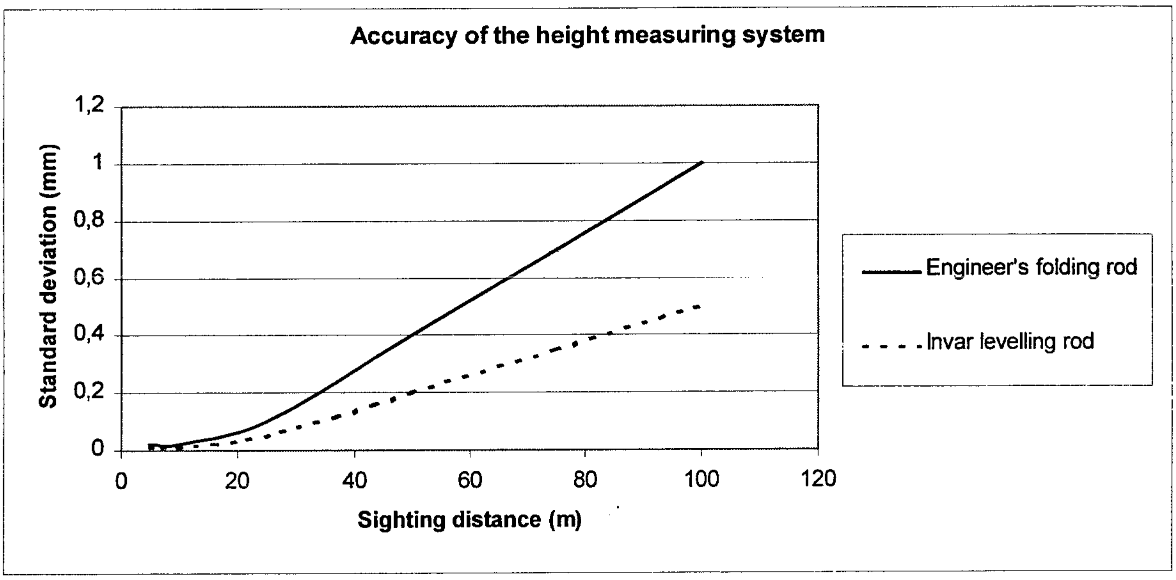


Figure 3: Obtainable accuracy of the height measuring system as a function of sighting distance and type of staff, determined from repeated sightings

Whether these accuracies can be achieved in a specific surveying job in a specific field situation remains to be analyzed by the user. Factors to be considered include the different sighting distances to be used, and the accuracy to which the line of sight can be adjusted.

The next illustration shows the distance measuring accuracies obtainable with the DiNi®11 and DiNi®11T instruments and the two different staff types.

These accuracies can also be described as follows:

DiNi®11T with Invar staves: $\sigma = 0.5\ D * 0.01$ (D in m)

DiNi®11 with Invar staves: $\sigma = (0.005 + 0.00002\ D + 0.00003\ D^2) * 100$ (D in m)

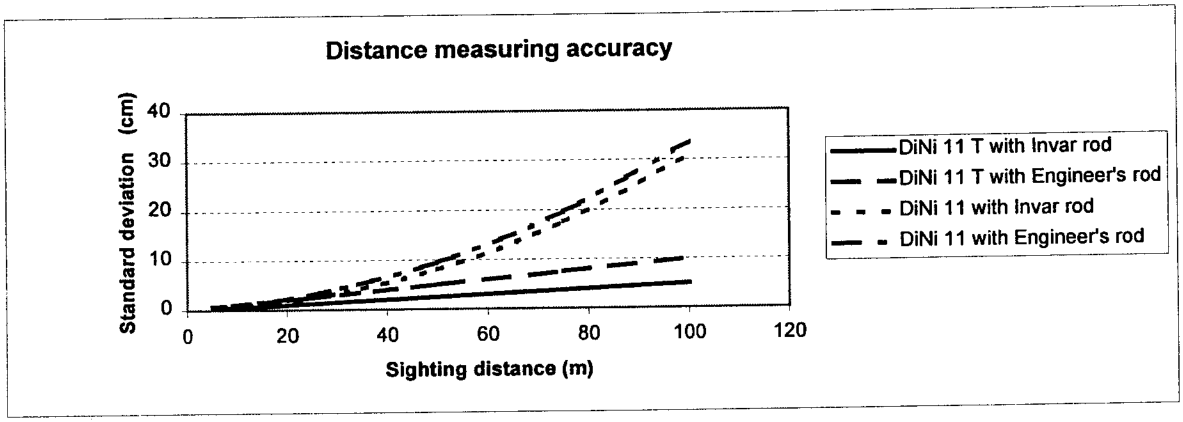


Figure 4: Obtainable distance measuring accuracy as a function of sighting distance and staff type

3.2 Remarks on using digital levels

While the DiNi® has a highly accurate electronic staff reading system, its mechanical and optical design is that of any other ordinary level and quite unlike that of the NI 002. Digital levels are therefore prone to influences known from other precise levels of earlier make.

One of the parameters to be considered is the instrument's temperature behaviour. Given the existence of an on-board computer with memory and an automatic reading system, it suggests itself to make allowance for the inclination of the line of sight with temperature (known as temperature response), and correct it. This is possible provided that this effect remains constant throughout the instrument's lifetime, and provided the availability of a simple function that efficiently corrects the error. The deviation of the line of sight from the horizontal due to temperature has various reasons. The error determined by the user or manufacturer results from different factors, such as the accumulated effect of thermal expansion coefficients of the various materials involved (glass, metal, plastics). Rarely ever can the individual causes of the temperature response be singled out. The history of instruments has seen mechanical temperature compensators, in which different materials were used deliberately to move some element in a staff reading raypath and thus produce an effect counteracting the temperature response. Attempts were made in the past to prevent temperature responses in precise levels from exceeding 0.3 " to 0.5"/K. Given the capabilities of the digital level, these effects can now be reduced even further. This has been accomplished for the DiNi®11 and DiNi®11 T levels. Fig. 6 shows, for example, the temperature responses for a DiNi®11 with correction.

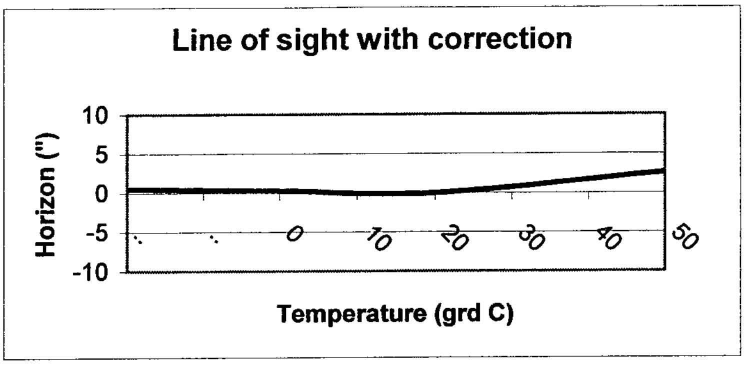


Figure 5: Temperature responses for a DiNi®11 with correction

The presently used correction procedure is backed by investigations that involved measurements at a great number of temperature intervals and included extremely stressed instruments to simulate a long lifetime. The validity of the correction is checked by consistent quality inspection. One should mind, however, that the correction works only with an instrument that has had time to adopt the ambient temperature. There is no possibility to shorten this time. The thumb rule still applies that the waiting time for precise levelling should be 2 minutes per degree of temperature difference. Temperature balance may be achieved slightly faster in this or that instrument, but this does not lead to any significant differences.

Another problem to be always considered in "normal" precise levels is the residual compensator error. The pendulum movements in precise levels are almost linear in the range in which the pendulum is to correct the instrument's inclination. Manufacturers take every effort to make the pendulum set to the horizontal as precisely as possible. Despite adjustment to a pendulum factor of 1 (for both the mechanical and optical effects), and despite meticulous care in designing and assembling the pendulum and the vertical axis system, a tiny angular error of 0.1 to 0.15" may occur between foresight and backsight.

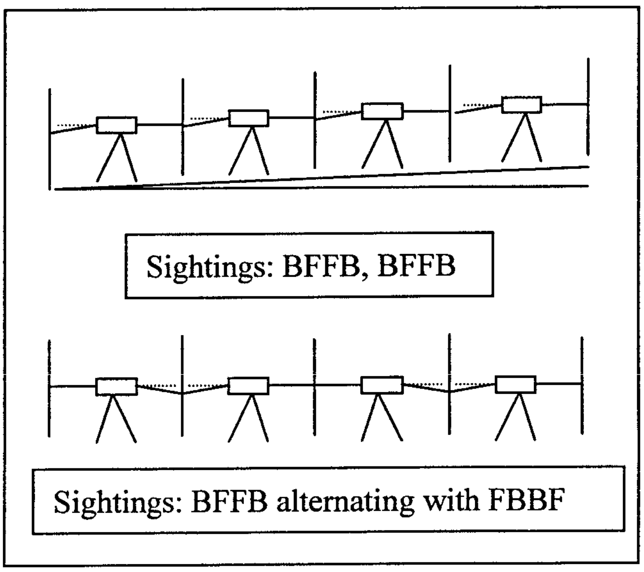


Figure 6: Measurement methods

Provided that lines of levels are run with consistent procedures, this angle constitutes a systematic error. A residual compensator error of 0.2" means a misclosure of a circuit of levels of about 1 mm. The advantages of the digital level (fast measurement, no subjective errors) help identify this error better than ever before. In the past, these errors were part of the random error and could not be ascertained, while now they can be found out even if they are very small. The conclusion to be drawn from this is that precise levelling jobs with a digital level of known make should in any case be performed by the alternating method (back/fore/fore/back - fore/back/back/fore), also known as the "red pants" method.

The accuracy obtainable with a level is implemented via the correct horizontal alignment of its line of sight. It is impossible, however, to keep the line of sight absolutely stable over many hours of work and at any temperature. It is therefore necessary to check and correct the line of sight from time to time, especially before running precise lines of level under conditions with heavy temperature fluctuations, and after excessive mechanical stresses. In optical precise levels known so far, users have made measurements by the familiar methods (Förstner etc.) and shifted the crosshairs to the nominal reading by a small amount. The crosshair line thickness is about 2" to 3". Accordingly, the changes and settings made with an optical level in the past must have been in the order of several seconds of arc. With digital levels, the digital form of results gave rise to some uncertainty, as there is no experience with the necessary amount of correction in digital terms. In the digital level, adjustment of the electronic horizon is made by a computed off-centre position of the linear CCD array, which plays the part of the crosshairs in the electronic system. Where the instrument is used for optical measurements, the crosshairs can be shifted to the nominal staff position after the electronic adjustment. A graphic representation of the changes observed against time provides a good overview and helps to make the right decision. Adjustment measurements repeated in succession yield differences of 2" to 3" under normal ambient conditions. With measurement under identical conditions, changes from day to day should not exceed 3" to 5".

3.3 Use of digital levels for special jobs

Digital levels can be used for checking and monitoring heights on structures and other objects. The DiNi®11T (DiNi®10T) with its digital horizontal circle is particularly suitable for such jobs. Two stepping motors can be attached to drive the lateral slow-motion and focusing screws. A control computer can be programmed for taking sights and readings at any number of staff segments fixed at the structure points to be checked. After program teach-in by carrying out a manual cycle of measurements, the computer will repeat the cycle any number of times at user-determined time intervals. The computer releases the instrument and recording functions via the RS232 interface. The necessary equipment configuration is available from manufacturers collaborating with Carl Zeiss.

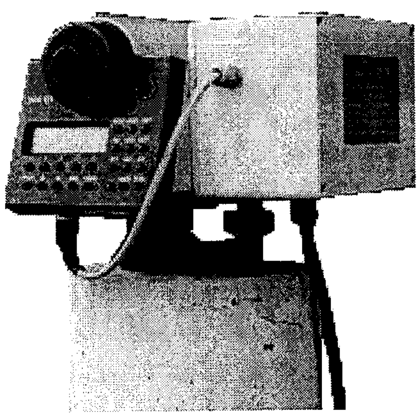


Figure 7: DiNi®10T modified for the monitoring of structures

4. Comparison between NI 002 and DiNi®11

From the explanations given it is obvious that the NI 002, by its very design, has the edge over the DiNi®11 on accuracy, whereas its lack of automatic reading relies heavily on the operator's concentration and involves the risk of subjective error.

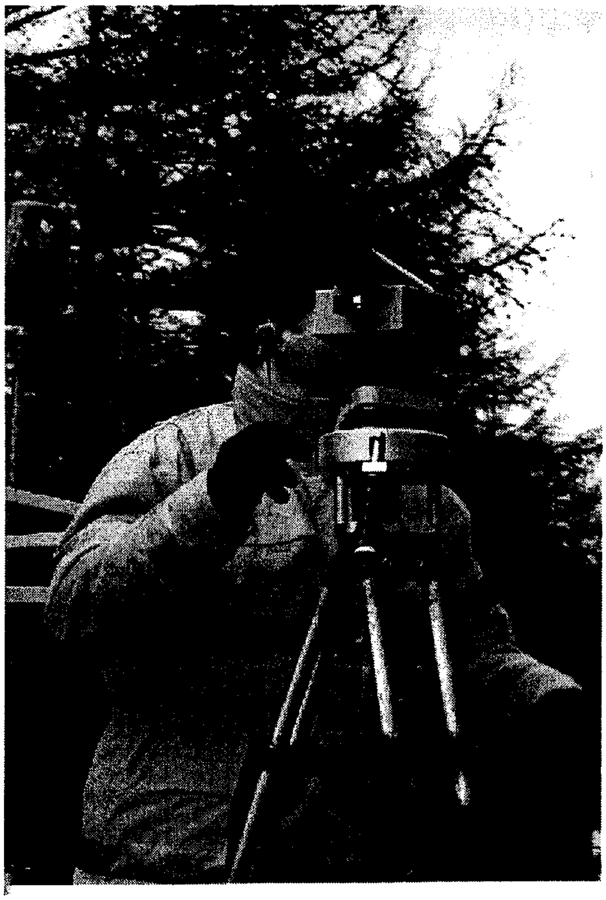


Figure 8: A DiNi in field use, Denmark, 1998

In addition to digital reading, digital levels have the decisive advantage that they register the complete data in a form capable of electronic processing and in a minimum of time. The results are less affected by staff subsidence and similar effects. Since the digital level brought automation to levelling procedures, many users have come to appreciate its capabilities for smooth, reliable and accurate surveying, the more so as does not take days to learn its operation. Although accessories such as the right-angle eyepiece are available and, in fact, employed in *motorized levelling*, digital levels are still little used for this method, compared to the NI 002 and its successors. This is for reasons that have little to do with the different capabilities of digital levels. On a worldwide scale, the present demand for large precise levelling projects is very low. In addition, motorized levelling takes too much manpower - a factor that will probably lead to a future phase-out of the method. A third reason is that historically developed attitudes towards *motorized levelling* in general differ from country to country. Some countries do not use it at all.

5. Summary and Outlook

After a look back on the history of level designing at Carl Zeiss in the past 25 years, I have discussed some technical details of the NI 002 and DiNi®11 instruments and their applicability, including their usefulness for the well-known method of *motorized levelling*.

Although they differ by their technical capabilities, both instruments can be used for *motorized levelling*.

The new DiNi® digital levels, which almost come up to the accuracy of the NI 002, make levelling jobs considerably easier for the surveyor. Their data logging, storing and processing capabilities and the elimination of subjective errors are of crucial importance to all applications of precise levelling. In top-precision surveying jobs involving widely differing sighting distances, the present digital levels cannot replace the NI 002 family.

References:

[1] Feist, W.: Das Präzisionsnivellier Ni 002A - ein Instrument für hochgenaue Höhenbestimmung in der Meßtechnik. AVN 2/1994. S. 56-63.

[2] Feist, W., Glimm, A.., Marold, T., Rosenkranz, H.: Die Total Level Station DiNi®10 T - Das erste digitale Nivelliertachymeter. VR 58/1. S. 1-7.

[3] Feist, W., Gürtler, K., Marold, T., Rosenkranz, H.: Die neuen Digitalnivelliere DiNi®10 und DiNi®20. VR 57/2. S. 78-89.

[4] Hüther, G.: Das neue Präzisionskompensatornivellier NI 002 des VEB Carl Zeiss Jena. Jenaer Rundschau. Sonderheft zur LFM 1973. Jena 1973. S. 56-60.

[5] Hüther, G.: RENI 002A und NI 002A - zwei neue Präzisionskompensatornivelliere. Jenaer Rundschau. 1988. Heft 4 S. 160-164.

[6] Jenaer Rundschau. 1983 Heft 1. Beilage der Jenaer Rundschau 75 Jahre Zeiss-Jena-Vermessungsgeräte.

[7] Köhler, H., Meier, H.-K.: 30 Jahre Forschung und Entwicklung im Zeiss Werk Oberkochen, Kapitel 7.6 Geodätische Instrumente. Carl Zeiss Oberkochen 1990.

[8] ZEISS Vermessungs-Instrumente. Firmenkatalog Nr. 520431 CARL ZEISS Oberkochen/Württ. 1969.

Address of the author:

Dipl.-Ing. Matthias Menzel, Carl Zeiss Jena GmbH, Geschäftsbereich Geodätische Systeme, Tatzendpromenade 1a, D-07743 Jena, Menzel@Zeiss.de .

Experience with various digital levels in both motorised and conventional „on foot“ precise levelling in east Germany

Prof. Dr.-Ing. Wolffried Wehmann

Hochschule für Technik und Wirtschaft Dresden (FH) - University of applied sciences, Germany

1. INTRODUCTION

The Swiss instrument manufacturer ***Leica*** introduced the first digital level, the **WILD NA 2000** in 1990 after discarding a number of prototypes, for instance reference can be made to a comparable development at the Technical University of Dresden in the mid-eighties. The introduction of the NA 2000 heralded a new phase in the development of efficient levels because it was the first instrument to allow the automatic measurement and storage of levelling data. The high degree of automation thus achieved resulted in very large gains in productivity in the determination of heights. Leica introduced the first digital precise level, the NA 3000 at the end of 1991. Two further high precision levels were introduced by other manufacturers in 1994: the German firm ***Carl Zeiss*** with the **DiNi 10**, designed and produced in Jena, and the Japanese firm ***TOPCON*** with the **DL 101**. Meanwhile all three of the above-mentioned firms have subsequently made further developments in their high precision instruments. The **NA 3003** with the software version 4.3, the **DiNi 11** and the **DL 101C** are considerably better than their predecessors. At the German „Geodätentag“ in 1997 the Japanese firm **Sokkia** as the fourth manufacturer presented its own digital level, however for the time being only a level of medium precision. The large number of these and various digital engineering levels sold indicate that digital levels have achieved a wide acceptance and have replaced optical levels.

All the above mentioned digital levels have been investigated under practical and field conditions in the Faculty of Surveying and Cartography at the Hochschule für Technik und Wirt-schaft Dresden (FH) - University of Applied Sciences - over the last 7 years. The results of the investigations of the latest versions of the precise digital levels of the firms Leica, Carl Zeiss and TOPCON will be presented in the first part of the paper. The three precise digital levels have been repeatedly and thoroughly tested under practical and comparable conditions, e.g. during the measurements to determine the heights of two levelling networks in the south of Dresden. The following levels were used during the investigations: five Zeiss DiNi 10, two DiNi 11, six Leica NA 3003 (Software version 4.2), one TOPCON DL 101C, one DL101 and one DL 102, all instruments with their precise equipment, in particular with invar staves, produced by the German firm NEDO.

The digital levels were tested in respect of their accuracy, reliability, ease of use and in respect of the influence of external factors on the levelling results. As far as external factors are concerned, particular attention was paid in the investigations to the influence of:

- changes in temperature on the stability of the line of collimation,
- vibration and influences of wind on the reliability and accuracy of the results,
- various lighting conditions and shadow patterns on measurement ability and on the results,
- partial obscuring of the staff and staff inclination.

2. CONVENTIONAL MEASUREMENTS IN A PRECISE LEVELLING NETWORK

The true quality and reliability of levels can be best judged by the results of precise levelling to determine a free network. In the south of the city of Dresden, in Dresden-Leubnitz there is a suitable levelling network to test the digital levels. The network consists of 28 accurate benchmarks, 41height differences and 5 km of levelling. It contains steep sections (maximum slope 15%), level sections and sections along busy streets. The section length varies between 30 and 500 metres, with an average of 130 metres. The network exists to monitor movements of the "Leubnitzer Kirchhang" slope and the subsidence of a number of buildings in the area. For this reason it was only connected to one benchmark outside of the zone of movement. It has been levelled twice a year by the Dresden University of Applied Sciences since 1996 and is also used as a network for the testing of new digital levels. The double levelling method of measurement after FÖRSTNER has consistently been applied. Measurements are taken in multi-measurement mode (n>4) in order to achieve the required standard of accuracy of

σ_h **= 0,50 mm/km.** The measurements taken in April 1997 and April 1998 involved the use of two levels from each manufacturer under comparable conditions in order to evaluate the quality of the instruments.

	Initial Levelling	1. Relevelling		2. Relevelling *	3. Relevelling		4. Relevelling
Used level	1 DiNi10	1 DiNi 10	1 NA3003	1 DiNi10 +. 1 NA3003	1 NA3003	1 DL101C	2 DiNi10
Observation Method	$\overline{X}$	$\overline{X}$	$\overline{X}$	$\overline{X}$	Median	$\overline{X}$	$\overline{X}$
Measurement period	August 1996	April 1997	April 1997	September 1997	April 1998	April 1998	September 1998
s_{1kmDN} from differences of double meas.	0,60 mm/km	0,38 mm/km	0,47 mm/km	0,43 mm/km	0,33 mm/km	0,45 mm/km	0,37 mm/km
s_{1kmDN} from the net adjustment	0,79 mm/km	0,59 mm/km	0,65 mm/km	0,96 mm/km	0,47 mm/km	0,62 mm/km	0,60 mm/km
rate of repeated meas.	10 %	12 %	10 %	14 %	0 %	10 %	8 %

Table 1: The results of the precise levellings in the level network Dresden-Leubnitz

*) The measurements were carried out during a practise of students by the 6th semester. These students had no practical experiences with precise levelling until this practise.

Table 1 shows that all of the Leica, Zeiss and TOPCON precise levels used fulfilled the accuracy requirements of conventional precise levelling without major problems. No significant differences in efficiency or accuracy were apparent between the determinations with the NA 3003, DiNi 10/11 and the DL101C. The standard deviation per Kilometre of levelling as determined from double measurement differences lay between 0,33 mm/km and 0,60 mm/km and confirmed the instrument makers' specifications. The network adjustment with only one fixed point resulted in accuracies between 0,47 mm/km and 0,96 mm/km being determined. It should be noted that, independent of the level used, the results obtained in September and in particular at the end of August 1996 were at least 0,2 mm/km poorer than those obtained in spring. This was caused by the unfavourable weather in late summer: the levelling was carried out both in August 1996 and September 1997 mainly in sunny weather and ambient temperatures over 20°C, so that particularly the influence of refraction is responsible for the poorer accuracies achieved in these periods. The summarised results in Table 1 also show that the measurement mode **"Median"**, which has only been available in the NA3003 software up to now, delivers considerably more reliable results than the measurement modes **"Mean"** or **"Mean with predefined standard deviation"**. This is proved not only by the better accuracies, but also by the reduced necessity of repeat measurements. The calculation "Median", which uses the central value of a series of **n** measurements, reacts less sensitively to outliers (values which differ markedly from the others) in the series than does the mean calculated from the same measurements. The NA3003 software displays the bandwidth $x_{max} - x_{min}$ at the same time, so the user has a value analogue to the standard deviation in "mean" mode as an aid to quality control, to prevent problems by taking repeat measurements when indicated. Considerable investigation has lead the author to suggest that the "Median" mode should be preferred when precise measurements are being made with the NA3003 (as from software version 4.2). At least five observations should be carried out and measurements with a bandwidth exceeding 0,60 mm should be rejected and repeated.

As far as efficiency is concerned, the speed of measurement with the DiNi 10 and DiNi 11 instruments was 5 – 10% higher than the speed obtained using the NA3003 and the DL 101C, both of which gave almost identical performances. This agrees well with investigations in other height nets and comparative investigations at the Dresden University of Applied Sciences.

A fundamental problem in the determination of height networks of the highest precision became apparent in the investigation of the network adjustment results. Statistical evaluation shows that the normalised corrections of most of the adjustments are not normally distributed, as confirmed by figure 1. Where the modes "Mean" and "Median" were used, significantly more measurements respectively Normalised corrections are found in the region of the expected value than would be postulated by the Gaussian normal distribution. In addition, the occurrence of normalised corrections with differences of 1σ to 2σ is significantly less frequent than predicted by the Gaussian distribution function for normally distributed corrections. By contrast, if the mode "Mean" is used, more values occur in region 3σ than would be theoretically expected. This indicates the regular occurrence of so-called "small outliers" which could, for example, be explained by systematic collimation falsifications in the very short observation periods of the digital levels. Where the measurement mode "Median" in the NA3003 is used these "small outliers" occur much less frequently, so that the distribution function of the normalised corrections fits the Gausssin normal distribution more closely.

3. INVESTIGATION OF THE INFLUENCE OF EXTERNAL FACTORS ON THE DIGITAL LEVELS

Apart from the comparisons of the instruments under investigation under comparable conditions, not only in the Dresden-Leubnitz height net, a number of special tests were carried out to determine the effect on measurement accuracy and reliability of the external factors mentioned in Section 1. Evaluation of the software feature was also carried out. The main thrust in these investigations was the behaviour of the digital levels, and the accuracy and reliability of measurements performed with them, under typical practical conditions.

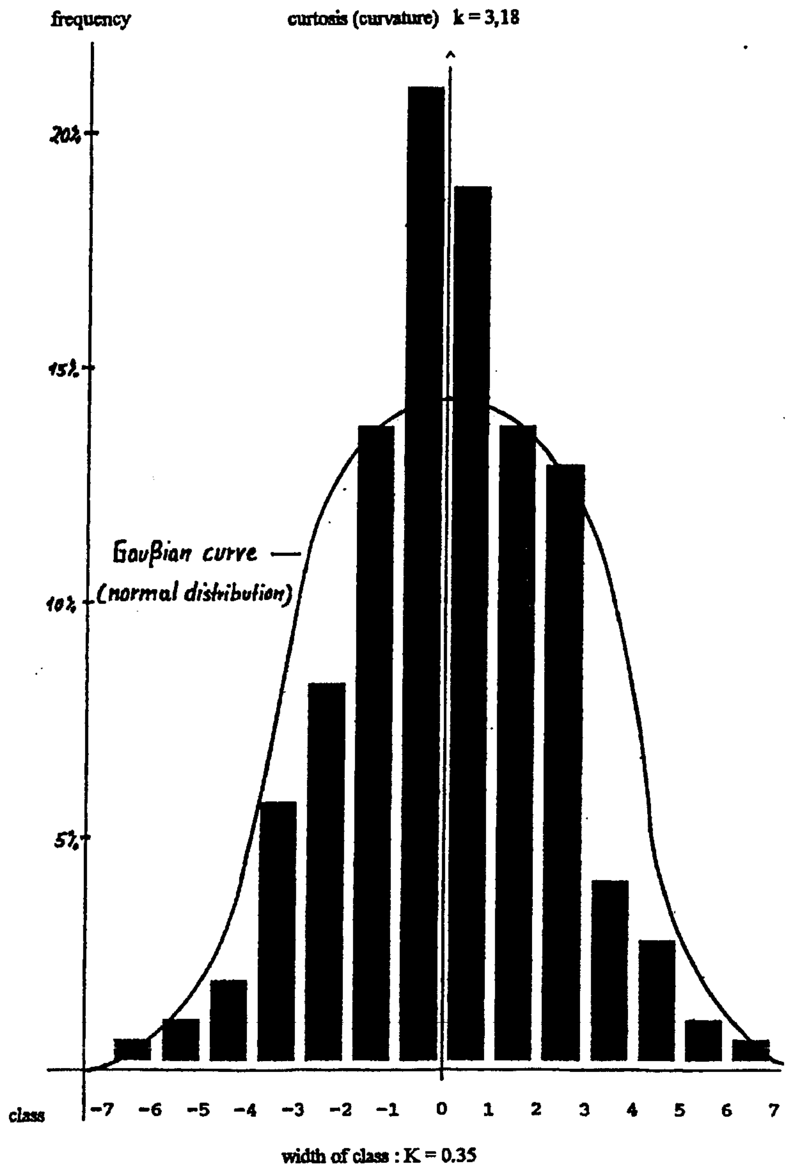


Figure 1: **Histogram of normalised corrections of a precise levelling network in Dresden, measured 1997 with DiNi 10 and NA 3003 (752 observations)**

The instruments made by Zeiss and Leica performed best in these investigations, where the differences between the latest versions of both instrument types are negligible. When using the possibility of setting an extended system accuracy, and an increased integration time, features first offered in software version 4.2 and when using the "Median" measuring mode, the NA3003 delivered the best results under complicated lighting conditions and shadow effects, albeit at the expense of increased measurement time. The DiNi 10 and 11 measures more rapidly under such conditions and is equally very reliable, but ceases measurement before the NA3003 under low light conditions (< 10 Lux at the staff in the measurement region). The DL 101C under investigation required considerably more light than the other instruments, namely 15 Lux at the staff and 10 Lux at the instrument. No systematic measurement errors were detected under unfavourable light conditions. Large differences in brightness between staff and instrument could cause measurement failure, which could be remedied (when measuring from darkness into light) by turning the staff slightly or by illuminating the measurement sector of the staff with a simple torch.

The uniformly good results achieved with all six NA3003 instruments under low lighting conditions are partly contradictory of previous publications concerning the NA 2000 and NA 3000 /2/, /4/. All NA 3003 instruments operated correctly and without losing accuracy in light intensities of 7 Lux at the staff and 4 Lux at the instrument in normal mode, and 6 Lux at the staff and 2,5 Lux at the instrument with extended system accuracy and an increased integration time of 5 seconds. These investigations were carried out in darkened and closed rooms at ranges between 5 and 15 metres without using special light sources with an increased infrared component. It can be concluded that the manufacturer, Leica, has been using qualitatively better CCD arrays sensitive mainly in the visible spectral range in its digital levels since 1995. The comments in /2/ and /4/ in this respect are thus now obsolete, since the detector arrays in the new NA3003 instruments are equally sensitive in the near infrared and the visible range of the spectrum.

One drawback in the instrument illumination system common to all of the tested instruments under low light conditions, such as those encountered inside buildings or dam walls is the lack of illumination of the circular bubble.

As a result of further development the influence of shadow patterns on measurement ability can be almost totally neglected as far as the DiNi's and the NA 3000 instruments are concerned. No erroneous results or drop in accuracy due to shadow patterns were detected, although in a small number of cases measurements were aborted, but the problem was solved by slight rotation of the staff. On the other hand, shadow patterns on the whole staff or on the lower portion of the measurement regions caused major problems for the TOPCON instruments. For example, strong shadow patterns on the staff caused by leafy branches frequently caused measurement failures in both the DL 101C and the DL 102. In some cases, when a measurement was obtained, systematic measurement errors up to 0,30 mm were detected. In the author's opinion this is caused by the processing algorithm used, since the fast Fourier transformation (FFT) is very sensitive to errors in the data points in the initial area. Problems caused by partial occlusion of the measurement area when measuring with TOPCON instruments can be attributed to the same cause. As soon as more than 10% of the staff section required for measurement is occluded by obstacles such as branches the DL10 can no longer measure. If a measurement is possible, systematic errors of up to 0,30 mm are possible, although these are generally detected by a high standard deviation in repeat measurement mode. These remarks apply particularly to occlusion of the lower part of the staff. Although the manufacturer's specifications claim that measurement is possible when 25 – 30% of the measurement region is covered, this was not achieved during the investigations. The DiNi 10/11 exhibited the least sensitivity to staff occlusion. As long as 25 of the required 30 cm of staff graduation in the region of the cross-hair are visible, the Zeiss instruments reliably provide correct results - if not, they do not measure at all. As far as the NA 3003 is concerned, more than 80% of the section necessary for measurement must be clear in order to obtain error free measurements without a drop in accuracy. The instruments can measure even though the occlusion is between 20% and 30%, but in some cases systematic errors of the order of up to 0,5 mm can occur.

However, these errors can be detected since such measurements principally exhibit an increased standard deviation. Operators of all digital levels doing precise levelling should always repeat measurements exhibiting a value for the standard deviation of a single measurement **in excess of 0,20 mm** and determine the cause. The instruments made by Leica will not measure if more than 30% of the required staff segment, which is over 1,5 m at ranges over 40 metres, is not visible.

Advantageous for the DiNi 10 and 11 is the greater sight distance and the smaller staff measurement region. While both DiNi instruments use a 30 cm staff segment /1/ and the TOPCON instruments do not require much more, the angle of view of the levelling optics of the Leica digital levels is 2° This is simply too large at long sight distances, because at a range of 40 metres at least 80% of the required staff segment of 1,40 m – i.e. 112 cm – is needed for a successful measurement. It is therefore necessary to take care to avoid obstacles when using the NA 3003 at ranges of over 25 metres, including obstacles which are not in the field of view of the telescope. This results in a reduction in the speed of levelling when working in very overgrown terrain. The firm of Leica should undoubtedly pay attention to reducing the aperture (the angle of the measurement bean) when developing their new levels, in order to reduce the length of the staff segment required for measurement. Sight distances of 60 metres were achieved to three metre long invar staves when using the NA 3003 and the DL101C, while ranges of up to 115 metres are possible using the DiNi 10/11 under favourable weather conditions. When performing precise levelling with a specified accuracy of $\sigma_h = \pm 0{,}50$ mm/km, however, the maximum sight distance should in general be limited to 45 metres.

The stability of the line of sight is approximately equal in the NA 3003, the DiNi 11 and the newer series of the DiNi 10 and can be rated as very good thanks to the internal temperature compensation system. Less than one minute per degree Kelvin temperature difference is required for acclimatisation. The requirement, mentioned in /4/, for a redetermination of the collimation error after a large temperature change during levelling is no longer necessary for these instruments. The firm TOPCON is lagging behind in this development, as its instruments are not temperature compensated. This means that a longer period of adjustment to the ambient temperature is necessary and that temperature changes during levelling can have a marked effect on the collimation error.

Tests to determine the effect of vibration due to vehicular traffic and due to wind showed similar results overall for all three makes of instrument. Very strong vibrations close to the instrument result in their aborting the measurement, unless a simple measurement mode is being used. In the case of single measurements ($n = 1$) compensator vibration can result in gross measurement errors of 1 mm and more. For this reason no measurements should be carried out with a digital level in simple measurement mode. The influence of vibration on measurements taken in repeat mode ($3 \leq n \leq 6$) can always be detected by means of the larger standard deviation or bandwidth value. The values for the standard deviation of a single measurement displayed were always larger than the systematic falsification of the mean (determined as the deviation from a previously determined correct value). Experienced observers will therefore be able to take repeat measurements and thus eliminate errors caused by vibration related to passing heavy trucks or trams. Vibrations caused by cars during the measurement can normally be ignored, as they do not have a significant effect.

As far as the influence of wind is concerned, however, the instruments under test reacted differently. The DL 101C, which has significant improvements over its predecessor DL 101, has the best compensator of all of the tested instruments. In addition, this instrument is the only one, which is fitted with vibration correction. If this function is activated, the compensator swings are measured and a correction applied to the measurements. Test measurements showed, however, that this vibration correction was only effective if the vibrations were intense, regular, and continuous over a long period of time. As far as other vibrations are concerned, the deactivation of the vibration correction did not have adverse effects on the reliability or measurement accuracy of the instrument. The results obtained in the investigations into the influence of wind from the DiNi 10 were only marginally inferior to those of the DL 101C. The DiNi11, which is also the heaviest, is hardly affected in terms of its measurement ability or accuracy by continuous or gusty winds up to force 6 at the Beaufortscale. The NA 3003, on the other hand, is more sensitive to wind. In particular, gusty wind often caused measurements to be aborted in normal measuring mode. This problem can, at least partially, be avoided by increasing the integration time to 4-6 seconds, albeit at the cost of increased measurement time. Comparative investigations with a DiNi 10 yielded approximately the same results, but the observation time per station of the NA 3003 were twice as long.

Investigations into the influence of staff tilt yielded similar, unsatisfactory results for all three makes of instrument. None of the instruments detected even extreme tilt in all directions. The measurements were so severely systematically affected, as a function of the magnitude of the staff tilt, that totally unusable results were obtained, as already mentioned in /4/. This means that the staff bearers have the particular responsibility of ensuring staff perpendicularity when taking part in precise levelling. In this respect it is pleasing to note that the staff manufacturing firm Nedo has been fitting its invar staves for digital levelling with two bubbles, mounted at different heights. This allows for a simple and effective check on the bubbles' adjustment.

To summarise, the network measurements and the investigation of particular aspects lead to the conclusion that the DiNi 11 and the NA 3003 are practically equivalent and both fully meet the requirements of precise levelling of the highest accuracy. The advantages of the NA 3003 are its good handling characteristics, the possibility to measure under low lighting conditions, the measuring mode „Median" and the possibility of setting a higher system accuracy and increased measurement time. Somewhat of a disadvantage for this instrument are the lack of the PCMCIA card as a storage medium, the impossibility of editing point numbers, the lack of a height section processing facility in the instrument, the measurement start control on the side of the instrument and the relatively large focussing lens travel of 0,3 mm per 10 m sight length difference. The latter problem means that sight length imbalances of more than 5 metres cannot be tolerated in precise levelling where the sight lengths are over 25 metres, in spite of the earth curvature correction and the stable line of sight.

The DiNi has the best thought out and clearest menu structure in the software, an easily understandable display and a clearer data format than the NA 3003. The DiNi 11 is more robust than the NA 3003, in particular against the influence of wind, but is larger and heavier. The battery and memory compartments on the underside of the instrument are not optimally constructed and hinder the changing of these components to some extent. Furthermore, the lack of a carrying handle is a disadvantage when setting up the instrument and when packing it away.

The DL 101C has a very good design, the best data storage philosophy and the best compensator. In view of the problems mentioned above concerning the influence of lighting conditions and shadow patterns, temperature changes and staff occlusion on the measurements and in view of a not yet completely developed software it is still at a disadvantage compared with the instruments of the other two manufacturers. The use of the DL101C for motorised levelling cannot be recommended at the moment. It must be conceded, however, that the sample size of all of the investigations into the TOPCON instruments was much smaller than was the case for the other two manufacturers.

4. MOTORISED LEVELLING WITH DIGITAL LEVELS IN SAXONY

Motorised levelling was developed as early as 1973 by professor Peschel and Dipl-Ing. Seltmann at the Technical University of Dresden. After the introduction of the Ni 002 precise level, which was specially developed for this measurement technology, by the firm VEB Carl Zeiss Jena a short time later, the use of motorised levelling for geodetic levelling spread to many countries all over the world in a relatively short time. This is mainly an achievement of the Swedish Land Survey and in particular of Professor Jean Marie Becker, who introduced this technology to many countries. In the former German Democratic Republic almost all levelling in the national first and second order networks was done by motorised levelling. The equipment included three “Trabant” vehicles, and the Ni 002 level was used exclusively.

After the reunification of Germany the volume of traffic on East German roads quadrupled within two years. The motorised levelling vehicles, which were already obsolete in 1980, were not accepted as equal participants in traffic because of their small size, as the author experienced a number of times as a survey party leader during Autobahn levelling between Dresden and Bautzen in the spring of 1990.

The development of the NA 2000 created the possibility of reducing the high stress levels experienced by the observer and of realising a total data flow in the instrument. Cost pressures also suggested the development of the first motorized levelling using a digital level, when the chief mine surveyor of the firm “Lausitzer Braunkohle AG” (“LAUBAG”, an open cast lignite mining company) approached the author at the end of 1991 and suggested the joint development of this method. The goal was to ensure that the approximately 1100 Km of double levelling performed by the LAUBAG survey parties annually for main and connection levelling could be carried out as efficiently and cost-effectively as possible. Because this levelling was mainly done along heavily used two-lane roads, partially without pavements or shoulders, conventional “on foot” levelling involved the temporary closure of one lane and the use of addi-tional safety personnel /3/. The area to be levelled by the LAUBAG lies mainly in a relatively level part of the Oberlausitz region, which is also favourable for motorized levelling. Motorized levelling using an NA 2000 was jointly developed to the stage of practical application by the LAUBAG and the Dresden University of Applied Science by 1993. The development was generously and unselfishly assisted by Professor Becker and further colleagues of the “Lantmäteriet”, for which our most hearty appreciation is hereby recorded. The result of the development can be seen in Figures 2 and 3. The LAUBAG motorized levelling method now involves the NA 3003 as its measuring instrument and a Volkswagen Taro as the instrument vehicle (Figure 3) as well as two further VW Polo vehicles for the staves.

Figure 2: The LAUBAG motorized levelling party

In 1998 some 650 km of double levelling had been completed with this equipment by mid October, at a rate of 1,4 – 1,5 km of single levelling per hour. More than 80% of this levelling was undertaken on heavily used main roads. Through the use of a new specially made tripod with extended legs and an ungraduated extension of 0,50 m at the lower end of the levelling staves it was possible to achieve an average height of collimation of 2,30 above ground level, which greatly reduced the influence of refraction. The achieved accuracies of the order of 0,5 – 0,6 mm/km double levelling were well under the specified 1,0 mm/Km, so that the observation regime "b-f" (backsight foresight) was generally considered sufficient. However, on main levelling lines, which are also parts of the first order level networks of the states of Saxony or Brandenburg, and on lines where subsidence occurred, the "b-f-f-b" method of observation was used in order to provide an additional check through the station differences and in order to reduce the effects of systematic staff or instrument sinkage during the levelling operations. The proportion of repeat motorized levelling lies under 5% in the experience of the LAUBAG. The survey parties consist of a survey technician as observer and three mining technicians or chainmen as drivers. All party members were trained in the theory and practice of precise levelling during a 60-hour course run by the author.

Figure 3: The instrument vehicle of the LAUBAG motorized levelling party

A further application of motorized levelling was developed by the State Survey Administration of Saxony in 1996. The equipment is shown in figure 4. The levels used were Leica NA 3000 or NA 2000 models. A modified Renault Rapid is used as the instrument vehicle, while two Renault Clio cars are used for the staves. The survey party consists of four survey technicians, as is the case at the LAUBAG. The same electronic trip meter are used to determine the sight distance and instrument stationing as are used by the LAUBAG. They determine sight distances to an accuracy of ± 1m. The average height of collimation above ground is 2,0 metres. As is the case at the LAUBAG, the safety of the personnel is ensured by large warning signs, flashing-lights signals and direction arrows on the vehicles. Measurements of the third order levelling network in Saxony are routinely carried out with this equipment using the "b-f" regime, and the specified accuracy of 1,0 mm/km is attained without problems.

The use of a DiNi 11 instrument in this motorized levelling method of the State Survey Administration of Saxony was successfully tested in the first and third order state levelling networks during the course of an undergraduate thesis supervised by the author at the Dresden University of Applied Sciences in spring 1997. For this purpose a 26 km long line of first order levelling consisting of 28 sections and running along a heavily travelled main road was relevelled. Double levelling was used, the observation method being "b-f-f-b" at average sight distances of 35 m and a maximum sight distance of 40 m. Repeat measurement mode (n>2) was used for the observations and the maximum permissible station difference was 0,40 mm.

Figure 4: The instrument vehicle of the motorized levelling of the State Survey Administration of Saxony.

The resulting accuracy was found to be 0,37 mm/km of double levelling, which was within the specification of 0,40 mm/km. The line closure error of 1,5 mm was well under the specified 8,1 mm for this line, so the test can be regarded as very successful. The repeat levelling rate was 7% (2 of 28 sections). These occurred in a section where the road pavement consisted of concrete slabs and where extreme hindrance due to traffic was experienced. The speed of levelling on this first order line was 0,85 km of single levelling per hour or 5,4 km per day. The investigator, Wolfram Riech, estimated that this rate could be increased by 25% in routine use. Further motorized levelling measurements were carried out using the DiNi 11 on a 13,5 km long line of the third order network using the "b-f" regime and double levelling. In this case no repeat levelling was necessary. An accuracy, calculated from the double measurement differences, of s_h = 0,51 mm/km was achieved. The rate of levelling was 1,8 km of single levelling per hour or 9,0 km per day.

It was thus possible to conclude that the DiNi 11 proved to be equally as good as the NA 3003 in motorised levelling. At the same time the result achieved by both the LAUBAG and the by the State Survey Administration of Saxony show that motorized levelling with digital levels in Eastern Germany delivers comparable results to those achieved using the Ni 002, at considerably lower observer stress levels. In addition it can be mentioned that the State Survey Administration of Mecklenburg-Vorpommern is using a further partly motorized survey party. The party of three is equipped with two Polo Fox vehicles for the staves, but the NA 3003 instrument is carried on foot.

5. REFERENCES

/1/ Feist, W. u.a. (1995): Die neuen Digitalnivelliere DiNi 10 und DiNi 20, Vermessungswesen und Raumordnung 57, April 1995, Bonn, pp 65 -78

/2/ Ingensand, H. (1993): WILD NA 2002 NA 3000, Technischer Bericht Digitalnivelliere, Leica AG, Heerbrugg, Switzerland, 1993

/3/ König, D.; Ruhland, A. (1996): Einsatz des Motorisierten Präzisionsnivellements mit Digitalnivellier zum Nachweis von Bodenbewegungen im Lausitzer Braunkohlenrevier; Schriftenreihe 24/1996 DVW e.V., Wittwer Verlag Stuttgart, 1996, pp 12 - 22

/4/ Wehmann, W. u.a. (1994): Erkenntnisse zum Einsatz des NA 3000 für Präzisionsnivellements, ZfV 119, Juli 1994, Stuttgart, pp 346 - 359

The production line used in the third precise levelling of Sweden

Mikael Lilje, Per-Ola Eriksson

Abstract

The National Land Survey of Sweden (NLS) is deeply involved in the Third Precise Levelling of Sweden since the 1970s. The whole project represents more than 50 000 km double levelling (50 000 benchmarks) and the production work is supposed to be completed about year 2000.

This paper describes the unique production line that is used in the third precise levelling network. Most of the production line is digital and has been so since the start. We have been using the same production line almost throughout the project which mean that we have been trying to treat a specific type of data the same way during the 20 years that the project has been going on. Included in the production line is, besides the actual levelling, also the production of site descriptions, maps as well as storage of data in a suitable archive.

The production line can roughly be divided into five different phases that are synchronised in time. These are

1. *Planning the network*
2. *Establishment of benchmarks*
3. *Storing information about the benchmarks into the archive as well as preparations for the levelling*
4. *Levelling*
5. *Computation, archiving and delivery of results.*

The whole process takes for one region about four years of work to complete.

INTRODUCTION

Before the actual production work of the third precise levelling network started in 1979, major discussions took part concerning how the production work should be done the best way. It was decided very early that if the network should be of the highest quality, the production work should be done in a homogenous way and that means in the same way throughout the whole project. Therefore, it was necessary to have a production line that was as correct as possible from the beginning. It was also decided that the production work should be done in a digital production line. The manual work should be held to a minimum. We are proud to say that it was almost possible for us to create this production line from the beginning and that we are using the same methods today as we did in the beginning of the project. The 20 year old production line is still working excellent.

The work with the third precise levelling can be described in five different phases that must be synchronised in time. This is graphically described in figure 1 below and in the text in the paper.

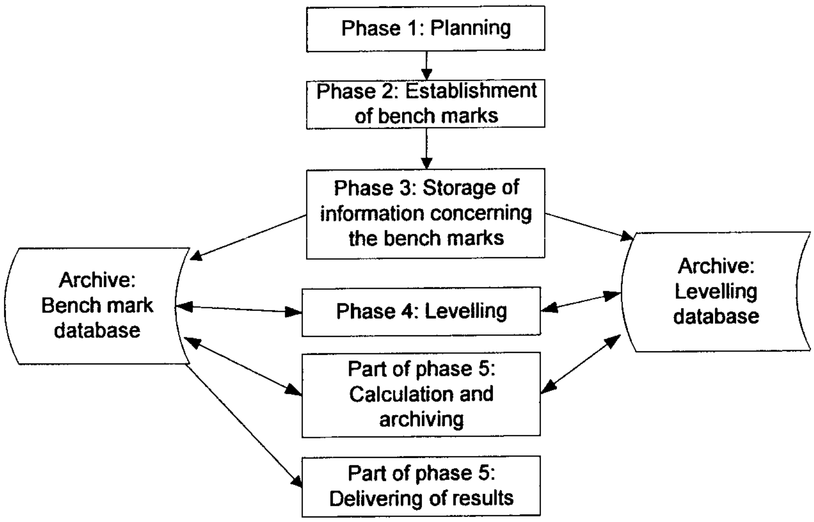


Figure 1. Phases in the third precise levelling project.

The benchmark database contains information about the benchmarks. The information consists of benchmark number, type of monumentation, description of how to find the benchmark etc. The levelling database consists of information about the levelling. All data in this database are stored section by section. The benchmark and the levelling databases are the two major databases in the project but there exists several other databases as well. All the databases have been on a PR1ME computer since the beginning. We are at the moment changing server for the databases and will be using an Oracle database to store all the information. Beside the storage of data in the two databases, all important files in the project are stored.

As mentioned earlier, it is important to do the right thing at the right time. Figure 2 shows the order of the five different phases as well as when these different phases takes place. We can see that the work in a certain region starts during the autumn in year 1 with the planning of the network. The planning stage is done during the winter and must be finished in time for the field work during the spring in year 2. The establishment of benchmarks is done during the summer and all the data from this work are stored in our databases during the winter. We are at this stage also preparing the levelling work that takes place during year 3. The field work is followed by the computations and storage of data during the next winter. The whole process covers almost three years of work spread over four years.

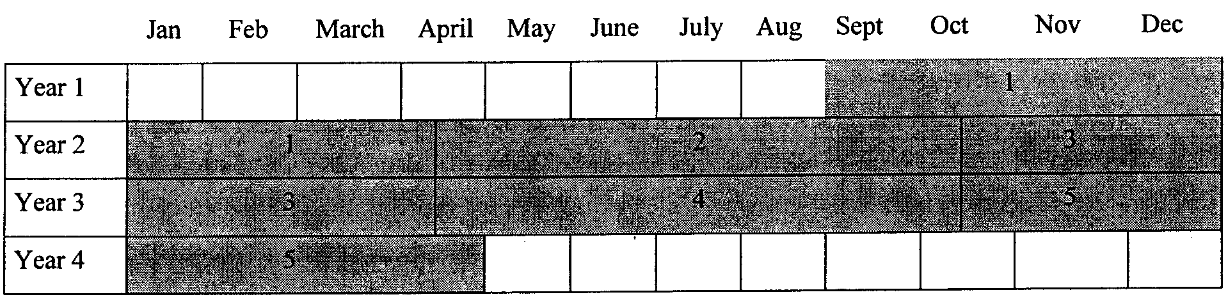


Figure 2: Time schedule for the process

1: Planning of the network. Planning for the location of the lines in collaboration with the local users.

2: Field work. Establishing the benchmarks. Discussions on detail level with the local users.

3: Storing benchmark descriptions into databases. Preparations for levelling.

4: Field work. Levelling.

5: Calculation, storing data and delivery of results.

PHASE 1: PLANNING OF THE NETWORK

The first phase in the creation of a new height network is a very important one. There is no point in creating a new network if the end-users are not satisfied with the network from the beginning. They who are the users of the network and they need to have good, stable benchmarks with good quality both in the monumentation and the heights. The benchmarks should also be easily reached from roads. These demands can not be taken care of if the planning phase is not properly done and involving the end-users in the specific area.

In order to involve the major end-users we send a map with the planned lines to all the local users, community authorities, road and railroad authorities and others who can possibly be a user of the points in the region. They are all invited to give their opinion of the plan. The views of the users are collected and we can make a final plan. In the plan, we try to combine all the different demands. This is not always easy and we may not be able to fulfil all the different demands on the location of the levelling lines. We also have certain demands of our own on the network configuration in order to create a strong and homogeneous network all over the country. We always have to find the best common solution.

We also collect benchmark maps and descriptions covering the region from the local users that we are in contact with. This is important since we do not want to establish a new benchmark at the same location as a local one. We do however have high demands on the benchmarks, which means that many local points can never be used for the third precise levelling of Sweden. For more details concerning our demands on benchmarks, see Eriksson (1999).

Sometimes a local user is interested in expanding the original plans of the precise levelling in the region to cover his own needs better. We try to meet these demands as much as possible. Of course, the local users will have to pay for the extra levelling work since it is not part of the third precise levelling but more as an extension to suit the local users needs in first place. However, we try to keep the price as low as possible, as we think that these extra works are important. It is important that the local users can connect their networks to the new national height network.

PHASE 2: ESTABLISHMENT OF THE BENCHMARKS

One of the main purposes by establishing a new height network is to give good and solid benchmarks for the users with heights that they can trust. Establishing a new benchmark is therefore an important job and must be done with the greatest skilfulness. The location of the benchmark should be chosen so that the point is well protected and easily reached at the same time. The point should be established on stable ground, preferably on bedrock. To choose and to establish a benchmark is therefore a job that requires personal with experience. Teams of two specially trained persons from the NLS do this work. We are using one to three teams each year depending on the needs of establishing benchmarks and what type of ground that we are working on. One team normally establishes 10 benchmarks a day. However, if there is no bedrock or solid rocks but only soil the work goes much slower. In this case, about three benchmarks are established every day.

This phase uses the plans originated in the first phase. These plans may though be affected by minor amendment to reach the best quality in the net and in the benchmarks. These amendments are made directly out in the field.

When the benchmark is established, a site description is created. The information on the description is e.g. point number (unique one for each point), type of benchmark and how to find it (both verbally and with a drawn sketch). All the information is collected in analogue form in the field and transformed into digital form in the office after the field work.

During the establishment, the location is also marked on a map with the scale 1:50 000 (or 1:100 000). The point is digitised directly using a MapInfo application developed at the National Land Survey. The point is accurate in the horizontal to approx. 50 metres and the digitalisation is merely a cartographic one. The purpose of the digitalisation is twofold; we need to have approximate horizontal co-ordinate of the point for the computation and we need to represent the point on a map. When we have all information in digital form, we use MapInfo to handle the digital geographic data.

PHASE 3: STORING SITE DESCRIPTIONS INTO DATABASES. PREPARATIONS FOR LEVELLING

It is essential to save all raw data throughout the whole project. Therefore a systematically storage is important. It is vital to have the opportunity during and after the project to go back to the actual readings and study these. This means however that we are forced to store a lot of files. Simple calculation concerning the number of raw data files from the levelling work gives at hand that each levelling team produces seven different files each day. For one levelling season covering 100 working days and three levelling teams, this means 2100 files from the levelling field work alone. However, the levelling field work is just one part of the production work. Just as important is all the data describing the point.

It is also important to store the information in an organised way, e.g. in databases. We are using two main databases within the project. These are the benchmark and the levelling databases. Besides these two databases, a number of other databases are used. More information about some of them can be found under phase 4.

The benchmark database is composed by information concerning the benchmark as approx. co-ordinates, benchmark number, type of benchmark etc. The information comes from the field work in phase 2. Most of these information can later be found on a site description. Everything but the sketch is in digital form. We have been using a PR1ME computer with the PRIMOS operative language since the beginning of the project. We are now forced to change computer system. Everyone who has been forced to change computer system in the middle of a major project knows about the amount of work that is involved. The new geodetic archive at the National Land Survey will be based on Oracle under Windows NT. Since the new archive is not complete yet, we are forced to use Access as a temporary solution for the levelling project.

The information in the levelling database is built up section by section. The information in the database is e.g. measured height difference, length of section, number of set-ups, observer, instrument nr, rod nr, type of road, type of weather etc. In total, 54 different items is stored for each section.

An important task in this phase that must not be forgotten is the preparation for the levelling field work that will take place in the next phase. This means that benchmark maps and site descriptions are copied, the equipment as instrument and cars are checked. This is done during the winter/spring before the levelling season starts and by personal at the National Land Survey.

PHASE 4: LEVELLING

This phase takes place during the third year. The data from the field is checked, processed and compiled for computation. For statistics from the production work, see figure 3.

Year	No of team	No of working days	Prod. total	Relev. km	Relev. %	Prod. netto	Netto km/day
		Total	Km	Km Total	Ave/team	Km	Ave/team
-78	2	284	3016	121	4,0	2895	10,2
-79	3	370	3164	394	12,4	2770	7,5
-80	4	487	4378	226	5,2	4152	8,5
-81	5	552	5491	240	4,4	5251	9,6
-82	5	566	6646	255	3,9	6389	11,3
-83	6	557	6896	295	4,8	6599	11,5
-84	5	397	4636	209	5,8	4429	10,5
-85	5	503	5370	347	6,1	5021	10,0
-86	5	541	6099	418	7,2	5680	10,4
-87	5	453	5326	301	5,9	5026	10,9
-88	4	474	5969	529	8,7	5440	11,5
-89	4	419	5063	451	8,9	4612	11,0
-90	4	462	5505	585	10,7	4919	10,6
-91	3	344	4249	278	6,5	3971	11,5
-92	3	309	3989	257	6,4	3732	12,1
-93	3	333	4180	258	6,2	3938	11,9
-94	3	304	4077	447	11,0	3630	11,9
-95	3	288	3964	425	10,7	3539	12,3
-96	3	306	3815	445	11,7	3369	11,0
-97	3	257	3470	291	8,4	3178	12,4
-98	3	254	3376	285	8,4	3091	12,2
SUM:		8460	98679	.7057		91631	

Figure 3: Statistics from the field work

As a summary the daily production varies between 4.5 and 6.5 km double run levelling for a team with an average relevelling frequency of 5 to 8 % for 5.5 hours measuring. One team consists of four persons. The field season production has varied between 1 600 and 3 300 km and up till now almost 46 000 km double run have been levelled, excluding the relevellings.

The levellings started in the south and have now reached the most northern parts, see figure 4. Due to weather conditions, the levelling season becomes shorter and shorter and it started last year in the beginning of June and ended in the beginning of October.

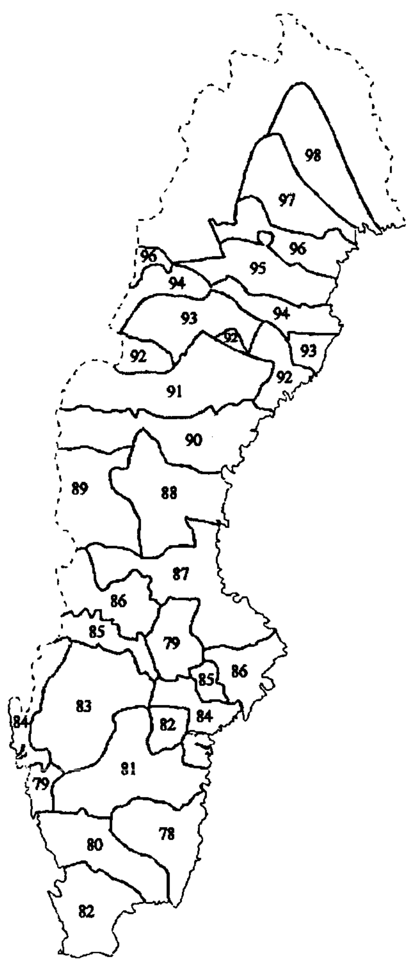


Figure 4. The progress of the third precise levelling of Sweden.

The average time per set up is less than 2 minutes including the moving of the cars. The maximum allowed sight lengths are 50 meters. All operations, except the connection to benchmarks, are done from the vehicles. For this reason working methods have been designed to minimise observing time and optimise the quality of the results.

The instrument used is NI002 from Carl Zeiss Jena. Due to the 360 degree rotational eyepiece, one can say that the instrument is the heart of the motorised levelling technique. This means that the observer can stand at one spot and shoot around the horizon. The system with the turnable pendulum that gives a quasi-absolute horizon is another big advantage of this instrument. The readings are made on the two scales of the rod, one scale in each pendulum site.

The sight lengths forward and backward are kept equal within 5 to 10% for each set-up. This is done with the help of Digitrips mounted in all the cars.

The observer tells the staff readings to the driver, who enters the readings into a small handheld computer (Micronic). The communication between the driver and the observer is done with a headset and a speaker in the car, because of the traffic noise. The datalogger calculates the difference between the two scales of the rod. If one of the fault limits is exceeded, the driver is told that a new measurement must be done. When the set-up is OK the driver is allowed to proceed. In this way, it is very hard to get a gross error into the measurements.

The same type of datalogger is used in the rod cars. The rod car drivers are storing information about point numbers and time for levelling. Type of weather, type of road surface and temperature of the invar band at an upper and a lower point on the band is stored for each set-up. This way of work leaves minimum of work for the observer and optimised amount of work for the whole team.

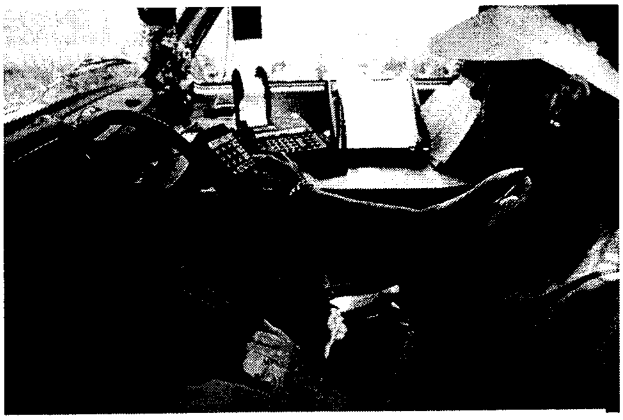


Figure 5. Entering reading to the Micronic.

The moving of the vehicles must always be done in the same chronological order and observations must be carried out in a predetermined and systematic order.

As mentioned earlier, each team produces three files each day. These files are transferred to a portable computer after each working day. The three files, called D-files, are checked and if necessary corrected. This work is done in the evening the same day as the levelling is done. Errors can be wrong point number, wrong time of measurement, wrong information of type of road or weather. After correction the result will be three new files called D/R-files and a T-file including information from all three D/R-files, see figure 6.

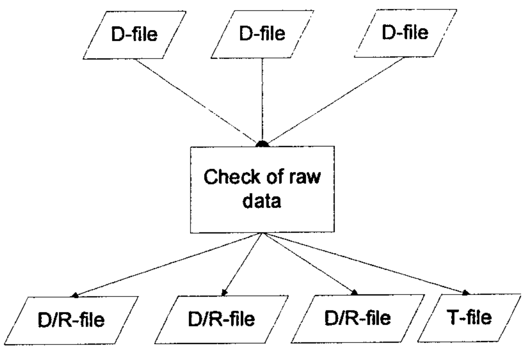


Figure 6: Check of raw data after levelling

All the files are sent home to NLS, where they are stored in a file database, see Appendix A. In the portable PC in the field there is a local copy of the levelling database, where all the measurements are stored during the field season. In this database corrections for earth curvature and temperature can be done. From that local database the measurements can be used to do some control calculations e.g. check of loop misclosures or other faults. That is a way to ensure that no gross errors are made during the field season. Rod corrections cannot be done here, since the rods are calibrated before and after the field season, and the corrections are interpolated from those calibrations. These corrections are done after the autumn calibration as well as the earth tide correction, see figure 7. Then the T-file and all the corrections are stored in the real levelling database.

To be certain to have a good quality of the levelling, all the equipment is checked with a regular basis. For instance, the instruments are checked once every week using a special designed check procedure. All the information from these checks are then stored in a special designed database for this purpose, the instrument database.

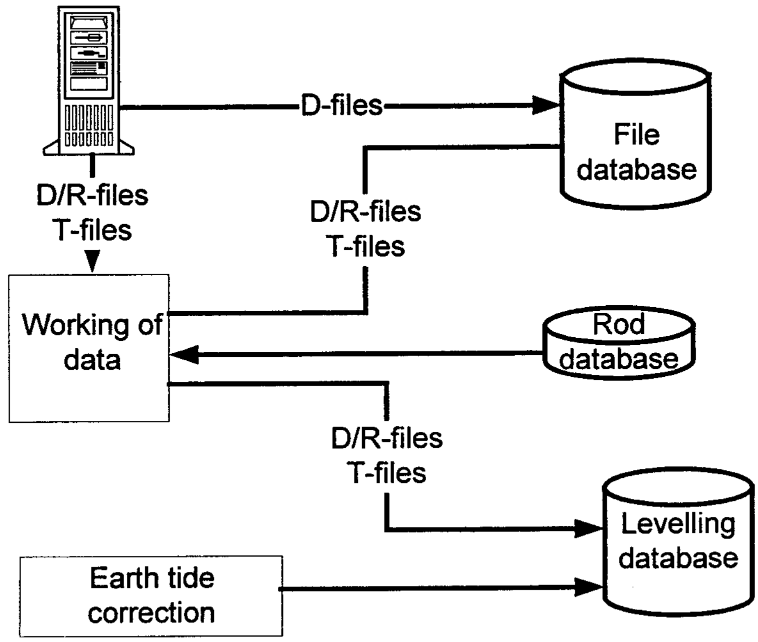


Figure 7. Correction of raw data after the season.

PHASE 5: CALCULATION, STORING AND DELIVERY OF DATA

This phase covers the work to be done during the autumn and winter after the levelling season. The computations are done in delimited regions defined by the loops from the second precise levelling. Since we cannot wait for the whole network to be completed before we deliver heights we will have to perform the calculations in the height system based on the second precise levelling, called height system RH70. First the measurements in a region are picked out from the levelling database and then a free adjustment is done. We are using a program developed by our selves for adjustment. Then the heights of the common benchmarks from the second and the third levelling are examined. Then it shows what points from the second precise levelling are of good quality. A number of the old points must be recalculated in each area, depending on e.g. poor benchmarks or errors in the measurements. After that the final calculation is done with the choosen points from the second precise levelling kept fixed. Since the calculation is done with points in height system RH70 fixed the new heights are also in system RH70. To show that these heights comes from the third precise levelling and that they should not be mixed with other heights in system RH70, we call the heights from this measurements RHB70.

Together with the data already stored in the databases the heights are stored in the benchmark database. There is also a register containing information about the calculation. All the heights are connected to an adjustment number. In that way we can always see from what computation the height is calculated. A scheme of the production line for the adjustment can be found in figure 8.

When the whole net is completed in a couple of years, a new adjustment will be performed including the entire network. On that basis, a new national height system will be established. Similar projects are going on in the other Nordic countries. Denmark is working with the implementation of their new height network DNN KMS1990 and Finland have reached as far as Sweden in their third precise levelling. The plans for Norway are unfortunately not quite clear now. The plan is that there will be a common adjustment for the whole Nordic block when we all have finished our levelling projects. To help with the preparation of this work, the Nordic Commission of Geodesy (NKG) has a working group working with height determination questions.

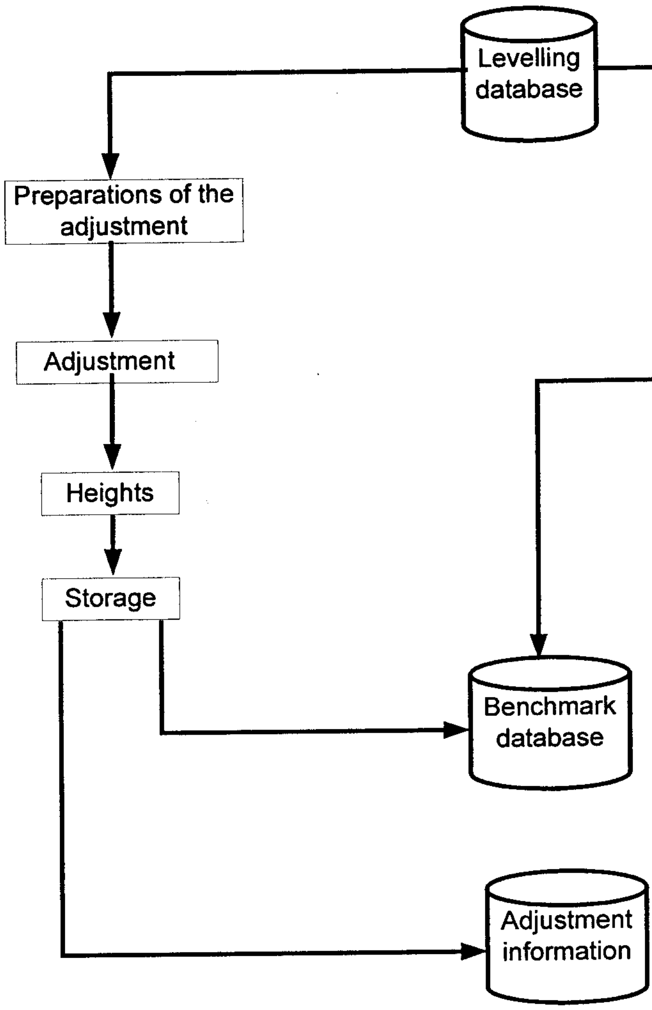


Figure 8: Production line for the adjustment

MAINTENANCE OF THE NEW HEIGHT NETWORK

The maintenance work can be treated as phase 6. The oldest parts of the network will be around 25 years when the new network is finished and the important implementation work starts. Investigations have shown that ½-1 % of the benchmarks are destroyed each year with the lower figure in woodlands. If we want to have a network with benchmarks, maintenance work needs to be done.

The work is done very much like the actual levelling project. The main differences are that the first phase concerns an investigation on how many of the original benchmarks are still useful and that new ones replace destroyed ones. Site descriptions are updated. We are working with digital sketches on the site descriptions within the maintenance work. The old sketches are scanned and minor changes are done using different types of drawing programs. When there are major changes, new sketches are drawn and scanned. The maps are updated as well.

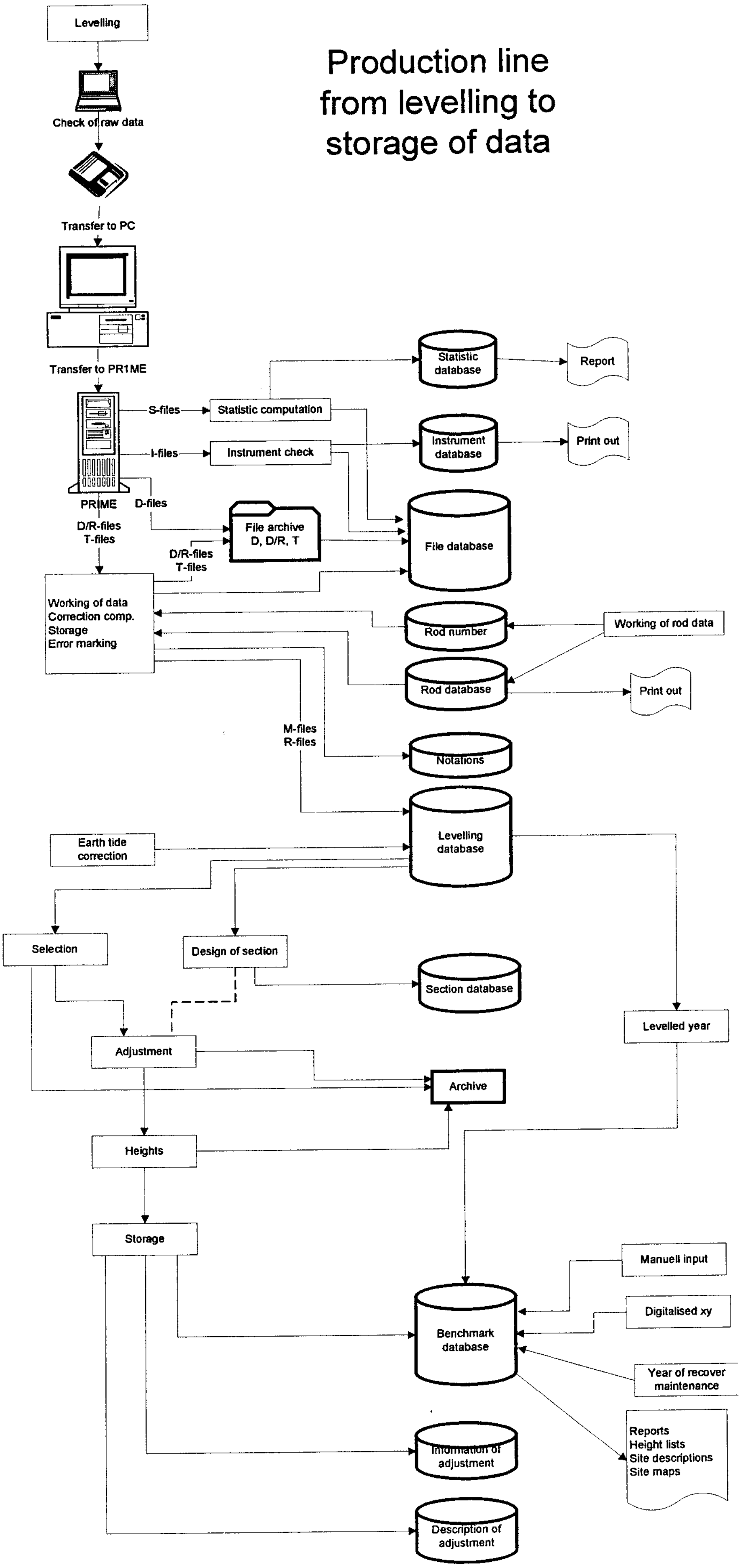
The levelling work is done with the same technique and the same equipment as the ordinary work within the third precise levelling. We have developed a special levelling methodology to make sure that the quality of new levelling is the same as the original one.

REFERENCES

Becker J-M, Lilje M, Eriksson P-O (1998): Establishment of third precise levelling network, Presented at *FIG XXI International Congress*, July 19-25, Brighton, United Kingdom, 1998.

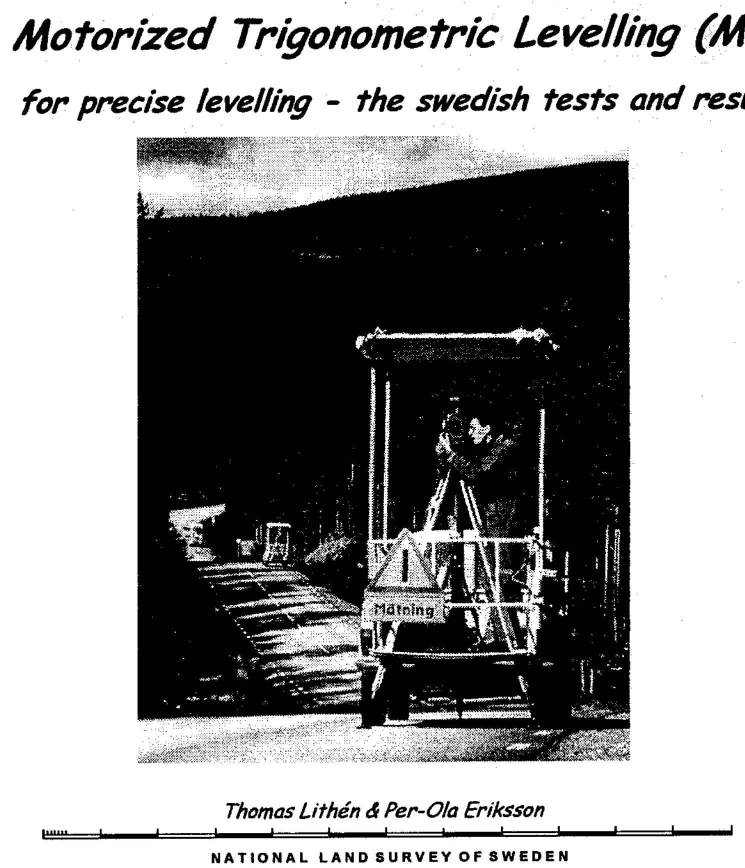
Eriksson P-O (1999): Requirements on the height benchmarks in the third precise levelling of Sweden, Presented at *Geodesy and Surveying in the Future, The Importance of Heights*, March 15-17, Gävle, Sweden, 1999.

Production line from levelling to storage of data



Motorized Trigonometric Levelling (MTL) for precise levelling – the Swedish tests and results

Thomas Lithén, Per-Ola Eriksson



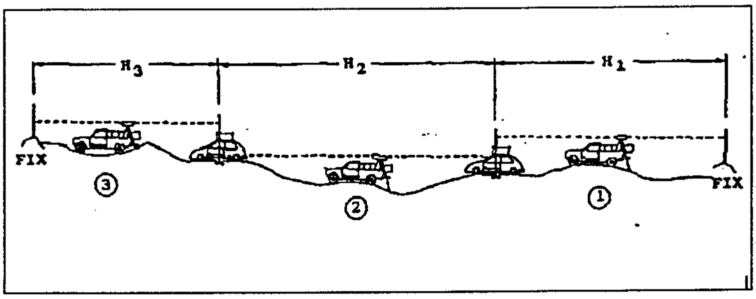
Abstract

The third national precise levelling programme of Sweden – called "riksavvägningen" (RA) – has been in progress since 1979 and is estimated to be completed in 2003. The National Landsurvey of Sweden, which is responsible for this project, has developed and used Motorized Levelling (ML) as the main technique to solve this task. Parallel to the main project, experiments and test has been done in order to improve the levelling techniques and make them more efficient, furthermore the tests has been used to find answers to different problems in the levelling program which have arisen during the project.

This presentation describes experiments using trigonometric levelling, and specially MTL (Motorized Trigonometric Levelling) during the period 1985-1990. The main goal with these tests were to find out if it was possible to increase the production of levelling. Futhermore, the National Land Survey was interested to have an alternative technique to check existing levellings or to use in areas where levelling isn't suitable.

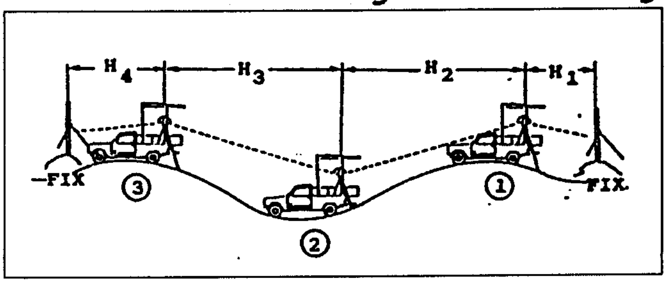
Precise levelling - according to the third national levelling programme

Motorized Levelling (ML)



* accuracy < 1 mm/SQR(km)
* production around 2 km/h

Motorized Trigonometric Levelling (MTL)



* accuracy ? mm/SQR(km)
* production ? km/h

NATIONAL LAND SURVEY OF SWEDEN

The development phasis of MTL

1 - Preliminary study
  * Gävle 1985

2 - Testfield study
  * Gävle 1985

3 - First productiontest
  * Falun 1985

4 - Full scale productiontest
  * Sälen 1986

5 - Production
  * Jönköping 1987
  * Sveg 1988 (FTL)
  * Tjallingen 1990 (FTL)

NATIONAL LAND SURVEY OF SWEDEN

Preliminary study

Precise levelling using trigonometric levelling possible under the following conditions:

* sight distances between 100 to 300 meters
* zenith angles between 95 -105 grads
* accurate connections to benchmarks
* regular checking and calibration of equipment
* minimizing the influence of refraction

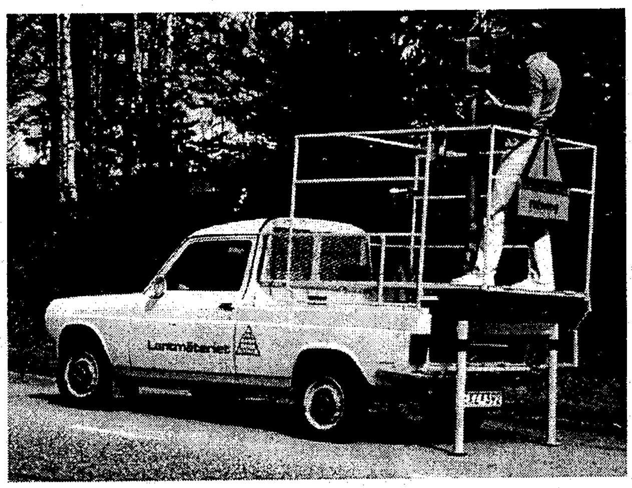
Production results better than 3 km/h possible with:

* an average sight distance of 250 meters
* a setup time of 4,5 minutes

NATIONAL LAND SURVEY OF SWEDEN

Testfield study

* Test of surveying procedures
* Test of totalstations
* Test of data storage and data processing
* Test of leveling staffs
* Test of tripods



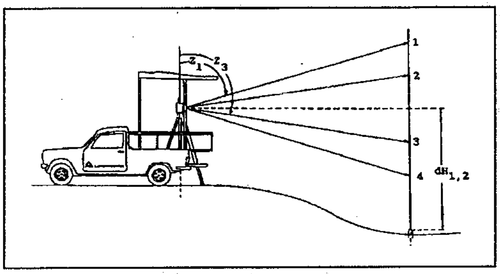
Conclusion: Precise levelling is possible using MTL

NATIONAL LAND SURVEY OF SWEDEN

Full scale productiontest - surveying procedures

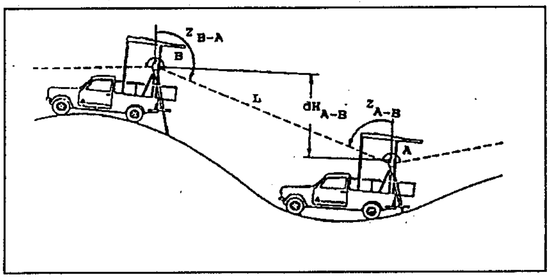
All sections are measured by double-run height traversing

Bench mark tie



* Four targets are measured in order to calculate two separate elevation differences

Observations between vehicles



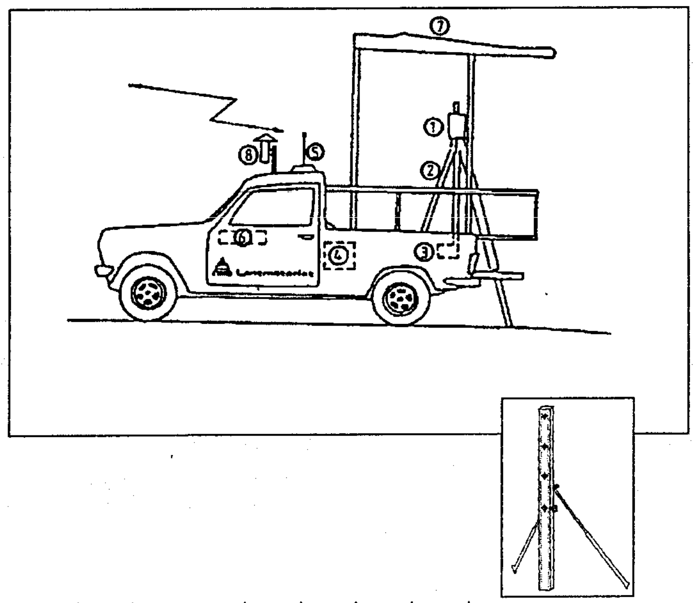
* Two reciprocal slope distances
* Three sets of simoultaneous reciprocal zenith angles

NATIONAL LAND SURVEY OF SWEDEN

Full scale productiontest - the equipment

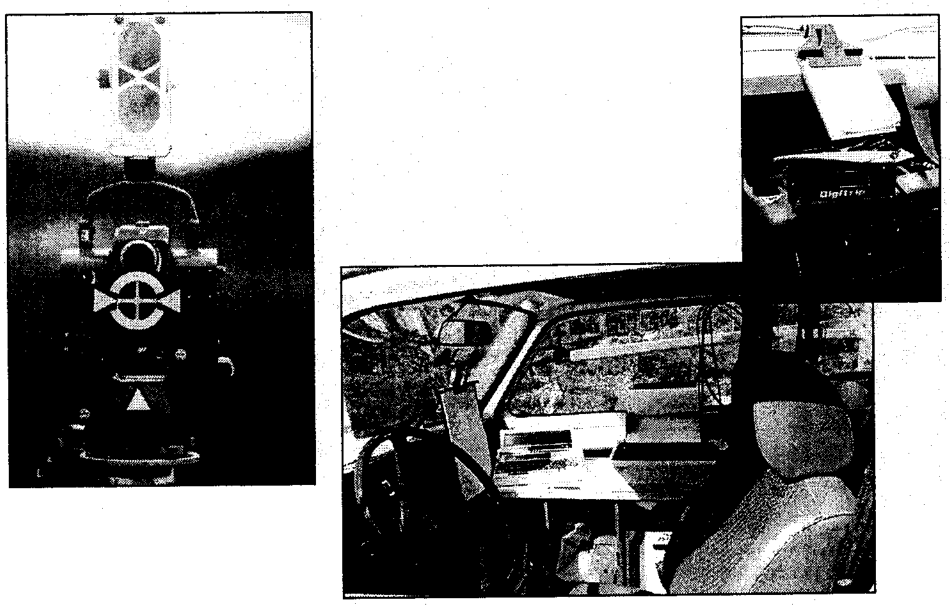
1 Total station
2 Tripod
3 Electric hoist
4 Telemetry equipment
5 Aerial for telemetry
6 Field computer
7 Folding rain/sun canopy
8 Thermometer

A levelling staff



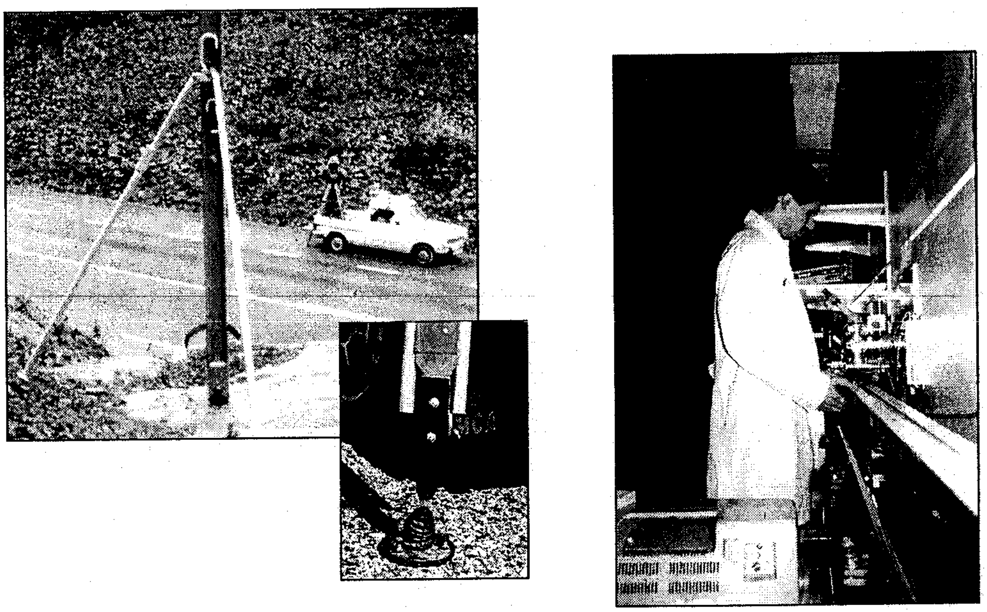
NATIONAL LAND SURVEY OF SWEDEN

Some important details



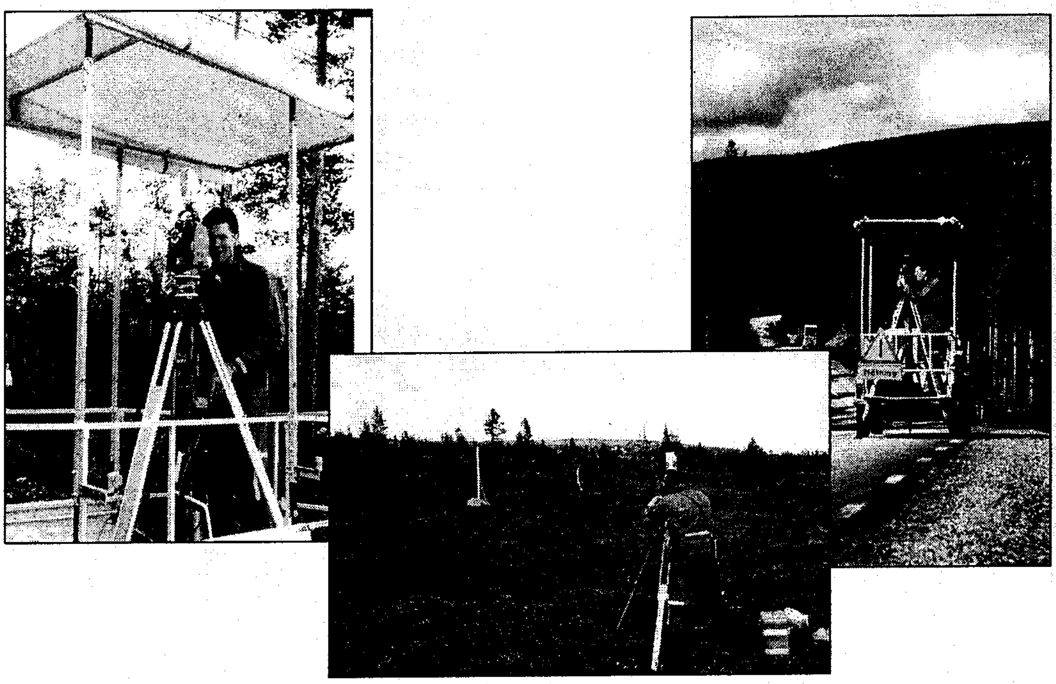
NATIONAL LAND SURVEY OF SWEDEN

Some more important details



NATIONAL LAND SURVEY OF SWEDEN

Full scale productiontest - in reality



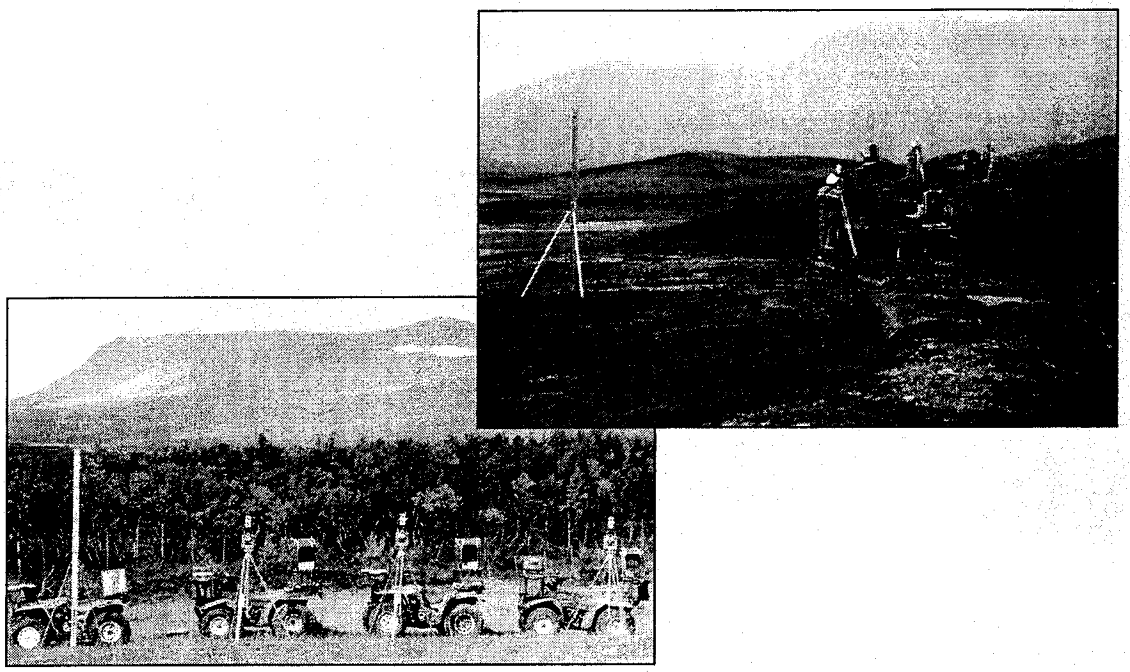
NATIONAL LAND SURVEY OF SWEDEN

Production results

Area	Year	Method	Dist (km)	No of sections	Time (H)	No of setups	Setups/ km	Mean sight distance (m)	Time/ setup (min)	Production (km/h)
Falun	1985	ML	42,2	48	26,0	640	15,2	33,0	2,44	1,62
	1985	MTL	36,6	40	22,9	174	4,8	284/20	7,90	1,60
Sälen	1986	ML	441,4	450	210,0	6004	13,6	36,8	2,10	2,10
	1986	MTL	479,8	478	252,8	2974	6,2	194/20	5,10	1,90
Jönköping	1981	ML	171,1	174	106,4	2548	14,9	33,6	2,51	1,61
	1987	MTL	190,2	190	116,4	1391	7,3	156/30	5,02	1,63
	1958	FL	124,0	120	186,0	1320	10,6	47,0	8,45	0,67
Total		ML	654,7	672	342,4	9192	14,0	35,6	2,23	1,91
		MTL	706,6	708	392,1	4539	6,4	186/23	5,18	1,80

NATIONAL LAND SURVEY OF SWEDEN

FTL



NATIONAL LAND SURVEY OF SWEDEN

Accuracy

Area	Year	Method	Rejection limit X*SQR(km)	No of section	No of relev.	No of > rej. %	Standard error of mean mm/SQR(km)
Falun	1985	ML	2	21	3	19	0,95
	1985	MTL	2	19	1	15,8	0,70
Sälen	1986	ML	2	225	27	15,1	0,71
	1986	MTL	2	233	37	21	0,85
Jönköping	1981	ML	2	90	7	8,9	0,66
	1987	MTL	2,8	88	5	6,8	0,76
Total		ML	2	336	39	13,7	0,71
		MTL	2,8	340	43	9,1	0,82
			2			20,1	
Sveg	1988	FTL	2,8	15	0	6,7	0,88
Tjallingen	1990	FTL	2,8	28	4	18,8	1,05

NATIONAL LAND SURVEY OF SWEDEN

Conclusions

The use of MTL and double run levelling meet the requirements for precise levelling, i e better than 1 mm/SQR(km)

MTL and ML both produces the same heights, which is not always the case with ML compared to older surveys in Sweden using FL (Wild N3)

The production of MTL is about the same as for ML under swedish conditions , ie around 2km/hour

The cost of production is, however, higher for MTL compared to ML due to the higher cost of investment and the need for more skilled personnel

We believe that MTL used in hilly terrain and open landscapes, specially for 2nd and 3rd order surveys, will compete with ML even in economic terms

NATIONAL LAND SURVEY OF SWEDEN

-

References

Becker, J-M Lithén, T	(1986)	Nivelliment indirect motorise (MTL) & technique motorisee XYZ (MXYZ) en Suede. FIG Commision 5, Toronto 1986, (p 52-68), The Canadien surveyor 1986:4, (p 461-475) & LMV-rapport 1986:7.
Becker, J-M Lithén, T	(1986)	Motorized Trigonometric Levelling (MTL) & motorized XYZ tecnique (MXYZ) in Sweden. LMV-rapport 1986:8.
Becker, J-M Lithén, T Nordquist, A	(1988)	Experience of Motorized Trigonometric Levelling (MTL) – a comparison with other techniques. LMV-rapport 1988:23 & Modern techniques in Geodesy and Surveying. NKG 1989, (p 81-108)
Becker, J-M Eriksson, P-O Lithén, T	(1990)	Experiences of Trigonometric Levelling in Sweden. The eleventh meeting of NKG, 1990

Nordquist, A

Chrzanowski,A Greening, T Kornach, W Secord, T Vamosi, S	(1985)	Applications and Limitations of Precise Trigonometric Height Traversing. NAVD-symposium 1985, Washington
Kasser, M	(1985)	Recent advances in high precision Trigonometric Motorized Levelling (NIPREMO) in IGN-France. NAVD-symposium 1985, Washington
Lithén, T	(1985)	Angående försök med motoriserad trigonometrisk höjdmätning (MTL)och motoriserad kombinationsmätning (MXYZ). (in swedish) LMV internrapport, dnr 402-964-85
Rüeger, J-M Brunner, F	(1982)	EDM-Height traversing versus Geodetic levelling. The Canadien surveyor 1986:4, (p 69-88)
Whalen, C	(1985)	Trigonometric Motorized levelling at the National Geodetic Survey. NAVD-symposium 1985, Washington

The use of motorised trigonometric levelling (MTL) in Denmark

Klaus E. Schmidt, Casper Jepsen

National Survey and Cadastre in Denmark

Abstract

The MTL-technique has been used for the last 10 years to achieve height determination of low precision. The equipment applied and the measuring procedures are described. Some practical field experience is reported.

1. Development of the Danish MTL system.

Starting in 1987 this was done in cooperation with the Swedish National Land Survey. A detailed description of the method and application is given in Becker, J.M et. al.: Experience of Motorized Trigonometric Levelling (MTL) - a comparison with other techniques. LMV-rapport, National Land Survey, Gävle 1998. The most important difference between the Danish and the Swedish system is that we in Denmark only use 2 cars instead of 3 cars in Sweden.

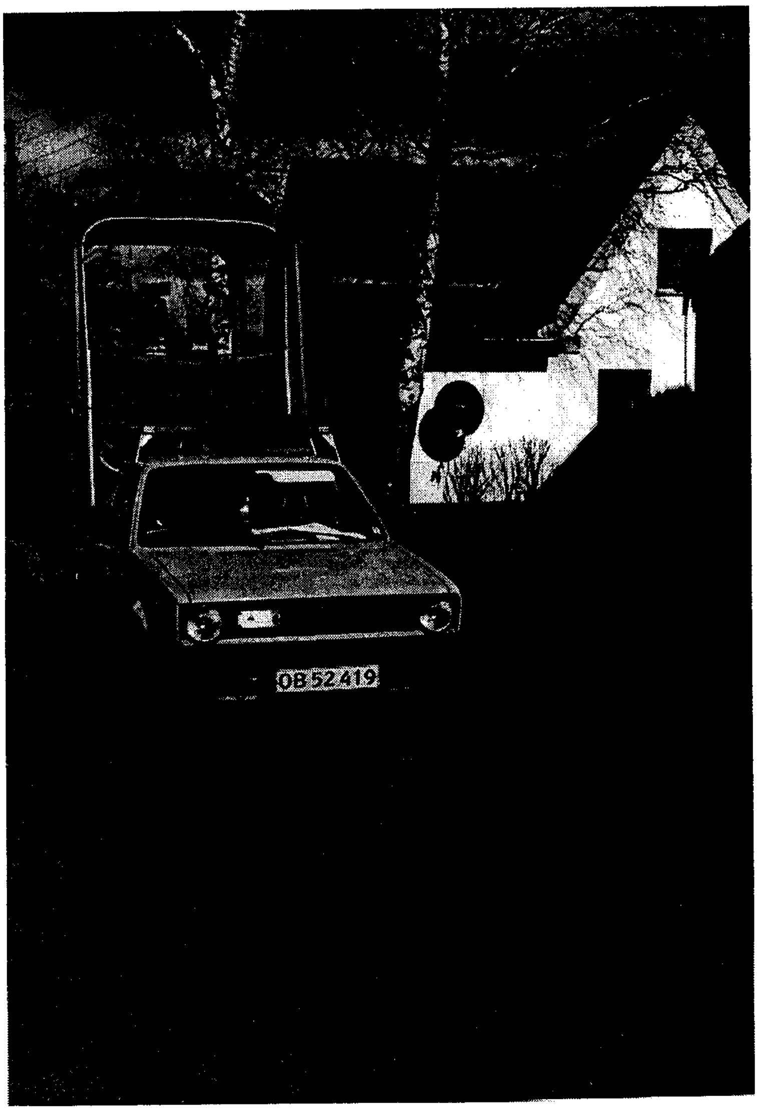


Figure 1. Photo of the benchmark connections by MTL.

In 1989 MTL was developed seriously for use in the geometric 2. order levelling. The first instruments (Kern E12) were changed to new and better instruments (Topcon GTS-4). Kern E-12 total station were dumped because of some problems with data communication, and problems with measuring long distances over 400 m. The display of the Kern E-12 was only in units of 2′′. The real production was started in 1992.

2. Application of MTL

The largest benefit of using MTL in the Danish levelling network was the increase of production in the order of 100% compared to Motorized Geometric Levelling (MGL). Even though the speed of MGL is very high there still is (and will always be!!) need for higher speed to measure the hole country in due time. This is achieved by MTL. In fact, this was the main reason for development of MTL for the production in Denmark. In a lot of areas we are ready to accept a decrease in accuracy in order to fulfil the users demands of height product in Denmark. The users of heights in Denmark are public authorities, private contractors and local surveyors. Many of the users are really not using the good accurracy that is given when measuring with MGL.

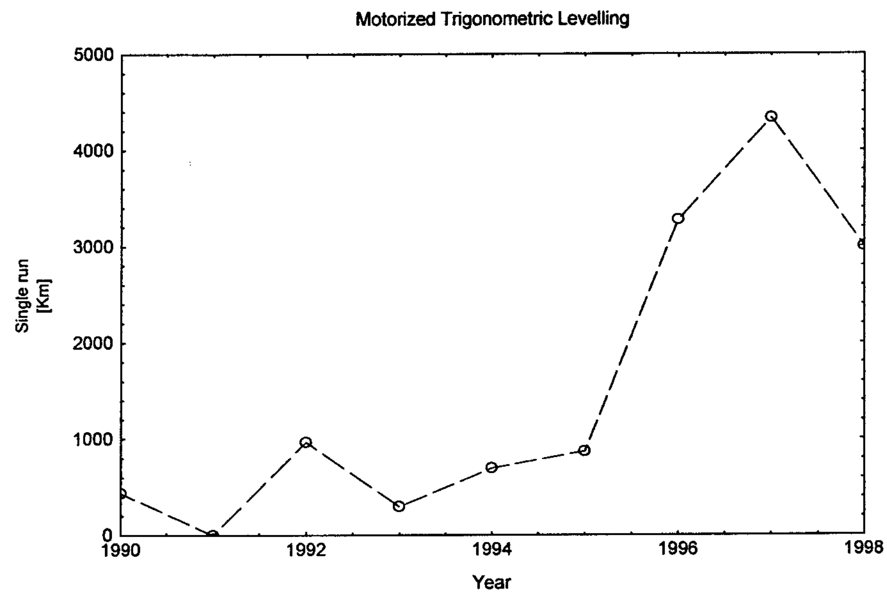


Figure 2. Production of MTL in Denmark since the beginning of production.

The fieldwork is done by one team, composed by 2 surveyors and 2 drivers (one of the drivers on each teams are very experienced) The daily produktion of a full day is between 25 - 30 km (average over a week) with a relevelling rate of 4 - 5%. The accuracy of MTL is approximately 4 times worse than MGL, normally the accuracy is 3 - 4 mm*√km.

Since the start of MTL in Denmark we have produced 14.000 km single run levelling with MTL. Since 1996 there has been 2 teams using MTL. One of these teams is over a season changing between MTL and MGL. The levelling season in Denmark is starting in the beginning of March and ends in the beginning of December with a summer break of approximately 7 weeks.

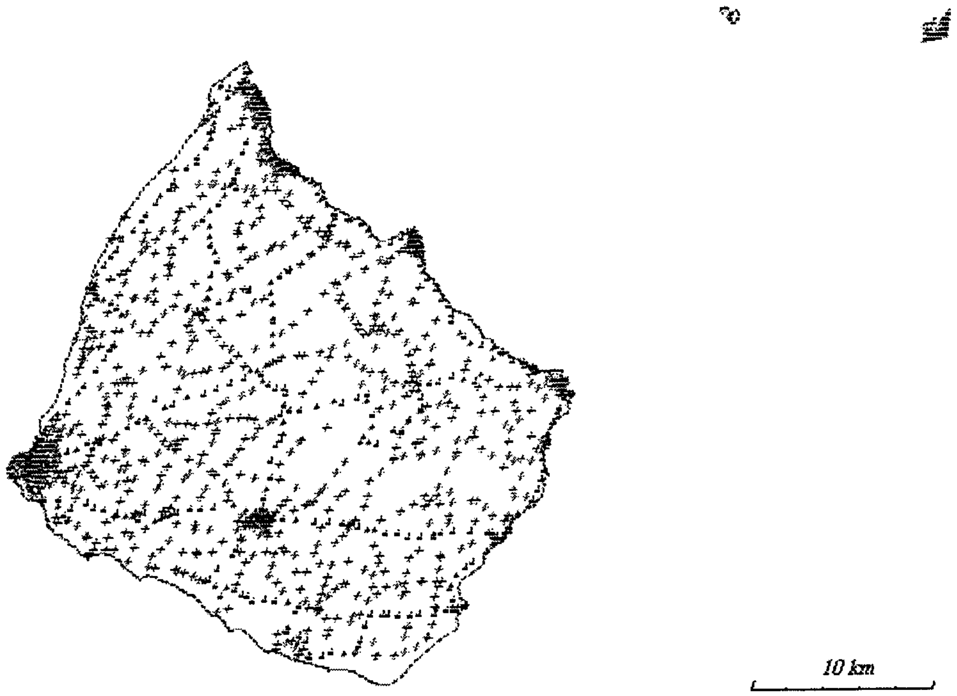


Figure 3. Plot of Bornholm with all the measured height benchmarks. The dark points (A) are points measured with MGL and the bright points (+) are measured with MTL.

3. Equipment.

Since the start of MTL production only a few changes of the practical use have been made, still using the same cars and the same instruments. The technical equipment of 2 cars we are using is as follows. Car number one is equipped with 1 Topcon GTS-4 (note that this instrument has a display unit in 1'') connected to a laptop computer and printer. On the car number 2 the Topcon is connected with the laptop computer on car number 1 by wireless datacommunication. The 2 cars are naturally equipped with radios (mobile) and so are the surveyors too. The laptop computer which is used is a equipped with TFT screen. The program that is used for data collecting is developed in Basic and runs in a normal DOS shell.

4. Experience from field work.

4.1. Comparison of MTL and MGL accuracy.

In the late summer of 1989 the first test measurements (70 km single run) were performed. Three adjoining loops measured with MGL were relevelled short time after applying MTL. Contrary to MGL the influence of refraction under unfavourable weather conditions is clearly visible. During a few days the measurements were very disturbed by flicker due to high air temperature. Though this should effect the levelling at random only the sum of deviations between MTL and MGL heights was increased by an amount of 5mm/km. At the same time the sum of forward and backward run deviations decreased in the order of 7mm/km.

Remeasuring the sections effected by flicker the following results were obtained:

loop closing error differences, MGL-MTL: 9.7 mm 13.8 km

0.0 mm 4.9 km

2.7 mm 9.5 km

absolute deviations between MGL and MTL heights:

52% < 1mm

94% < 2.5mm

4.2. Mean refraction from reciprocal zenith distances, measured simultaneously.

Denoting the true values by z_A and z_B and considering $z'_A=z_A-r_A$ and z'_B defined correspondingly, where r is the refraction counted negative if the sight length is below the straight line between instrument A and B, then it is well-known

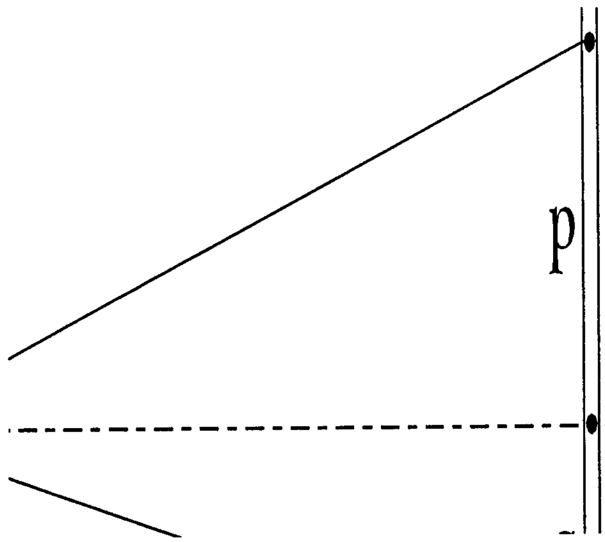
$$\tfrac{1}{2}(k_A+k_B)=1+(R/s)(\pi-(z'_A+z'_B))$$

where k is the coefficient of refraction, s is the reciprocal sight length and R is an earth radius. The value obtained from this equation inserting the actual measurements into the right-hand side is called the computed mean refraction. During the test measurements mentioned in the preceding section it was noticed that the computed mean refraction could become unexpectedly large when air temperature was high. To find out if this could be caused by the Kern E12 instruments it was decided to control the measurements when this problem occurred. Then two Wild T2 theodolites were set up close to the Kern E12 instruments and zenith distances were measured almost simultaneously with the Kern measurements. It turned out that the computed mean refraction from both types practically was identical. Also, the absolute value of the computed mean refraction could be about 20 times larger than standard refraction (k=0.13); this was noticed at 2 p.m. when air temperature was 28.1°C and the sight length 465m. It was noticed, too, that this maximum value changed considerably due to small variations of the solar radiation even though air temperature practically was unchanged.

Mean refraction is currently computed during the field work. Since large values also can indicate a large refraction difference sight length is shortened, when the computed absolute value is exceeding, say, 10 times the standard refraction. Negative values are normal, but a few times positive signs have been observed (before noon, when the sun was shining on snow-covered ground and late in the evening on a sunny summer day). This is in agreement with Jordan/Eggert/Kneissl: Handbuch der Vermessungskunde, 10. Ausgabe, Band III, § 33. According to the considerations, ibid., levelling should be stopped, when positive values are noticed.

5. Bench mark connections.

5.1. Measurements.



The theodolite's height above the bench mark has to be determined at both ends of the levelling section. At the ending point the instrument is kept in an un- changed position to avoid a new determination, if the levelling is continued. To achieve the height determination the theodolite is set up near the bench mark (5-75m) to measure the zenith distances of four selected cm-lines of a levelling rod placed on the bench mark. Due to Danish topography most- ly the instrument can be set up such that the horizontal sight line (instr. level) is crossing the rod. First the graduation lines are measured in telescope position I in the order of decreasing cm-values, and then in position II in the opposite order. The computed means of both positions are used as observations of the true zenith distances $Z_1,...,Z_4$ (corresponding to decreasing cm-values).

5.2. Height formula.

The figure above illustrates the configuration of theodolite, rod, and cm-lines. At first, only two of the cm-lines are considered for the sake of simplicity. Z_1 and Z_3 are the true zenith distances of the two lines and h_1 and h_3 are the corresponding true distances from the rod base. Then the instr.

$$(1) H = \frac{h_3 \cot Z_1 - h_1 \cot Z_3}{\cot Z_1 - \cot Z_3}$$

height $H=h_3+q=h_1-p$ above the bench mark is determined by

and, similarly, by the other two cm-lines.

5.3. Error propagation.

In order to make general conclusions in what follows it is supposed that the horizontal distance S from the theodolite to the rod exceeds the rod length and that the height elevation angles (90°-Z) of lower and upper end of the rod both are in the range of ±45°. Concerning the instrumental height H it is obvious from the formulas below the propagation of systematic errors and random errors as well is highly depending on the ratios p/d and q/d, where $d=h_1-h_3$. Note, p and q have to be counted negative, if the upper/lower line is below/above the instrument level. Applying the Taylor

$$(2) \delta H = \frac{p}{d}\delta h_3 + \frac{q}{d}\delta h_1 + \frac{p}{d}\left(1+\left(\frac{q}{S}\right)^2\right)S\delta Z_3 + \frac{q}{d}\left(1+\left(\frac{p}{S}\right)^2\right)S\delta Z_1$$

expansion of the height H the impact of systematic errors $\delta h_1,...,\delta Z_3$ on H can be written

As to be seen from this formula, if the two lines have been chosen on both sides of the instrument level, the configuration is favourable in the sense, that the propagation of each of the errors $\delta h_1,\ldots,S\delta Z_3$ is smaller than the error itself. However, such a configuration is not necessarily the best one, this is entirely depending on the kind of systematic error in consideration.

With respect to random errors the four zenith distance observations $Z_1,\ldots,Z_4$ are considered as uncorrelated and unbiased random variables, having the variance VarZ in common. Applying the law of error propagation to the Taylor expansion above the contribution of the random variables Z_1

$$(3)\,VarH=\left(\left(\frac{p}{d}\left(1+\left(\frac{q}{S}\right)^2\right)\right)^2+\left(\frac{q}{d}\left(1+\left(\frac{p}{S}\right)^2\right)\right)^2\right)S^2\,VarZ$$

and Z_3 to the variance of the corresponding random height can be written

Note, according to Hall, A.: Statistical Theory with Engineering Applications, second printing, 1955, formula (5.9.7), p, q, d, and S should be considered as the true values shown in the figure above. From this formula we can derive some rules for the choice of the two cm-lines to minimize varH keeping fixed the actual setup. Note, in any case, $\frac{1}{2}S^2$VarZ is a lower bound of varH.

Given a fixed distance d of the cm-section observed, varH will be smallest if the lines are chosen symmetrically on both sides of the instrument level. Then, however, varH practically is independent from the specific value of d. A strictly symmetrical choice cannot be done, of course, therefore the lowest bound above will be exceeded, but if the two lines are on both sides of the instrument level the situation is still favourable, since varH will be bounded by S^2VarZ. However, sometimes the lines have to be on the same side. In this case, regarding an arbitrary choice of the two lines, varH will decrease continuously if the corresponding section is enlarged upwards and downwards (without crossing the instrument level by assumption).

On the other hand, keeping p and q fixed and recalling that VarZ may depend on S, too, one may ask, if the actual distance S is the optimum distance to minimize varH. However, to determine the length of that distance VarZ as a function of S has to be known.

Finally we give an example illustrating that short cm-sections displaced from the instrument level and observed in a long distance should be avoided. The table below shows the values of $(\text{varH})^{1/2}$ computed from the formula above assuming S=75m and VarZ=$(2")^2$. Denoting the height above the instrument level of the midpoint of the cm-section observed by m the following values are found:

m=0m,d=3m	m=0m,d=1m	m=11.5m,d=3m	m=11.5,d=1m
0.51mm	0.51mm	4.07mm	12.12mm

The cm-lines of the other two zenith distances should be close to the lines of Z_1 and Z_3 (supposed they have been chosen according to the rules above). Then, the variance of the random height H_{24} corresponding to Z_2 and Z_4 is small, too, and p and q will have about the same value as for H_{13}. Consequently, both random heights will have approximately the same variance. Hence, the unweighted mean, _, is a good estimator of the instrumental height H, and Var_≈½VarH_{13}.

6. Determination of the section height difference.

6.1. Measurements.

After the connection of the bench mark at the beginning of the section the height differences between two theodolites, moving forward alternately, are determined from reciprocal measurements of simultaneous zenith distances and the corresponding sight lengths at each corresponding setup. For this purpose target plates and distance prism are fixed to the theodolites about 20cm above the telescopes in such a way that the measured distances are the sight lengths of the zenith distances. Sight lengths are measured by both theodolites, but only once, because the accuracy is fully sufficient for second order MTL. Zenith distances, however, are measured repeatedly, where the number of measurements (minimum=standard=2) is depending on the observers' estimation of the measuring accuracy.

To speed up the production the number of corresponding setups is kept as small as possible, i.e. sight lengths (about 330m on average) are very varying. All sections are measured forward and backward during one or two days aiming at the same number of corresponding setups. If possible, the positions of the backward run setups are chosen differently from the forward run positions. Sights close to terrain elevations are avoided.

6.2. Repeatability of zenith distance measurements.

These measurements are subject to errors from the measuring system (observers, technical equipment) and from the physical environment, i.e. first of all, refraction, which is separated into a mean refraction, constant during the short measuring time at the setup but possibly varying during longer time spans, and the corresponding refraction deviation. Hence, the observation z (the mean from both telescope positions) of the true zenith distance ζ is written

(4) $$z=\zeta-\delta z-r_m-\delta r$$

where δz, r_m, and δr are the error contributions from the measuring system, the mean refraction and the refraction deviation. With regard to repeated zenith distance measurements δz, the measuring error proper, and δr, the refraction fluctuation, are considered as random variables. Applying the usual formula of refraction, $r=sk/(2R)$, where k is the refraction coefficient, s is the sight length and R=6400 km, e.g., the variance of z can be written

$$(5)\ Var z = Var \delta z + \frac{s^2}{4R^2} Var \delta k$$

where δk is the refraction coefficient corresponding to δr.

Taking this equation as a reference point the significance of the refraction fluctuation δr has been investigated. Based on the measurements from 1995-98, including more than 12000 corresponding setups, the sample variance $s^2(z)=\frac{1}{2}(z_1-z_2)^2$ of the first and the second zenith distance observation has been computed for each single setup, resulting in an average value of about 2 square seconds. According to temperature and sight length these $s^2(z)$-values have been classified and the arithmetic mean of each class has been calculated. It turned out very clearly that even for the long sight lengths>1000m the $s^2(z)$-values do not increase with increasing temperature as it might be expected from the second term on the right-hand side of (5). Therefore, the classification according to sight length only seems to be sufficient. In the table below the means of the sight lengths and of the $s^2(z)$-values in square seconds are indicated for each class of sight length. Compared to the measuring error proper the influence of the refraction fluctuation δr on the variance of zenith distance measurements seems to be insignificant. Furthermore, apart from the short sight lengths, this variance seems to be almost unaffected by the sight lengths. With respect to the short sight lines the table might illustrate the well-known phenomenon that the pointing error is rapidly increasing for decreasing short sight lengths.

19m	39	63	88	125	175	247	348	486	686	884	1080	1389m
5.2	3.6	2.7	2.3	2.2	2.1	1.9	1.9	1.9	1.7	1.7	1.4	1.7

6.3. The computation of the height difference.

The true height difference H of the section is determined by the true instrumental heights h_0 and h_{n+1} referring to the bench mark connections at the beginning and at the end of the section, respectively, and the true height differences $h_1,...,h_n$ between the optical axes of the theodolites at the corresponding set ups

(6) $$H=h_0+(h_1+...+h_n)-h_{n+1}$$

Furthermore, the height diff. h_i from theodolite A to theodolite B, say, is determined by

(7) $$h_i=s_{Ai}\cos\zeta_{Ai}-\Delta_B=-(s_{Bi}\cos\zeta_{Bi}-\Delta_A)$$

Here ζ_{Ai} is the true zenith distance from A to the target plates of B, s_{Ai} is the corresponding sight length, Δ_A is the displacement of the signal plates of A from the telescope, etc.. Hence, we are computing from each pair of simultaneous zenith distances z_{Aij}, z_{Bij}, $j=1,...,p_i$, the height differences h'_{Aij} and h'_{Bij} according to (7) and from this the arithmetic mean

$$h'_{ij}=\tfrac{1}{2}(s_{Ai}\cos z_{Aij}-s_{Bi}\cos z_{Bij}-\Delta_B+\Delta_A)$$

Note, $h'_{Aij}-h'_{Bij}$ mainly reflects refraction but not the measuring accuracy proper. Now, the arithmetic mean of the h'_{ij}- values is the computed height difference h_i'. Finally, the section height difference is calculated from computed values according to (6).

6.4. Error propagation.

To derive a formula of the variance of h_i' we linearize h_i' with respect to the zenith distances and replace the sines by 1 and s_{Ai} and s_{Bi} by s_i. Then the law of error propagation is applied and (4) is taken into account, considering mean refraction as a random variable. Now, assuming an equal variance, $\mathrm{Var}\delta z_i$, of the measuring errors proper and of the mean refractions as well as, in addition to vanishing correlations, except the correlation between the mean refractions, and finally neglecting the variances of the refraction fluctuations according to the investigation mentioned in sect.6.2., we obtain

$$(8) V'ar\ h_i = \frac{s_i^2}{2p_i} Var\delta z_i + \frac{s_i^4}{16R^2} Var\Delta k_i$$

where Δk_i is the difference between the mean refraction coefficients at the setups of A and B. With respect to the variance of the computed section height difference $H'=h_0'+(h_1'+...+h_n')-h_{n+1}'$ an equal variance, $Var\Delta k$, of the refraction differences Δk_i is assumed. Also, referring to the investigation in sect.6.2., an equal variance, $Var\delta z$, of all the δz-errors involved. Furthermore, concerning the contributions from the bench mark connections they normally are of minor importance due to the short sight lengths, i.e. a rough estimation of Var_ is sufficient. Following the rules outlined in sect.5.3., $Var\_ \approx \frac{1}{2} VarH_{13} \approx \frac{1}{2} S^2 VarZ$. According to an investigation from 1991 VarZ is depending on the sight length S; however, VarZ=1 square second is a good average value. Thus, according to sect.6.2., $Var\delta z$ is about twice as much, hence, on average

$$Var\_ \approx \frac{1}{4} S^2 Var\delta z.$$

Now, denoting the sight lengths from the connections at the beginning and at the end of the section by s_0 and s_{n+1} and neglecting the covariances between all the computed height differences involved the variance of the computed section height difference H' can be

written

(9) $$VarH' = \frac{1}{2} S'^2 Var\delta z + 1/(16R^2) \cdot S''^4 Var\Delta k$$

where $S'^2 = \Sigma_{i=0}^{n+1} s_i^2/p_i$, $S''^4 = \Sigma_{i=1}^{n} s_i^4$, and $p_0=p_{n+1}=2$. However, the mean refractions of the forward and of the backward sights at the same setup probably are correlated, implying a correlation between neighbouring corresponding set ups. As to be seen from (9) subdividing the section length into several corresponding set ups with equal length is decreasing VarH'. Note, the term $1/S'^2$ is the weight used in our adjustment program.

It should be investigated if the refraction term (R) of (9) can be neglected. This could be done based on a sufficient number of forward and backward runs, each run with one corresponding setup and $p_1=2$. Then the S'^2-values of both runs will almost be identical (of course, short sections<200m, say, should be excluded). Consequently, defining the normed run difference

$$R_1 = (H'_f + H'_b)/S'$$

where S'^2 is the mean of the S'^2-values from both runs, the square root of $VarH'_f VarH'_b$, which occurs in the expression of the correlation $\rho_{H'}$ between forward and backward run, should be replaceable by the mean of $VarH'_f$ and $Varh'_b$. Hence

(10) $$VarR_1 = (1+\rho_{H'})(Var\delta z + (1/S'^2)((R)_f + (R)_b))$$

i.e. $VarR_1$ is made up by two terms, one is independent from section length, whereas the other one depends on it through refraction. Consequently, classifying the R_1-values appropriately and computing for each class the sample variance $s^2(R_I)$ of the corresponding values should give an idea of the significance of the refraction influence.

This investigation has not been done, yet, but we can indicate a result from an investigation concerning measurements from 1993 (however, summer measurements have not been represented). Here $s^2(R_I)=5.1$ square seconds was found from about 160 double runs all of the specific kind mentioned above. According to (10) this is an estimate of $(1+\rho_{H'})\mathrm{Var}\delta z$ plus a certain mean influence of refraction. Hence, the computed value is a significant increase of the estimated value of $\mathrm{Var}\delta z$ in sect.6.2., indicating a considerable contribution from the refraction, especially, when it is recalled that $\rho_{H'}$ might be negative as in geometric levelling.

7. Future developments.

In Denmark we have been discussing how to develop MTL for the future. One of the things we have seen as a possibility is the use of the new servototalstations. Servototalstations have built in automatic features for measuring. It might be a possibility to get a higher accuracy from these instruments and a higher production. Anyway we have to look for new instruments for our teams because the 4 instruments we are using now are very worn. In fact last week we lost one of our instruments (it was dropped from a car) and the team is now using our only spareinstrument and we do not have any substitute for this right now.

Sensitive high-speed-railway track control

Erik W. Grafarend
Department of Geodetic Science
University of Stuttgart
Geschwister-Scholl-Str. 24D
D-70174 Stuttgart
Fax 0049-711-121 3285
e-mail: grafarend@gis.uni-stuttgart.de

Abstract

High-speed-railways / bullet trains / Transrapids / need „extreme reliable control systems“ (*extasy*) to avoid catastrophic events. Here we consequently focus on two sensitive problems related to *extasy*: (i) a local high resolution representation of the track design (clothoid, circle, straight line) in UTM *map matching coordinates* is given. (ii) the *Mixed Model* (universal Kriging) is used to discriminate measurement errors from track displacements. (*E. Grafarend and B. Schaffrin:* Ausgleichungsrechnung in linearen Modellen, B.I. Wissenschaftsverlag, 483 pages, *§2f,* Mannheim 1993)

Levelling by GPS: the state of the art in France.

Henri Duquenne

Ecole Supérieure des Géomètres et Topographes, 1 boulevard Pythagore, Campus Universitaire du Maine, F72000 LE MANS, France

e-mail : henri.duquenne@esgt.cnam.fr

Abstract

For GPS levelling applications, it is convenient to express accurately the height reference surface in a suitable geodetic reference system. This can be obtained through a set of levelled GPS points which realizes efficiently for the end-user both the geodetic reference system and the height reference system. Unfortunately, available data are sparse. The paper shows how a gravimetric geoid can be used to interpolate the height reference surface issued from GPS and levelling. Both surfaces do not coincide exactly with each other for several reasons, which are explained. At this point, one must compare two realisations of the geoid, detect outliers, retrieve (if possible) the causes of the discrepancies and finally combine the two kinds of data. The paper presents a method to reach these objectives and how it was applied in France. Numerical tests show that levelling by GPS is feasible by standard methods, leading to a precision of 2-3 centimetres.

Introduction

Although GPS does not provide directly orthometric or normal heights as it do for ellipsoidal coordinates, surveyors aim at simultaneous horizontal and vertical positioning for economical reasons. The well known solution to this problem lies in the determination of the geoid (or quasigeoid) undulation. Two kinds of methods were proposed during the last years : local ones and regional or global ones. Local method consists in measuring the height of some reference geodetic points in the area of interest (or the ellipsoidal height of levelling benchmarks) and to interpolate the geoid undulation on other GPS points. This method has some advantages : it seems simple and easy, it works well (but with some precautions) even if the reference geodetic network is old (bi-dimensional) or local. The method has also some drawbacks : time is loosen to link GPS surveys to the levelling reference network, and the optimal distance between needed links depends upon the variability of geoid undulation and is not well known. Regional or global methods proceed in a different way : geoid undulation or height anomalies are computed in a large area from several kinds of data (gravity values, deflection of the vertical, satellite altimetry, global geopotential model, etc.). As we shall see later, the geoid model must be slightly adapted, so that *the geodetic reference system of the geoid be the same as the one used for GPS.* Surveyors can then use the adapted geoid model to convert ellipsoidal heights into orthometric or normal heights. There are several advantages in such a solution : the geoid model constitutes a unique, global, well controlled height reference, avoiding proliferation of local references; no links are needed between surveys and the levelling reference network. The drawbacks are that a tri-dimensional geodetic network must pre-exist, it must be used to set up and use the geoid model, and the computation procedures of coordinates and heights must be slightly changed as the ellipsoidal heights are explicitly needed to derive orthometric or normal heights.

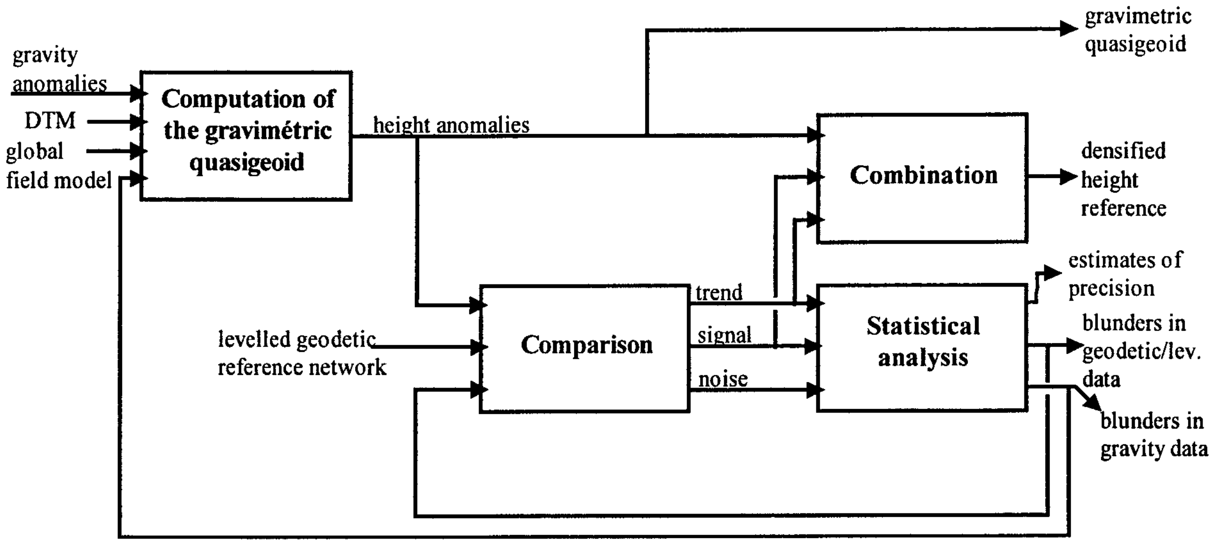


Figure 1 : Computation of a gravimetric (quasi)geoid and adaptation for levelling by GPS.

Several methods can be used to build up a (quasi)-geoid model and to arrange it for levelling by GPS. We will not review all of them, and present only the one which was used in France. The flow chart lies on figure 1. The stages of computation, including quasigeoid computation, comparison with levelled GPS points, statistical analysis and adaptation of the quasigeoid, are described in the paragraphs below. Evaluation tests by end-users are also presented.

The QGF98 quasigeoid

The computation of the QGF98 quasigeoid was carried out using the Residual Terrain Method (RTM) as described in Forsberg (1994). 557 913 gravity values were collected mainly from the BGI database (figure 2). Sandwell anomalies were used in the Atlantic ocean, and Morelli's data in the Mediterranean sea (Behrend et al., 1996). The University of Montpellier provided some data in the Pyreneans (Doerflinger, 1997). Gravity anomalies referred to the GRS80 ellipsoid were corrected from the effect of the atmosphere, using the IAG model (Moritz 1992). In the RTM, gravity free-air anomalies Δg and height anomalies ζ are split in three parts, the global model parts (Δg_{GM} and ζ_{GM}), the residual terrain parts (Δg_{RT} and ζ_{RT}) and the residual part (Δg_{Res} and ζ_{Res}). The global model and residual terrain parts of the gravity anomalies were computed by :

$$\Delta g_{GM}(r,\theta,\lambda)=\frac{GM}{r^2}\sum_{n=2}^{n_{\max}}(n-1)\left(\frac{a}{r}\right)^n\sum_{m=0}^{n}\overline{P}_{n,m}(\cos\theta)(\Delta\overline{C}_{n,m}\cos m\lambda+\Delta\overline{S}_{n,m}\sin m\lambda) \tag{1}$$

$$\Delta g_{RT}=G\rho\int_{RT}\frac{(H-H_0)\,d\tau}{l^3} \tag{2}$$

As a global model, OSU91A was used (Rapp and al., 1991). Replacing the global model OSU91A by EGM96 do not improve significantly the present solution, because no new gravity data from continental France was incorporated in EGM96 (Duquenne, 1997b). Terrain correction was obtained by numerical integration and TC program (Tscherning et al., 1992). Although FFT techniques are much faster, they use approximation formulae based on series which diverge in very rough topography : in the Alps, the height of the residual terrain often exceeds 500 m, it can be larger than the mesh size of the used DTM (140 m).

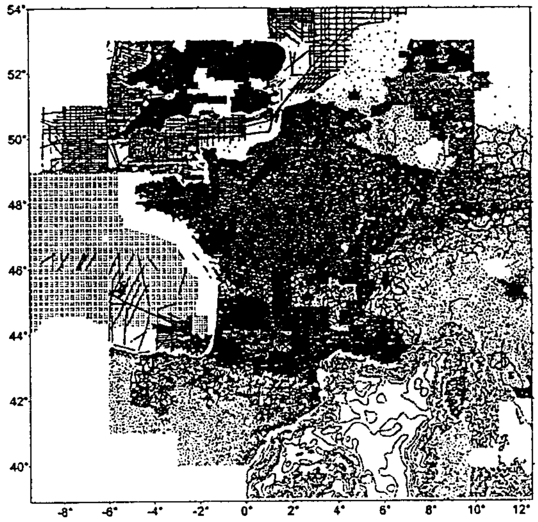


Figure 2 : Gravity data coverage

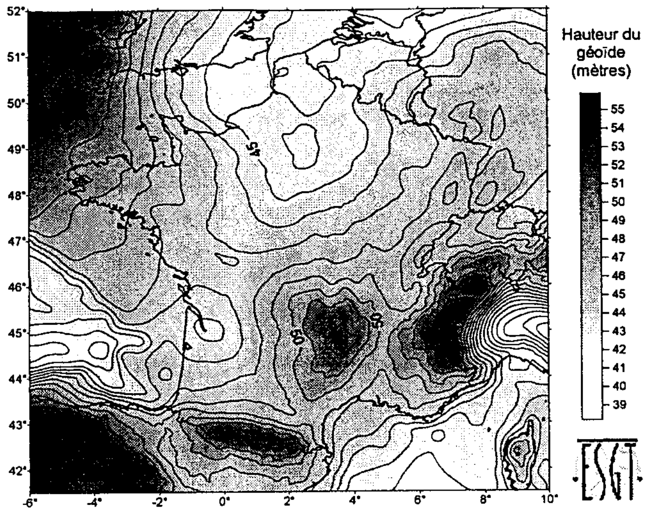


Figure 3 : The French QGF98 quasigeoid model

The residual gravity anomalies were then computed :

$$\Delta g_{Res} = \Delta g - (\Delta g_{RT} - \Delta g_{IRT}) - \Delta g_{GM} \tag{3}$$

Δg_{IRT} stands for the indirect effect. The residual anomalies were gridded, using a mesh size of $45'' \times 60''$, then integrated by the Stokes formula :

$$\zeta_{Res} = \frac{R}{4\pi\gamma} \int_{\sigma} \Delta g_{Res} S(\psi)\, d\sigma \tag{4}$$

After several tests, a radius of integration of 2° was retained in the final solution : this question is discussed in the sequel.

The residual terrain effect and the global model height anomalies were restored :

$$\zeta_{RT} = \frac{GR^2}{\gamma} \int_{\sigma} \frac{\rho(H - H_0)\, d\sigma}{l} \tag{5}$$

$$\zeta_{GM} = \frac{GM}{r\tilde{\gamma}} \sum_{n=2}^{n_{\max}} \left(\frac{a}{r}\right)^n \sum_{m=0}^{n} \overline{P}_{n,m}(\cos\theta)(\Delta\overline{C}_{n,m} \cos m\lambda + \Delta\overline{S}_{n,m} \sin m\lambda) \tag{6}$$

$$\zeta = \zeta_{GM} + \zeta_{Res} + \zeta_{RT} \tag{7}$$

Finally, the height anomalies are corrected for differences in *GM* and equatorial radius of the spheroids. The final quasigeoid model is mapped on the figure 3. It is available as a grid with mesh size of $1{,}5' \times 2'$.

Comparison of QGF98 with GPS-leveLling data

Since 1996, the Institut Géographique National achieved a GPS geodetic network called RGF (Réseau Géodésique Français) covering France and Corsica. More than 1000 of these points are precisely levelled, so that a value of the height anomaly can be derived from the ellipsoidal height h and the normal height H :

$$\zeta_{GPS-Lev} = h - H \tag{8}$$

These values differ from the ones computed from gravity data for several reasons :

- Unmodelled difference in geodetic references of the global model and the geodetic network may cause discrepancies of some centimetres with very large wavelength.
- Errors in the coefficients of the global model bring errors in the gravimetric quasigeoid, with wavelength error larger than the radius of Stokes integration.
- Systematic errors in gravity data, blunders and lack of data lead to locally correlated errors in the quasigeoid. The distance of correlation increases from some kilometres to about the radius of Stokes integration, according to the module and the expanse of the errors in gravity data. Limiting the radius of Stokes integration causes the same kind of errors.
- Tropospheric effects on GPS measurements produce locally correlated errors (up to 0.1 m) on $\zeta_{GPS-Lev}$. Distance of correlation may reach twenty or so kilometres, depending upon the mean length of the baselines and the number of GPS receivers. Other errors and blunders in the GPS geodetic network have uncorrelated effects.
- The French levelling network is affected by a systematic error towards the north (about 0.30 m/1000 km), and probably by small locally correlated errors in the mountainous areas. On the other hand, the datum surface of the levelling network is fixed at a tide gauge where the mean sea level does not coincide with the gravimetric geoid.

At any point P_i, the coordinates of which are (φ_i, λ_i), the discrepancies with wavelength or correlation distance larger than the size of France (1000 km) can be modelled by a trend, the other errors by residuals v_i :

$$\zeta_{GPS-Lev,i} - \zeta_{Grv,i} = a + b(\varphi_i - \varphi_0) + c(\lambda_i - \lambda_0)\cos\varphi + v_i \tag{9}$$

where a, b, c are parameters which can be computed by least squares adjustment, and (φ_0, λ_0) are the coordinates of any reference point. Residuals are locally correlated and can be represented by contour lines as in figure 4 and 5. Large isolated values indicate blunders in levelling or GPS networks (corrected or removed before drawing fig. 4 and 5). Spots with some extent reveal errors with medium distance of correlation, mainly due to errors in gravity data. The standard deviation of the residual σ_v is a synthetic estimate of the discrepancies with medium or short distance of correlation. In order to optimise the final QGF98 solution, various radii of Stokes integration were tested. From figures 4, 5 and 6 it can be concluded that increasing the radius improves significantly the precision along the Atlantic coast, but increases drastically the residuals along Côte d'Azur, worsening the global precision.

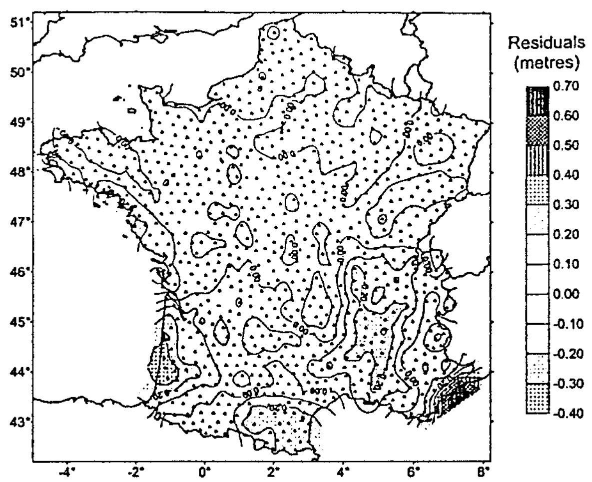


Figure 4: Residuals of the comparison of QGF98 (Stokes radius 2°) with the RGF levelled GPS points.

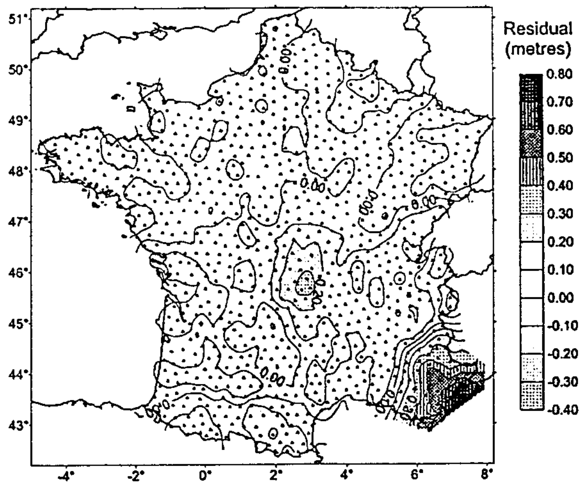


Figure 5: Residuals of the comparison of QGF98 (Stokes radius 3°) with the RGF levelled GPS points.

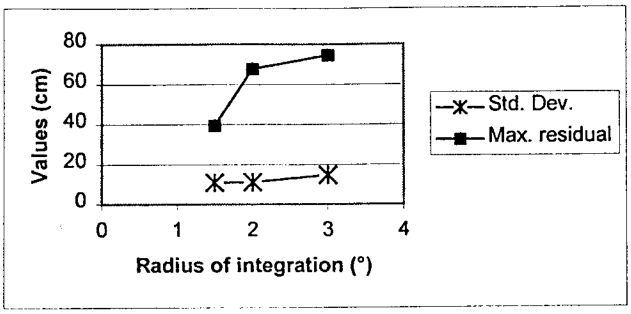


Figure 6 : Variation of precision of the quasigeoid with the radius of Stokes integration.

Attempting to include the Corsican part of the geodetic network in the adjustment revealed large errors in the gravity data on or around this island. The radius of integration chosen for the final solution (2°) is a compromise between global and local precision.

Surveying blunders and errors in gravity data may produce errors in the gravimetric quasigeoid and in the levelled GPS points of the same order of magnitude. In order to tell apart the two kinds of errors, it is possible to refine the analysis of the residuals by splitting them in signal and noise :

$$v_i = s_i + n_i \tag{10}$$

The signal represents the correlated part of the residual (mainly errors in gravity data, but also tropospheric effects on GPS measurements, correlated levelling errors), while the noise represents the uncorrelated part (surveying errors, blunders). For any levelled GPS point P_i of the control network, the signal and noise can be estimated by interpolation of the residuals of the neighbouring points $P_j, j \neq i$. Using the most precise linear interpolator (collocation) (Heiskanen and Moritz, 1967, p 266-270), one gets :

$$\hat{s}_i = \sum_{\substack{j=1 \\ j \neq i}}^{n} \alpha_{i,j} v_j \tag{11}$$

$$\hat{n}_i = v_i - \hat{s}_i \tag{12}$$

n is the number of levelled GPS points and the coefficients $\alpha_{i,j}$ are obtained by solving :

$$\sum_{k \neq i} \alpha_{i,k} C_{m,k} = C_{i,m} \tag{13}$$

The $C_{m,k}$ are covariances between residuals and are computed from a covariance function modelled from the data. The conditions $j \neq i$ and $k \neq i$ in (11) and (13) avoid the effect of large errors in the estimation process and secure independence of $\hat{s}_i$ towards v_i.

It can be proved that the $\hat{n}_i$ are on average not correlated (Duquenne, 1998). If they are assumed to be normally distributed, or if this assumption is checked a posteriori by a χ^2 test, detecting blunders in levelled GPS network becomes very easy to be automated. The $\hat{n}_i$ provide independent estimates of uncorrelated errors in the levelled GPS data set, *even if the gravimetric geoid is contaminated by errors in gravity data.* This procedure was applied for the comparison of the QGF98 quasigeoid and the RGF. Levelled GPS points with noise module larger than 4 times noise RMS were rejected. Results are summarised in the table 1.

	Parameter	*Unit*	*Value*
a	constant bias	m	0.060
b	northward tilt	m/1000 km	−0.5258
c	eastward tilt	m/1000 km	0.7852
Min v	minimal residual	m	0.379
Max v	maximal residual	m	0.676
σ_v	RMS of residuals	m	0.109
σ_s	RMS of signals	m	0.104
σ_n	RMS of noises	m	0.034

Table 1 : Comparison of QGF98 and RGF GPS levelled points.

RAF98, a tool for levelling by GPS

Levelling by GPS is a practical application of geoid determination. Nevertheless, the gravimetric geoid must be corrected : one needs a numerical realisation of the reference surface ($H = 0$) of the height datum (IGN69 in continental France) expressed in the geodetic datum (RGF93 in France). (Milbert, 1995) proposed to use an error model like equation (9), the trend and noise on the levelled GPS points being interpolated on the nodes of the grid of the geoid. This technique was used for the first time in France to produce the RAF96 grid (Duquenne, 1997a), and used again to adapt the QGF98 for levelling by GPS, providing the RAF98 grid. Interpolation of the trend and signal was performed using collocation, GEOGRID software (Tscherning et al., 1992) and the same covariance function as in section 3. In order to validate the RAF98 grid before distribution, it has been tested, mainly by a working group established by the "Conseil National de l'Information Géographique", comprising geodesists, GPS receiver constructors and end-users, under the chairmanship of P. Willis. The group instituted test procedures and collected data to estimate the precision of levelling by GPS in standard conditions. The data consist of sets of levelled GPS points provided by surveyors and national institutions. The normal heights of the points observed by

Area		Point number		Discrepancies (cm)			
Name	Mean height (m)	Total	Rejected	Mean	Min.	Max.	RMS
Bordeaux	53	41	2	-2.1	-6.6	+0.1	2.7
Briançon	1141	18	0	+0.2	-3.7	+7.1	3.9
Carmeaux	283	8	8	-2.8	-3.8	-1.4	2.9
Cevennes	906	3	1	+0.2	-1.5	+1,8	1.7
Dijon	257	95	2	-0.1	-2.6	+2.5	1.2
Fréjus	165	6	0	+1.2	-1.5	+4.1	2.1
Le Mans	86	7	0	-0.1	-1.0	+0.9	0.6
Manosque	587	76	2	+0.4	-3.9	+5.4	1.9
Nice	184	12	0	+1.4	-1.4	+5.0	2.2
Paris	74	3	0	-4.3	-9.4	-1.5	5.6
Pau	361	3	0	+0.1	-1.2	+2.2	1.5
Pays Basque	91	26	2	0.0	-5.3	+4.7	2.1
Toulon	148	8	2	+2.0	0.0	+3.9	2.6
Var	163	5	1	+0.7	-3.0	+3.8	3.0

Table 2 : Tests of the RAF98 grid by independent levelled GPS points.

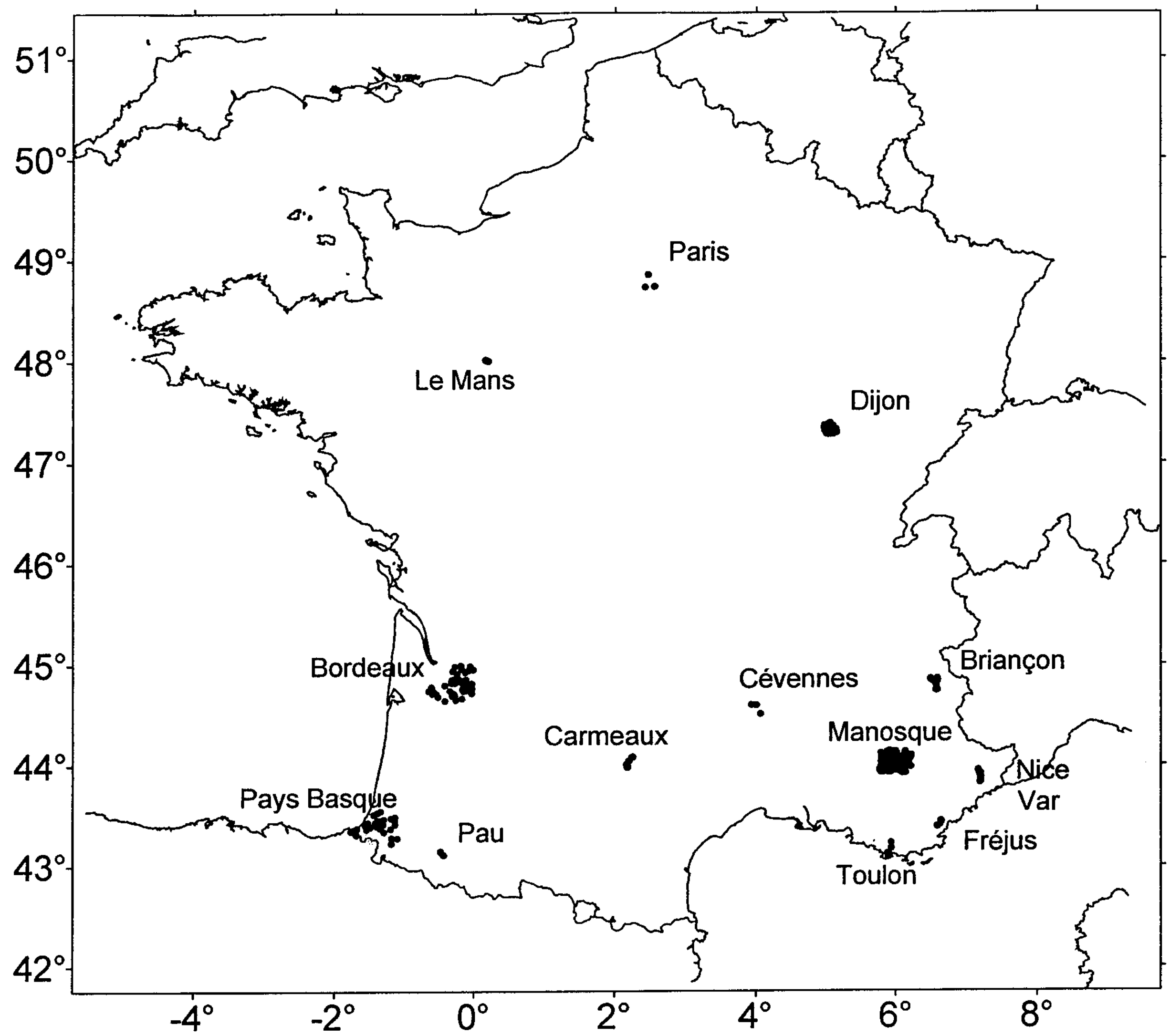


Figure 7 : Location of the data used to test RAF98

precise levelling have been compared with heights derived from GPS (static rapid) and RAF98. Although the report of the working group is not yet published, statistics issued from these tests are available and summarised in the table 2. Some interesting tests not proposed to the working group are incorporated in the table. Figure 7 shows the location of the tests.

It must be noticed that the last column of the table represents the root mean squares of the discrepancies, which were computed without removing of the mean discrepancies. It is not possible to separate the errors in RAF98 ("Référence Altimétrique Française 1998") from the one of the levelled GPS points used for testing it. Nevertheless, some conclusions can be drawn from these studies. Except for one set of points ("Paris"), the mean discrepancy (the mean local bias) is below 3 cm, and generally below 2 cm. The RMS of the discrepancies exceed 3 cm in two cases : for the test "Paris", which seems to be in error, and for "Briançon", in a high mountainous area. Two other tests in mountainous or hilly areas provide good results ("Cevennes" an "Manosque"). It is important to remark that in the south-eastern part of France where the quasigeiod model is erroneous, the test "Fréjus" shows that it is well corrected by the procedure of combination with levelling and GPS data. To summarise, it seems possible to retain the figure of 3 cm as a general estimate of levelling by GPS using the French geodetic network RGF, the RAF98 grid and static rapid GPS measurements.

Conclusions

This study shows how a modern (tri-dimensional) geodetic network and a gravimetric geoid model make possible levelling by GPS in a efficient way. The geodetic network, the points of which are levelled, realises the geodetic and the levelling references systems. But the network is not dense enough as a reference for levelling, due to the variabilty of geoid undulation and the large distance between geodetic points. The gravimetric geoid acts as a precise interpolator, but has no absolute accuracy, in the sense that it differs from the reference surface of the levelling network. Combining the geodetic network and the gravimetric geoid by a suitable process leads to an accurate realisation of the reference surface for heights determination. At the level of one centimetre, the old height reference surface is not modified and surveyors can use their old references with new measurement methods, without any trouble. In the specific case of France, one can remarks that, due to local conditions (high mountains, errors in gravity data), the gravimetric geoid model is not very precise (about 10 cm). But the combination with the levelled GPS network corrects the geoid at the level of 3 cm.

Acknowledgements

The author gratefully thanks the many Institutions and Persons who provided data and software, making this work possible:

the Professors R. Forsberg, C.C. Tscherning and P. Knudsen for the GRAVSOFT package, the Bureau Gravimétrique International, the Institut für Erdmessung (Hannover), and the Laboratoire de Géophysique et Tectonique (Montpellier 2 University) for the gravity data sets, the Ohio State University and Professor R.H. Rapp, for the global geopotential model, the AGIP Society, the National Research Council, the National Geological Survey of Italy and Professor Sansò for the Italian DTM, the Institut Géographique National and the Ecole Nationale des Sciences Géographiques for the French DTM and the geodetic data, the French Centre Géographique Interarmées for the DTM of other European countries, the Institut de Physique du globe de Paris, the Centre National de la Recherche Scientifique, the Société Nationale des Chemins de Fer Français and Electricité de France for GPS levelling data sets.

References

Behrend D., Denker H., Schmidt K. (1996) - Digital Gravity Data Sets for the Mediterranean Sea Derived from Available Maps. - in : *Bulletin n° 78*, Bureau Gravimétrique International.

Doerflinger E. (1997) Utilisation de la méthode de positionnement satellitaire GPS pour la détermination précise des altitudes relatives et absolues. - Thèse de doctorat, Université de Montpellier II.

Duquenne H. (1997a) - Le modèle de quasi-géoïde français et la surface de référence d'altitude RAF96. - Institut Géographique National et Ecole Supérieure des Géomètres et Topographes.

Duquenne H. (1997b) - Comparison between the EGM96 model and the French quasi-geoid model. *In*: Bulletin n°6 of the International Geoid Service. - DIIAR, Politecnico di Milano, Italy.

Duquenne H. (1998) - Comparison and combination of a gravimetric quasigeoid with a levelled GPS data set by statistical analysis. - Presented at the XXIII° General Assembly of the European Geophysical Society, Nice, France, 20-24 April 1998. Accepted by Physics an Chemistery of the Earth.

Duquenne H. (1998) - QGF98, a new solution for the quasigeoid in France. - *Proceedings of the Second Continental Workshop on the Geoid in Europe.* - Reports of the Finnish Geodetic Institute, M. Vermeer and J. Adam, editors. - Masala, Finland.

Forsberg, R. (1994) - Terrain effects in Geoid Computations. - Lectures Notes. In : *International School for the Determination and Use of the Geoid.* - International Geoid Service. DIIAR, Milan, Italy.

Heiskanen W.A., Moritz H. (1967) - Physical geodesy. - Institute of Physical Geodesy, Graz, Autriche - reprint 1981.

Milbert, D.G. (1995) - Improvement of a High Resolution Geoid Height Model in the United States by GPS Height on NAVD 88 Benchmarks. - In : *New geoids in the World.* - International Geoid Service, DIIAR, Milan, Italy *and* Bureau Gravimétrique International, Toulouse, France.

Moritz, H (1992) - Geodetic Reference System 1980. - In: *The Geodesist's Handbook,* Bulletin Géodésique, 66-2.

Rapp R.H., Wang, Y.M., Pavlis, N.K. (1991) - The Ohio State University 1991 Geopotential and Sea Surface Topography Harmonic Coefficients Model. - O.S.U. Report number 410.

Tscherning C.C., Forsberg R., Knudsen P. (1992) - The GRAVSOFT package. - First Continental Workshop on the Geoid in Europe. - Research Institute of Geodesy, Topography and Cartography, Prague, Czech Republic.

HEIGHTING WITH GPS: POSSIBILITIES AND LIMITATIONS

Matthew B. Higgins

ABSTRACT

Global Positioning System (GPS) surveying is now seen as a true three dimensional tool and GPS heighting can be a viable alternative to other more conventional forms of height measurement. This paper examines the limitations and possibilities of GPS heighting. The first part of the paper details the limitations of GPS heighting, including those factors that affect the GPS height measurement itself and the associated issues of geoid modeling and compatibility with the local vertical datum. The second part of the paper examines the possibilities for GPS heighting, focusing on three application areas currently generating interest for the practicing surveyor; deformation monitoring, real time GPS surveying and machine monitoring and guidance. These applications cover the range of achievable GPS heighting accuracy.

INTRODUCTION

Global Positioning System (GPS) surveying has been used extensively and with great success for the production and propagation of survey control. During the development of GPS surveying the focus was typically on horizontal control with the ability of GPS to measure height being seen as an added extra. GPS surveying has now matured to the point where it is seen as a true three dimensional tool. However, application of GPS to the measurement of height can be complex and solving the problems involved can account for the majority of the effort in finalising a GPS surveying project.

GPS measures heights related to the ellipsoid. In some cases ellipsoidal heights alone are sufficient for the type of survey being undertaken. However, many applications require heights that are related to a physically meaningful surface such as the geoid, or at least some attempt at realizing the geoid such as a surface based on locally observed mean sea level. Such physically meaningful heights take the form of orthometric or normal heights. For this discussion the difference between orthometric and normal heights is not that significant and the term orthometric height will be used for convenience throughout this paper.

Before examining the possibilities for GPS heighting it is necessary to understand the limitations.

LIMITATIONS TO GPS HEIGHTING

In practice, GPS heighting typically involves measuring ellipsoidal heights with GPS, applying some form of geoid model and making any adjustment to fit the resulting orthometric heights to the existing vertical datum. Therefore, in examining the limitations of GPS heighting it is necessary to consider three broad areas:

- limitations of the GPS measurement

- limitations due to the available geoid model
- limitations due to vertical datum issues

Some or all of these issues vary in importance depending on the overall extent of the GPS survey in question. GPS surveys over national and continental scale are typically associated with datum level geodetic operations and need to consider many more issues than day to day surveys which extend over a few kilometres or less. In considering limitations to GPS heighting, this paper will attempt to highlight when the scale of the project is relevant.

Limitations of the GPS measurement

Obviously, the first limitation in GPS heighting is the quality of the GPS solutions used to obtain a height. Three broad categories of GPS observation types are possible:

- Point Positioning which is the stand alone navigation mode for which GPS was designed;
- Differential GPS (DGPS) which uses a differential correction approach but which is primarily based on pseudo range measurements and
- GPS Surveying using a differential approach but primarily based on measurement of the phase of the GPS signals.

While DGPS and even Point Positioning may be useful for producing heights in certain applications, the term GPS Heighting is typically taken to refer to the use of phase measurement techniques that can be grouped under the broad heading of GPS Surveying. This paper concentrates on heighting using these higher precision GPS Surveying techniques.

Within GPS Surveying, an overall consideration is whether the phase ambiguities have been resolved to integer values. Ambiguity resolution affects all three dimensions, not only height. For the measurement techniques known as *Rapid Static* and *Real Time Kinematic (RTK)*, which are used for shorter baselines, ambiguity resolution is a prerequisite and should be achieved for most day to day GPS surveying applications. It is important to realise that RTK uses the smallest possible amount of data and even the best algorithms sometimes resolve the ambiguities incorrectly. To avoid such errors, which can reach the metre level, it is important to build redundancy into a survey by, for example, occupying stations more than once.

Two aspects that can affect the overall quality of the baseline solution are errors in the ephemeris or in the starting coordinates used in the processing. The effect of these can reach several parts per million and apply to all three dimensions. Assuming that the broadcast ephemeris quality remains as high as in recent times, its effect will be minimal for most applications over short baselines. However, it should be noted that obtaining a WGS84 three-dimensional starting position of a reasonable quality (say +/- 10m or better) could be more problematic in some areas of the world.

Another error source to consider is multi-path. Reflective surfaces can mean that some of the signal reaching the antenna does not travel on a direct path from the satellite. The effects of multi-path can reach the decimetre level in three dimensions. Observation over time to allow the satellite geometry to change sufficiently enables the effect of multi-path to be reduced through averaging. However, with the short observation times typical of the Rapid Static and RTK techniques in common use, it is necessary to pay attention to this issue. While modern hardware and software designs include various ways of reducing the effects of multi-path, even over short observation times, it is important to choose stations so as to reduce the likelihood of multi-path and build redundancy into the survey to enable detection of any remaining effect.

Atmospheric delay is another issue to be considered. For a short baseline one can reasonably assume that the radio signals measured by both receivers pass through the same part of the atmosphere. However, as baseline length increases that assumption begins to break down and atmospheric effects need more consideration. Two components of the atmosphere are relevant, the ionosphere and the troposphere.

Problems due to the upper atmospheric layer known as the ionosphere can affect all three dimensions and become significant on lines longer than, say 20km. For such longer baselines, processing software is able to take advantage of the fact that the ionospheric effect is related to the frequency of the signal and dual frequency measurements can be used to remove most of the effect. The effect of the ionosphere is greater near the poles and the geomagnetic equator and varies over time in association with solar disturbance cycles. Therefore, it should be noted that for certain areas and certain times the ionospheric effect can be significant, even over short baselines.

The effect of the troposphere is particularly significant for height measurement. Unlike the ionosphere, tropospheric delay cannot be mitigated using dual frequency measurement. Furthermore, the GPS signal can be delayed due to both the dry and wet components of the troposphere. Most GPS processing includes models to account for the dry component of the troposphere. However, it is difficult to model the wet component given its greater variability and estimating the wet delay as part of the overall baseline estimation process is the best approach.

Over longer baselines (say 100km or more) it is typical for many hours of GPS data to be observed and the high level of redundant data allows for the tropospheric delay to be estimated at regular intervals through the data set (e.g. one delay each hour). For such long baselines, ignoring the tropospheric delay can cause a height error of several centimetres (see, for example, Dodson et al, 1996).

For many day to day surveys, the baselines are typically quite short and the effect of the troposphere is less significant. Also the data observation times are short meaning that less data is available to estimate the delay even if it were significant. Generally speaking, such surveys can simply use the software model for the dry component and the remaining effect will not be significant. However, it should also be noted that the tropospheric effect could be significant when there is a significant change in height between the ends of the baseline. For steep baselines, even when relatively short, there may be situations where longer observation time and estimating the delay may be warranted.

Tidal phenomena may be significant for GPS heighting in certain circumstances. These include the earth tide and the ocean tide's variable loading of the crust in and adjacent to the coastal zone. While not usually significant for day to day GPS surveying over short baselines, there can be significant differential effects for baselines of 100km and longer; amounting to centimetre level errors in height. Some software packages enable modeling of these tidal effects for those situations when they are significant.

The other major source of error for GPS heighting involves the antenna. The first and most obvious problem is that the height of the antenna above the survey mark must be correctly measured. Many RTK systems use a pole for the roving antenna to decrease occupation time compared to tripod usage. An advantage of this for heighting is that the fixed height of the pole minimises the possibility of incorrect antenna heights. When variable height tripods are used it is important to have a field routine for checking the height measurement at each station. Use of a slant height (to the outside of the ground plane) and comparison to the vertically measured height is a technique in common usage. Measurement in both metric and imperial units is another approach.

A less obvious antenna issue arises when various antenna types are mixed in the same survey. The problem with antenna mixing is that different antenna may have their effective antenna phase centre (also called the electrical centre) in different positions. This effect can be most significant in the height component and can reach values of several centimetres. The International GPS for Geodynamics Service (IGS) has had to address this issue to account for the many types of antennae used at its various permanent tracking stations. Antenna models are available and they can be applied in some software packages to mitigate the effects of antenna mixing (see Mader and MacKay, 1996).

Most GPS surveying applications use receivers and antennae that are all from the same manufacturer and this problem is minimised. However, there may be situations where a survey mixes antennae that have significantly different characteristics and surveyors need to be aware of this issue. One situation where this could arise is when using data from a base station run by another organisation. At present such a possibility is limited mainly to post processed applications but the increasing popularity of RTK along with adoption of standard RTK data formats is likely to lead to mixing of receiver and antenna types from different manufacturers, even in RTK surveying. For such situations antenna modeling will need to be addressed to ensure reliable height measurement.

Accuracy of GPS measurement for Height

Despite all of the issues outlined above, the *bottom line* for the practicing GPS surveyor is what can be achieved using typical Rapid Static and RTK approaches? A pragmatic approach to answering that question is to look at the accuracy claims of manufacturers. A quick scan of product brochures or information on web sites of a number of manufacturers led to the following:

- Leica state baseline rms values for their new SR530 for real time static of 5mm + 2ppm and real time stop & go and kinematic of 10mm + 2ppm. No differentiation is made between horizonatal and vertical accuracies.
- Ashtech produce a RTK system that uses both GPS and the Russian GLONASS systems and measures single frequency data from both systems. The stated vertical accuracy for the GG-RTK system is 1cm +1ppm at 1 sigma.
- Javad Positioning Systems make a general statement of 1mm + 1ppm for dual frequency and 2mm + 2ppm for single frequency.
- Trimble gives a quite comprehensive outline in the data sheet for their 4800 GPS Total Station product (summarised in the following Table). While not specifically stated, it would appear that these are 1 sigma values. Note that the term *fast static* used by Trimble and the term *rapid static* used in this paper are the same. Also note that for RTK the stated accuracy varies according to the update rate used (1Hz is a rate of 1 update per second and 5Hz equals 5 per second).

Mode	Accuracy	Latency
Static and Fast Static	5mm + 1ppm Horizontal	
	10mm + 1ppm Vertical	
Post processed kinematic	10mm + 2ppm Horizontal <10km	

	20mm + 1ppm Horizontal >10km	
	20mm + 1ppm Vertical	
RTK at 1 Hz	10mm + 2ppm Horizontal	0.4 sec
	20mm + 2ppm Vertical	
RTK at 5 Hz	30mm + 2ppm Horizontal	0.1 sec
	50mm + 2ppm Vertical	

If one accepts these figures then the vertical accuracy possible for the types of GPS surveying modes used in most day to day surveys are shown in the Tables below. The table shows values in millimetres for baseline lengths of 1, 5 and 10km. As well as 1 sigma, the table also shows 3 sigma values to give an indication of the worst results that may be expected.

Mode	mm	+ ppm	Error in mm (1 sigma)			Error in mm (3 sigma)		
			1km	5km	10km	1km	5km	10km
Fast Static	10	1	11	15	20	33	45	60
Kinematic	20	1	21	25	30	63	75	90
RTK 1 Hz	20	2	22	30	40	66	90	120
RTK 5 Hz	50	2	52	60	70	156	180	210

It must be remembered that the issues and accuracy values outlined in this section are only for the GPS measurements. For day to day surveys over project areas less than 10km in extent, the GPS measurement is often the least significant part of the GPS Heighting problem.

Limitations due to Geoid Model

GPS surveying measures differences in ellipsoidal heights (h in Figure 1) and to produce physically meaningful heights such as orthometric heights (H) there is a need for a sufficiently precise model of the separation between the geoid and the ellipsoid; the geoid height (N). Also, GPS surveying can measure that ellipsoidal height difference over large distances very efficiently. These two points can highlight problems in the existing geoid model or vertical datum, or both.

In some areas of the world the only available geoid model is a global geopotential model (GGM). A GGM is typically computed as a series of spherical harmonic expansions to a maximum degree and order. Many recent GGM use an expansion to degree and order 360. That means they are able to resolve features in the geoid with a wavelength down to half a degree (nominally 55km). With such resolution, even state of the art models such as the Earth Geopotential Model 1996 (EGM96 - Lemoine et al, 1996) are limited to absolute accuracy at the metre level and relative accuracy at the several decimetre level.

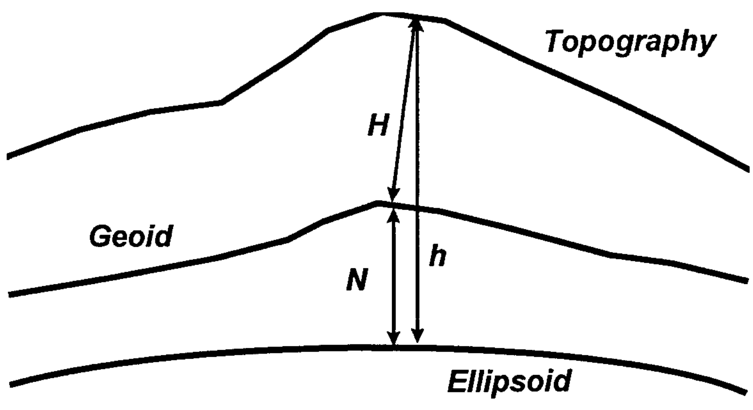


Figure 1 Geoid Height

In many regions of the world, it is desirable to improve upon the accuracy possible using only the GGM by computing a local geoid model. The model typically takes the form of a grid of geoid heights to be interpolated by users as required. Local geoid models are derived in two parts. The long wavelength component comes from the best available GGM while the short wavelength component is computed using locally observed gravity data. The short wavelength component of the geoid can be computed using several techniques, including doing the integration directly (with quadratures or rings), least squares collocation or fast fourier transform. Sunkel (1996) gives a summary of recent geoid developments in various countries. Obviously, geoid height accuracy is strongly influenced by how well the gravity data used in the computation represents the actual gravity field and recent improvements include:

- use of satellite altimetry data for increasing gravity data density offshore,
- use of digital elevation model data (DEM) to account for terrain effects on the gravity field,
- use of DEM data to reflect high frequency gravity field variations and improve the gridding or interpolation of the raw gravity data.

The limiting factors for the inherent accuracy of a geoid model are the amount of variability in the gravity field and in the terrain. A geoid model will typically be least accurate in areas with rugged terrain and highly variable underlying geology.

However in applying a geoid model, its inherent accuracy is not the only limitation. How well the geoid model can be used in conjunction with the existing vertical datum also requires consideration.

Limitations due to Vertical Datum

The definition of the vertical datum in many areas of the world has often been localized with realization through published orthometric or normal heights from local adjustment of networks of spirit level, barometric and trigonometric heighting observations. Sometimes multiple vertical datum developed with each propagating from a single point such as a tide gauge. Some regional or national vertical datum have been developed by constraining the adjustment of the leveling and heighting observations to the height of mean sea level at one or more tide gauges.

In the realisation of the Australia Height Datum (AHD) for example, the adjustment of 97,320km of leveling was constrained to mean sea level at 30 tide gauges. Oceanographic influences along with possible errors in the leveling mean that the surface formed by the base of AHD could be distorted significantly from the purely geopotential surface of the geoid (see for example Featherstone, 1998). In such cases a possible solution is to address the problem at the level of the vertical datum by developing an appropriate model of the distortion and adding it to the geoid model.

Whether or not such a distortion process has been incorporated into the geoid model, it is prudent for the GPS survey to verify the agreement between the geoid model and the vertical datum in a given project area. Occupying at least three existing stations with vertical datum height values as part of the GPS survey can do this. If there is some residual local distortion it can be removed as part of the process of adjusting the survey.

Another issue for modern vertical datum is the need to refine our information management in relation to heights. Prior to GPS, most geodetic databases typically stored only an orthometric (or normal) height for a station because that was the typical form for height observations. With GPS, vertical datum will increasingly be made up of stations at which heights are observed directly as orthometric heights and others at which ellipsoidal heights are observed. There is also a need to treat a geoid height like any other observation type in that it is of a particular quality at a particular time. Without careful management of these different data types any problems can blur into one another and make maintenance and improvement of the vertical datum difficult.

POSSIBILITIES FOR GPS HEIGHTING

The rest of this paper will examine the possibilities for GPS heighting, focusing on three application areas currently generating interest for the practicing surveyor and covering the full range of achievable accuracy.

Possibilities for Deformation Monitoring

While many applications for GPS surveying need to produce orthometric or normal heights there are some applications where ellipsoidal heights alone are useful. One such application is vertical deformation monitoring where the most important issue is to quantify a change in height over time and whether any change is relative to the geoid or ellipsoid is not particularly relevant.

Using GPS for deformation monitoring brings the normal advantages of GPS surveying such as no requirement for inter-visibility between stations and the ability to span large distances with high precision. Also, deformation applications require many repeated observations over time and GPS is well suited to automated survey processes that can significantly reduce cost.

Given the limitations outlined earlier in this paper, issues to consider in designing a GPS deformation survey include ambiguity resolution, quality of ephemeris and starting coordinates, multi-path, troposphere, tidal phenomena, antenna modeling and antenna height measurement.

It is possible to account for those issues that are significant over short baselines using commercially available equipment and software and obtain centimetre accuracy heights. However, short baselines are not always practical in deformation surveys where it is necessary to measure to stable fixed stations well outside the deformation zone. Therefore, the accuracy possible from Rapid Static and RTK will not satisfy many deformation monitoring requirements and longer observation periods with data processing in static mode may be required. Centimetre accuracy is possible even over baselines longer than 100km with occupation times of several days and using specialised data processing.

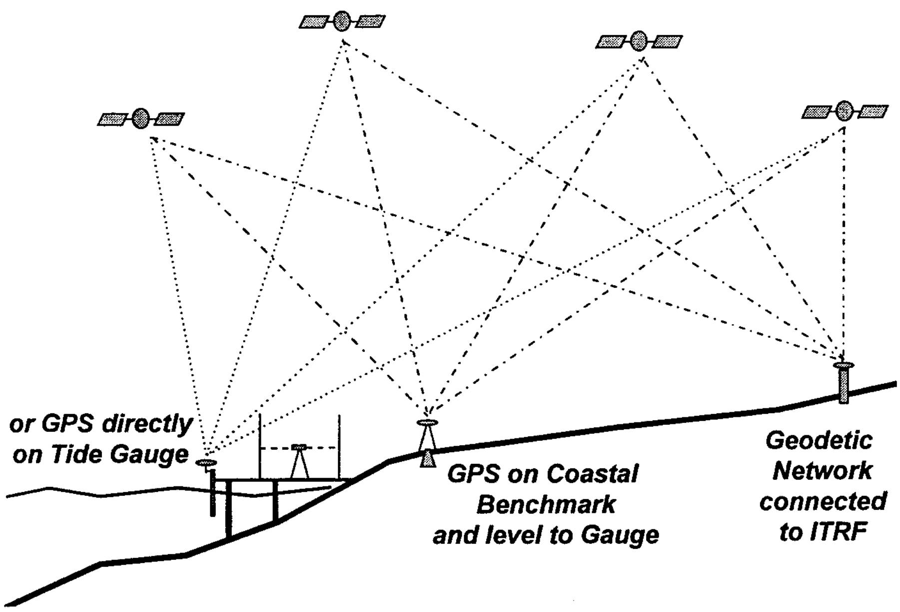


Figure 2 Monitoring Vertical Deformation of a Tide Gauge using GPS

One of the most demanding current applications for GPS heighting is in the centimetre or better accuracy required for monitoring the vertical deformation of high precision tide gauges being used to measure possible sea level rise due to global warming. Early thinking on this problem saw the need for coastal and inland arrays of marks tied to each other and to the global fiducial network by GPS and with leveling from the coastal array to the tide gauge. GPS observations took several days repeated at regular epochs. However, there were problems with isolating the deformation between the arrays and the gauge as well as accuracy limitations with episodic GPS. The state of the art approach recommends use of a permanent tracker mounted directly on the gauge as the best means for obtaining the required accuracy. Both approaches are illustrated in Figure 2. For a comprehensive description of the state of the art in this demanding application see Neilan et al (1997).

Application in Real Time GPS Surveying

The high productivity of RTK with its requirement for only a few epochs of data recording, means that many surveyors would like to apply the technique to the height measurements required in engineering applications. However, some caution is required and it is necessary to consider how RTK errors increase with baseline length using a particular equipment configuration. For initial route finding or contour and detail surveys requiring accuracy at the several centimetre level, RTK may well be suited. For many engineering surveys however, the heighting accuracy required is at the one centimetre level and RTK may be suitable but should be restricted to baselines shorter than a kilometre. For projects extending more than a kilometre, several RTK base stations may be required. One way of improving the accuracy of RTK over longer baselines is to observe for longer periods at a point. This approach can be thought of as *real time rapid static*, and brings improved accuracy while maintaining the logistical advantages of real time results.

Whether range is extended by multiple base stations or by longer observation time, for projects extending over many kilometres the issues of geoid and local vertical datum distortion will need to be considered. Many systems allow incorporation of geoid models into the real time data processing. However, that assumes any local distortions are incorporated in the geoid model to an accuracy sufficient for the project. Some manufacturers provide an additional feature referred to as field calibration whereby any remaining three dimensional distortion can be determined in the field using real time procedures to occupy existing control stations. Obviously, field calibration is subject to any error in the GPS observations over the baseline lengths required to connect to control.

In practice then, when contemplating RTK for centimetre level GPS heighting, it is necessary to develop field procedures that address the many possible error sources. Such procedures also need to be flexible and assessed on a case by case basis.

Application in Machine Monitoring and Guidance

Many GPS manufacturers are promoting the use of RTK techniques to automate machine monitoring and guidance for application in agricultural, earth moving and construction equipment. It must be realised that such procedures are subject to the same error sources as any other RTK surveying technique and, as outlined in the tables earlier, those errors vary according to baseline length and the update rate being used (e.g. 1Hz vs 5Hz).

It is important to realise that a major reason for the high precision of surveys supporting engineering applications is to give a margin for error. Normally, such margins are desirable given the error which can propagate through the several steps and sets of measurements which may be required over the life of a construction process. If, for some applications, the machinery is positioned directly from the local control in a single step that margin of error may not need to be so stringent. However, other applications will require a high accuracy that is at the limits achievable with GPS in real time. In road construction for example, real time machine guidance may be feasible for earth moving but not for pavement laying equipment.

The major concern is a tendency toward black box thinking and someone who is not aware of all the issues could misuse these highly automated systems. These include the GPS measurement, geoid and datum issues discussed in this paper that can affect the final achievable accuracy for a machine guidance system. It is to be hoped that surveyors who are aware of these issues will continue to be involved in connecting and assessing the local control, establishing the base stations required and monitoring overall quality.

CONCLUSION

This paper examined in detail the limitations of GPS heighting, including GPS measurement, geoid and datum issues. GPS measurement accuracy can be limited by ambiguity resolution, quality of ephemeris and starting coordinates, multi-path, troposphere, tidal phenomena, antenna modeling and antenna height measurement. The effect of availability and quality of geoid models was also addressed. The characteristics of the vertical datum and its suitability for combination with the geoid model and ellipsoidal heights from GPS were shown to also be significant limitations to the achievable height accuracy. It was pointed out throughout the paper that all these limitations vary in significance from one situation to another.

The paper then examined the possibilities for GPS heighting by looking at the three applications of deformation monitoring, real time GPS surveying and machine monitoring and guidance. These applications cover the complete range of achievable accuracy. The significance of the limitations of GPS heighting also varies between these applications. The paper closed with a caution against too automated an approach to the use of real time GPS techniques in machine systems especially where high accuracy heights are required.

REFERENCES

Dodson, A.H., P.J. Shardlow, L.C.M. Hubbard, G. Elgered and P.O.J. Jarlemark, 1996, Wet Tropospheric Effects on Relative GPS Height Determination, Journal of Geodesy, Vol. 70, pp 188-202.

Featherstone, W.E., 1998, Do we need a Gravimetric Geoid or a model of the Australina Height Datum to Transform GPS Heights in Australia, The Australian Surveyor, Vol. 43, No. 4, December, pp 273-280.

Lemoine, F.G., D.E., Smith, R. Smith, L. Kunz, N. Pavlis, M. Klosko, D.S. Chinn, M.H. Torrence, R.G. Williamson, C.M. Cox, K.E. Rachlin, Y.M. Wang, E.C. Pavlis, S.C. Kenyon, R. Salman, R. Trimmer, R.H. Rapp and R.S. Nerem, 1996, The Development of the NASA GSFC and DMA Joint Geopotential Model, Proceedings of the International Symposium on Gravity, Geoid and Marine Geodesy, Tokyo, Japan, September 30 - October 5.

(Internet copy available from: http://cddis.gsfc.nasa.gov/926/egm96/egm96.html)

Mader, G.L. and J.R. MacKay, 1996, Calibration of GPS Antennas, IGS Workshop Proceedings, Silver Spring, Md. March 19-21, 1996. available from Jet Propulsion Laboratory, Pasadena, California. JPL Publication 96-23 10/96. pp.81-105.

Neilan, R.E., P.A. Van Scoy and P.L Woodworth (Editors), 1997, Workshop on Methods for Monitoring Sea Level, Proceedings of the Workshop, Pasadena, California, March 17 – 18, 1997. available from Jet Propulsion Laboratory, Pasadena, California. JPL Publication 97-17 3/98.

Sunkel, H., 1996, International Geoid Commission Activity Report 1991 - 1995, International Geoid Commission, Commission XII of Section III of the International Association of Geodesy.

(Internet copy available from: http://www.ngs.noaa.gov/~dennis/geoid.html)

State of the art and present developments of a general approach for GPS-based height determination

Prof. Dr.-Ing. Reiner Jäger

Fachhochschule Karlsruhe – University of Technology

Moltkestraße 30, D-76133 Karlsruhe, Fax: 0049 721 9252591

Email/Internet: reiner.jaeger@fh-karlsruhe.de; www.fh-karlsruhe.de/fbg

ABSTRACT

The contribution treats a sophisticated concept in the area of GPS-based height determination with components being appropriate to branch out into different classes of standard approaches, depending on the kind of data sources as well as on the principal target. So, besides a GPS-based height determination, also a height system transformation may be set up. Basically any kind of height data, namely geoid models N (e.g. EGG97), heights H, levelling ΔH, GPS heights h and GPS baselines Δh may be combined. Partly a Finite Element Model (FEM) is set up for the representation of height reference surfaces (HRS). This FEM is parametrized by bivariate polynomials sets, and continuity conditions guarantee a continuous transition of the FEM surface along the edges of neighbouring meshes in any area size. In opposite to digital terrain models, the nodes of the FEM mesh may differ from the position of the data used for the FEM determination.

The first part of the contribution treats the class of already practical working standard approaches, developed to transform in a statistically controlled way ellipsoidal GPS heights h into heights H of a standard height system. First the role of use and the kind of a datum and systematics adaption of geoid models N in a GPS height integration are discussed. The „geoid refinement approach“ standard means that a datum adapted geoid model N is used as direct observation, while the FEM serves as additional overlay to improve the final representation of the HRS. The special case of the „pure FEM approach“ arises, if the FEM representation of HRS is computed purely by geometric observations H, ΔH, h, Δh. The „pure geoid approach“ means, that only a datum adapted geoid model N is used in a GPS height integration. The three approaches provide a flexible area of models implemented in the software HEIDI2. Different pilot projects in several parts of Europe finished successfully, and the height integration concept is meanwhile used as a standard in some state survey agencies. The experience shows that a high precision level for a GPS based height determination up to a 5 mm level in rather large areas is achieved, e.g. using the EGG97.

The second part and class of approaches treats the application of the FEM component for the purpose of height system transformation (e.g. conversion of NN-heights to normal heights).

The third part of the presentation and class of approaches considers the so called general approach, where the HRS is completely established by a FEM, using different datum adapted geoid models N, terrestrial height information H and ellipsoidal GPS heights h as data sources. The result of the computation and "geoid mapping" respectively, leads to a Digital FEM Height Reference Surface (DFHRS). The DFHRS may be set up as data base for a datum free direct GPS-based online

heighting in DGPS networks. First results of a pilot project in the German SAPOS network are reported.

1. INTRODUCTION

The transformation of a geocentric cartesian position (x,y,z) determined from DGPS provides the plane position represented by the geographical latitude and longitude (B,L) and the ellipsoidal height h, all referring to the datum of the respective reference station(s) used in DGPS. Both (B,L) and h depend on the metric and shape of the reference ellipsoid (a=main axis, f=flattening) used in the computation of (B,L,h). In general the GRS80 or the WGS84 ellipsoid are used in the context with GPS. The transition of GPS results $(B,L,h)_1$, in the following described as system 1 to a set of national network coordinates $(B,L,h)_2$, described as system 2, is to be performed by a threedimensional similarity transformation. There three translations (u,v,w), three rotations $(\varepsilon_x,\varepsilon_y,\varepsilon_z)$ and one scale difference Δm between both datum systems have to be taken into account. Using a taylor series expansion with linearization point $(B,L,h)_1$ and assuming small rotations we may write the datum transition from system 1 to system 2 on splitting the threedimensional problem equivalently into the plane and the height component in the following way [9],[5]:

Plane components (1), (2) of the threedimensional datum transition

$$\begin{aligned}
B_2 &= B_1 + \partial B_1(d) = B_1 + \partial B_1(u,v,w,\varepsilon_x,\varepsilon_y,\varepsilon_z,\Delta m,\Delta a,\Delta f)\\
&= B_1 + \left[\frac{-\cos(L)\cdot\sin(B)}{M+h}\right]_1\cdot u + \left[\frac{-\sin(L)\cdot\sin(B)}{M+h}\right]_1\cdot v + \left[\frac{\cos(B)}{M+h}\right]_1\cdot w +\\
&\left[\sin(L)\cdot\frac{h+N\cdot W^2}{M+h}\right]_1\cdot\varepsilon_x + \left[-\cos(L)\frac{h+N\cdot W^2}{M+h}\right]_1\cdot\varepsilon_y + [0]\cdot\varepsilon_z +\\
&\left[\frac{-e^2\cdot N\cdot\cos(B)\cdot\sin(B)}{M+h}\right]_1\cdot\Delta m + \left[\frac{N\cdot e^2\cdot\cos(B)\cdot\sin(B)}{a\cdot(M+h)}\right]_1\cdot\Delta a + \left[\frac{M\cdot\sin(B)\cdot\cos(B)\cdot(W^2+1)}{(M+h)\cdot(1-f)}\right]_1\cdot\Delta f
\end{aligned}\tag{1}$$

$$\begin{aligned}
L_2 &= L_1 + \partial L_1(d) = L_1 + \partial L_1(u,v,w,\varepsilon_x,\varepsilon_y,\varepsilon_z,\Delta m,\Delta a,\Delta f)\\
&= L_1 + \left[\frac{-\sin(L)}{(N+h)\cdot\cos(B)}\right]_1\cdot u + \left[\frac{\cos(L)}{(N+h)\cdot\cos(B)}\right]_1\cdot v + [0]\cdot w +\\
&\left[\frac{-(h+(1-e^2)\cdot N}{(N+h)\cdot\cos(B)}\cdot\cos(L)\cdot\sin(B)\right]_1\cdot\varepsilon_x + \left[\frac{-(h+(1-e^2)\cdot N)}{(N+h)\cdot\cos(B)}\cdot\sin(L)\cdot\sin(B)\right]_1\cdot\varepsilon_y + [1]\cdot\varepsilon_z +\\
&[0]\cdot\Delta m + [0]\cdot\Delta a + [0]\cdot\Delta f
\end{aligned}\tag{2}$$

Ellipsoidal height component (3) of a threedimensional datum transition

$$\begin{aligned}
h_2 &= h_1 + \partial h_1(d) = h_1 + \partial h_1(u,v,w,\varepsilon_x,\varepsilon_y,\varepsilon_z,\Delta m,\Delta a,\Delta f)\\
&= h_1 + [\cos(L)\cdot\cos(B)]_1\cdot u + [\cos(B)\cdot\sin(L)]_1\cdot v + [\sin(B)]_1\cdot w +\\
&\left[e^2\cdot N\cdot\sin(B)\cdot\cos(B)\cdot\sin(L)\right]_1\cdot\varepsilon_x + \left[-e^2\cdot N\cdot\sin(B)\cdot\cos(B)\cdot\cos(L)\right]_1\cdot\varepsilon_y + [0]\cdot\varepsilon_z +\\
&\left[h+W^2\cdot N\right]_1\cdot\Delta m + \left[\frac{-N\cdot W^2}{a}\right]_1\cdot\Delta a + \left[\frac{W^2\cdot M\cdot\sin^2(B)}{1-f}\right]_1\cdot\Delta f
\end{aligned}\tag{3}$$

N(B) and M(B) are introduced as the latitude dependent quantities of the so called normal and the meridian radius of curvature respectively. For W(B) and e^2 we have W(B)=a/N(B) and $e^2=2f-f^2$. In general the parameter changes Δa and Δf are known, and the respective quantities are introduced as deterministic corrections. Respective corrections due to Δa and Δf are therfore not mentioned in the following.

The integration of GPS-results may be carried out with respect to (1), (2) by using only plane coordinates $(B,L)_2$ in the domain of the identical points with respect to the determination of the datum parameter set $\mathbf{d}=(u,v,w,\varepsilon_x,\varepsilon_y,\varepsilon_z,\Delta m)$. This has the advantage, that in addition to the ellipsoidal GPS height h_1 no further height information (ellipsoidal height h_2, geoid N_G and standard heights H_2) is necessary from the national network system 2 [14].

If we take vice versa a look to (3) we need in the context with $H_2=h_2-N_G$ the introduction of a so called „geoid" model N_G, as we generally dispose only on the standard heights H_2 referring to the physically defined height reference system HRS (fig.1). But in practice we have to consider, that a geoid model N_G taken from a geoid data base [3] has – being another surface in space – an own more or less small but unknwon datum $\mathbf{d}_G=(u,v,w,\varepsilon_x,\varepsilon_y,\Delta m)_G$.

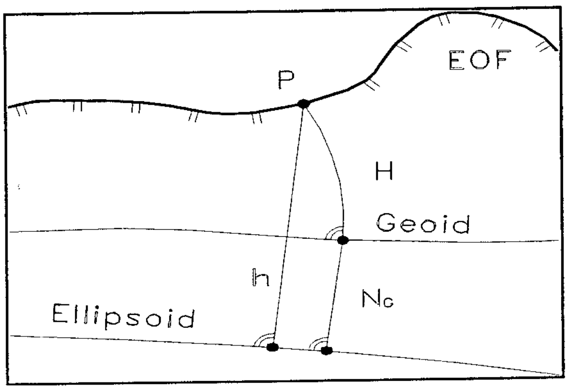


Fig. 1: Ellipsoidal GPS height h, height reference surface HRS or briefly „geoid" and earth surface EOF at a point P(B,L)

Instead of the „raw" ideal and datum effects neglecting standard formula

$$H = h - N_G \tag{4a}$$

we arrive starting from (3) in real life practice at the complete formula reading

$$H_2 = h_1 + \partial h_1(u,v,w,\varepsilon_x,\varepsilon_y,\Delta\Delta m - (N_G + \partial N(u,v,w,\varepsilon_x,\varepsilon_y,\Delta\Delta m_G) = h_1 + \partial h_1(d) - (N_G + \partial N(d_G)) \ . \tag{4b}$$

From (1), (2) and (4b) follows that a threedimensional GPS-integration based on standard heights H_2 and a geoid model N_G has to consider in total 13 parameters within $\mathbf{d}$ and $\mathbf{d}_G$. Assuming that the data base geoid values N_G are referred to $(B,L)_1$, we see directly that the parameters within the different sets $\mathbf{d}$ and $\mathbf{d}_G$ separate due to the variation of the heights h_1 and N_G within the coefficients belonging to $\mathbf{d}$ and $\mathbf{d}_G$ respectively (1),(2),(3). The standard approach of a threedimensional transformation with only one common set of seven parameters $\mathbf{d}$ is therefore not free of systematic errors.

If we however restrict a GPS-integration to the isolated GPS height integration problem, meaning the transformation of GPS heights h_1 to standard heights H_2, we derive from (4b) :

$$\begin{aligned}H_2 = {} & h_1 - N_G + [\cos(L)\cdot\cos(B)]_1 (u-u_G) + [\cos(B)\cdot\sin(L)]_1 \cdot (v-v_G) + [\sin(B)]_1 \cdot (w-w_G) + \\ & [e^2\cdot N\cdot\sin(B)\cdot\cos(B)\cdot\sin(L)]_1 \cdot (\varepsilon_x-\varepsilon_{x,G}) + [-e^2\cdot N\cdot\sin(B)\cdot\cos(B)\cdot\cos(L)]_1 \cdot(\varepsilon_y-\varepsilon_{y,G}) + \\ & [h_1+W^2\cdot N]\cdot\Delta m - [N_G+W^2\cdot N]\cdot\Delta m_G \\ = {} & h_1 - N_G + \partial_{h_1,G}(u',v',w',\varepsilon_x',\varepsilon_y') + [h_1+W^2\cdot N]\cdot\Delta m - [N_G+W^2\cdot N]\cdot\Delta m_G\end{aligned} \tag{5}$$

We recognize from (5), that due to some common coefficients one set of parameter-differences **d**‘=(u‘=u-u_G; v‘=v-v_G; w‘=w-w_G; ε_x‘=ε_x-$\varepsilon_{x,G}$; ε_y‘=ε_y-$\varepsilon_{y,G}$) may be introduced instead of two different sets in a geoid-model based GPS-height integration (8a,b,c). Separate parameters have to be kept only for the scale parameters Δm and Δm_G.

2. STANDARDS OF GPS HEIGHT INTEGRATION

With the trend of replacing old national datum systems in favour of ITRF-related datum systems and respective DGPS reference station systems (like e.g. SAPOS in Germany), the datum problem for the plane component (B,L) in GPS-based positioning will vanish by and by. But for the reason of a physically different height reference surface HRS for the standard heights H (fig. 1) defined by geopotential numbers, the problem of a transition of ellipsoidal GPS-heights h_1 to the standard heights H_2 referring to a HRS – or briefly spoken „geoid" (a true geoid for an orthometric height system, a quasi-geoid for a normal height system etc.) – will remain .

Different approaches have been developed from the „Karlsruhe working group" [7] up to now. The advantages of the above splitting into the plane (1),(2) and height component (5) led to a powerful and flexible set of GPS-integration approaches [4], [5], [6], [7] which will be presented in the following chapters.

2.1 Finite Element Representation (NFEM) of the Height Reference Surface (HRS)

A powerful and central tool within the GPS height integration approaches of the „Karslruhe working group" consists in the representation of the „geoid" N_G or better the height reference surface HRS (fig. 1) as a finite element surface. In this way the finite element model NFEM(**p**,B,L) of HRS represents in the ideal sense h=H+N_G - datum free and independent of the type of the standard height system H - the height N_G of the HRS over the ellipsoid as a function of the plane position (B,L) and the parameter vector **p**. As described in details in [4] and [5] the finite element representation NFEM(**p**,y,x) of the HRS is performed over a square grid with nodal points. The plane position (B,L) is replaced by metric coordinates (y(B,L)="Eastern" and x(B,L)="Northern") such as UTM or Gauß-Krüger coordinates to be computed from (B,L).

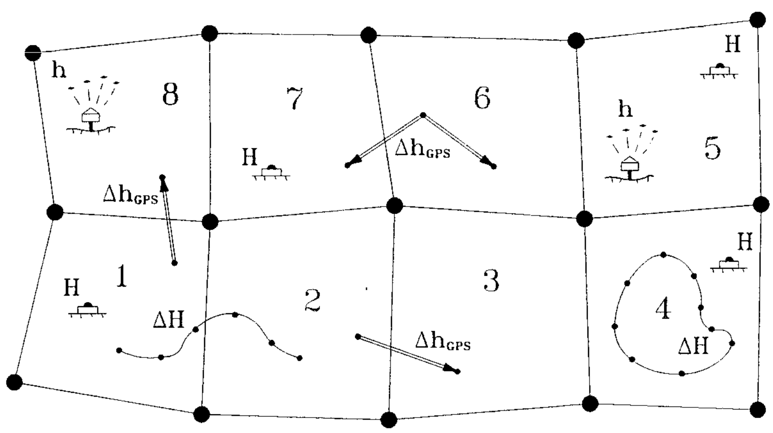


Fig. 2: Nodal points • and edges of a FEM-meshing and geodetic measurements (h,H,ΔH,Δh) used (eventually together with a geoid-model N_G as additional data source) for the determination of the NFEM(**p**,y,x) model of the HRS.

The mesh size and shape (fig.2) and at the same time the approximation quality of NFEM(**p**,y,x) with respect to the true HRS (fig. 1) may be chosen arbitrary. A special advantage and characteristic of the NFEM(**p**,y,x) representation consists last but not least in the fact, that the nodal points (•, fig. 2) of the FEM grid are totally independent of the geodetic network and data points (h,H,ΔH,Δh and geoid data N_G), which are used for the determination of the parameter vector **p** of NFEM(**p**,y,x). Without loss of generality we choose in the following bivariate polynomials of degree l as basic function to carry the surface NFEM(**p**,y,x) within the different meshes. The corresponding polynomial coefficients are introduced as $a_{ij,k}$, so that the parameter vector **p** consists of all coefficiental sets $a_{ij,k}$ over m meshes (i=0,l; j=0,l and k=1,m).

Dependent on the plane position (y,x) the local „geoid height" N_G is to be received from the finite

$$\mathrm{NFEM}(p) =: \begin{Bmatrix} N(p_k) = \sum_{i=0}^{l} \sum_{j=0}^{l-i} a_{ij,k} \cdot y^i \cdot x^j \\ C_{0,1,2}(p_m, p_n) \end{Bmatrix} \qquad (6)$$

element representation NFEM(**p**,y,x) by first identifying the corresponding k-th mesh according to the position (y,x) by means of the vector of nodal point positions. Then N_G is to be evaluated from NFEM(**p**,y,x) by the local polynomial with coefficients $a_{ij,k}$ at the plane position (y,x).

To imply a continuous surface NFEM(**p**,y,x) one set of continuity conditions of different type $C_{0,1,2}$ has to be set up at the computation of NFEM(**p**,y,x) for each couple of neighbouring meshes m and n. The continuity type C_0 implies the same functional values along each common mesh border. The continuity type C_1 implies the same tangential planes and the continuity type C_2 the same curvature along the common borders of the HRS model NFEM(**p**,y,x). The continuity conditions occur as additional condition equations related to the polynomial sets of the coefficients $a_{ij,m}$ and $a_{ij,n}$ of each couple of neighbouring meshes m and n. The number and the mathematical contents of these condition equations depend on the polynomial degree l as well as on the continuity equation type [5].

The standard in the application of NFEM(**p**,y,x) in GPS height integration research and projects up to now was to use C_0 conditions and a degree of l=1 for a small mesh sizes up to 10 km, and degrees l=2,3 for larger mesh sizes. For the case l=3 and C_0-continuity for NFEM(**p**,y,x) we have to introduce for each neighbouring mesh border the following condition equations [5] :

$$da_{30}\,dx^3 + da_{21}\,dx^2dy + da_{12}\,dxdy^2 + da_{03}dy^3 = 0 \qquad (7a)$$

$$da_{30}\Delta^3 + da_{20}\Delta^2dy + da_{10}\Delta dy^2 + da_{00}dy^3 = 0 \qquad (7b)$$

$$da_{10}dxdy^2 + da_{01}dy^3 + 2da_{20}\Delta dxdy + da_{11}\Delta dy^2 + 3da_{30}\Delta^2dx + da_{12}\Delta^2dy = 0 \qquad (7c)$$

$$da_{20}dx^2dy + da_{11}dxdy^2 + da_{02}dy^3 + 3da_{30}\Delta dx^2 + 2da_{21}\,\Delta dxdy + da_{12}\Delta dy^2 = 0 \qquad (7d)$$

With respect to the known nodal points $A(y_A,x_A)$ and $E(y_E,x_E)$ of the mesh grid (fig. 2) we introduced the abbreviations $dx=x_E-x_A$ and $dy=y_E-y_A$ as well as $\Delta=dy\cdot x_E-dx\cdot y_A$ and $da_{ij}=a_{ij,m}-a_{ij,n}$ in (7a,b,c,d).

2.2 Standard approaches of GPS height integration

Starting with formula (5) we immediately arrive at the so called „pure geoid approach". This approach is to be applied in a GPS height integration, as soon as good geoid information $N_G(B,L)$ is available. The parameters fo the datum part $\partial_{h,G}(\mathbf{d}',\Delta m_G)$ have to be estimated. With some simplification in the scale term[1] for Δm, the „pure geoid approach" reads in the corresponding system of observation equations as follows:

$$H \quad +v = m\cdot H + N_G \tag{8a}$$

$$N_G(B,L) \ +v = \qquad N_G + \partial_{h,G}(\mathbf{d}', \Delta m_G) \tag{8b}$$

$$H \quad +v = \quad H \tag{8c}$$

GPS heights h, a geoid model N_G (B,L) and terrestrial heights H may be used as obervations. Of course the formulas are easy to extend to levelling ΔH and GPS height baselines Δh, which are also both included in all subsequent approaches. Apart from the datum part $\partial_{h,G}(\mathbf{d}',\Delta m_G)$ the geoid model N_G (B,L) is treated as so called ‚direct observation'. The datum part $\partial_{h,G}(\mathbf{d}',\Delta m_G)$ may also model and remove some systematics [4], [5], [6], [7].

In polarity to (8a,b,c) and above all in the case, that no geoid-information exists, we may derive the HRS completely from the observations ($h,H,\Delta H,\Delta h$) as finite element representation NFEM(**p**,y,x) of the HRS as given in (6). This approach is called the „pure finite element approach". It reads:

$$h + v = m\cdot H + NFEM(\mathbf{p},y,x) \tag{9a}$$

$$H + v = \quad H \tag{9b}$$

The powerful synergy of both above approaches finally leads to the so called „geoid-refinement approach". It is used for the case that the available geoid information N_G (B,L) is to be refined by a finite element model NFEM(**p**,y,x), which is acting as additional overlay to improve the geoid model (fig. 3). The geoid-refinement approach reads:

$$h \quad +v = m\cdot H + N_G \tag{10a}$$

$$N_G(B,L) \ +v = \qquad N_G + \partial_{h,G}(\mathbf{d}', \Delta m_G) + NFEM(\mathbf{p},y,x) \tag{10b}$$

$$H \quad +v = \quad H \tag{10c}$$

All above GPS-height integration approaches are implemented as standard models in the software package HEIDI2 ©Dinter/Illner/Jäger. The approaches are described in different papers and were proved in different projects [2],[4],[5],[6],[7],[8],[10],[11],[12].

[1] The scale term following (3) looks like the expression for m in (12a).

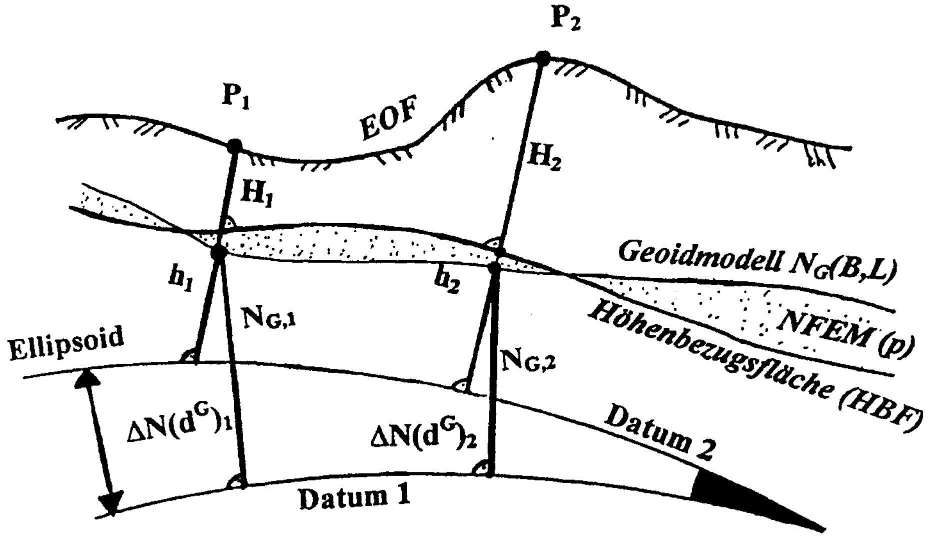


Fig. 3: Geoid-refinement approach as a synergetic combination of geoid information $N_G(B,L)$ submitted to a datum change (datum 1 -> datum 2) and the finite element model NFEM(**p**,y,x). NFEM(**p**,y,x) is introduced to model remaining systematics (dotted) between the introduced geoid model $N_G(B,L)$ and the true height reference surface HRS (="Höhenbezugsfläche (HBF)").

2.3 Example of a GPS height integration performed with the software HEIDI2

The following example of a GPS height integration treats the use of the commercially available EGG97 geoid model [3] for an integration of GPS heights h into the normal height system H of the height network of Tallinn, Estonia. The network has an extension of 40 km by 25 km. The computations were done by the author in the frame of a TEMPUS project between the Tallinn Technical University, the University of Technology Karlsruhe and other European universities. The given 23 ellipsoidal GPS-heights h in the EST92 datum were introduced with a quality of 3 mm, as proved before in a free adjustment of the respective GPS height baselines. The given normal heights were introduced with a quality of 3mm, and the EGG97 observations $N_G(B,L)$ with a precision of 5 mm. The different versions of the GPS height integration were computed on the base of the pure geoid approach (8a,b,c) with the software HEIDI2.

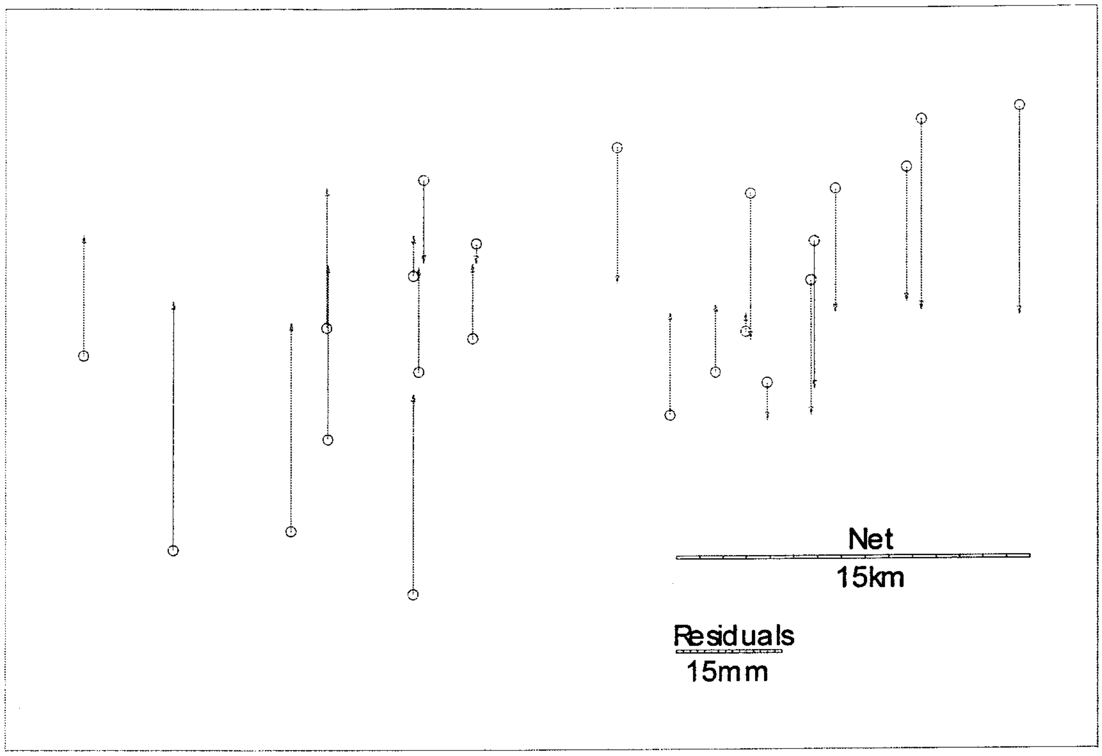


Fig. 4: GPS height integration for the Tallinn network by the pure geoid approach without taking a necessary datum-transition $\partial_{h,G}$ (**d**‘,Δm_G) part for h and N_G into account: The residuals in the known control points - treated as new points - show the systematics of a datum tilt up to ± 3.5 cm.

The result of a first version, where - in sense of the unrealistic ideal (4a) - no datum transition $\partial_{h,G}$(**d**‘,Δm_G) for h and N_G was introduced, is presented in fig. 4. Each known point was once computed as a „new point“ determined by „GPS and geoid“. The residuals in the identical points H are in the range of up to ± 3.5 cm and show the typical effect of a neglected datum tilt in this high range.

The fig. 5 shows the next set of computations in the pure geoid approach (8a,b,c) used as computation model for a GPS height integration. Now a datum transition $\partial_{h,G}$(**d**‘,Δm_G) was taken into account. The residuals in the known control remain less than 1 cm, the mean residual is in the range of ± 4 mm. In this version of a GPS-based height integration all observation components are consistent with their assumed a priori precision and no gross errors occur in all runs. An additional geoid refinement might be computed by the geoid refinement approach (10a,b,c).

For further examples of GPS height integration in medium and in large scale networks and also due to the other above approaches like the geoid refinement approach (10a,b,c) and the pure FEM approach (9a,b) it is referred to [4],[5],[6],[7],[10],[11],[12].

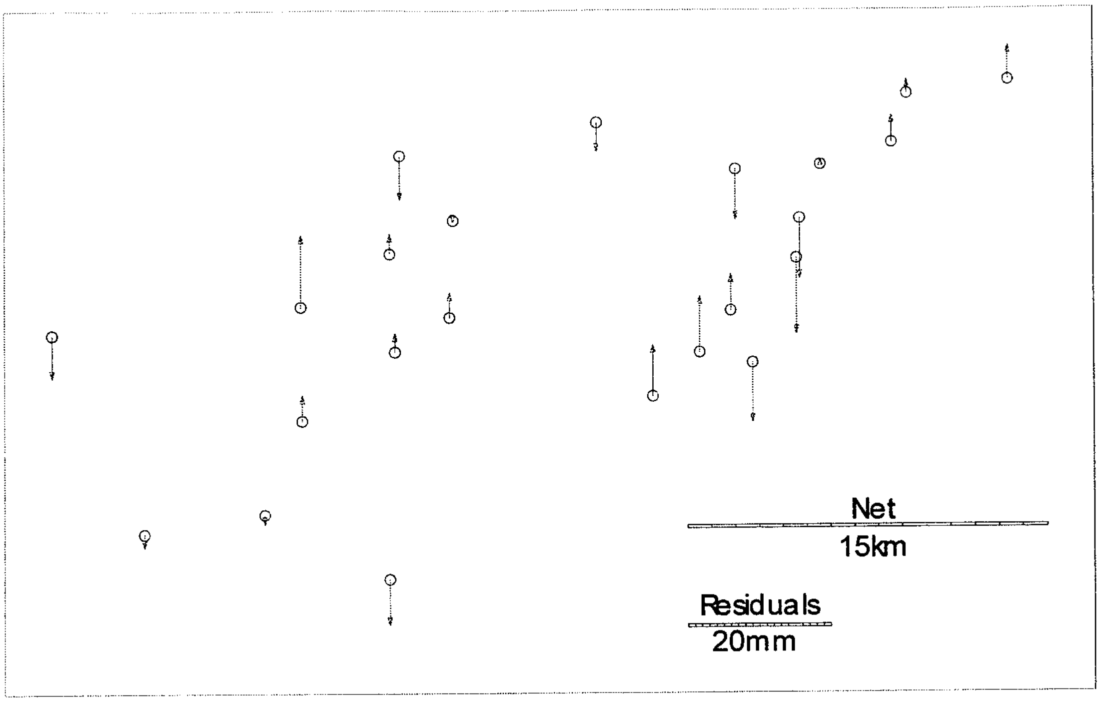


Fig. 5: GPS height integration for the Tallinn network by the pure geoid approach on taking the necessary datum-transition $\partial_{h,G}$ (**d'**,Δm_G) part for h and N_G into account: The residuals in the known control points - treated as new points – keep in the mean range of only ± 4mm.

3. HEIGHT SYSTEM TRANSFORMATION

The essential components of the above GPS height integration concept - namely the datum transformation part for heights (3) and the finite element representation NFEM(**p**,y,x) of a HRS (6) - may be transferred to the problem of transforming old heights H_{old} to new heights H_{new} of a new height system. In analogy to the above „geoid refinement approach" the most general approach for a height system transformation reads:

$$H_{old} + v = H_{new} + \partial H(\mathbf{d}) + NFEM(\mathbf{p},y,x) \tag{11a}$$

$$H_{new} + v = H_{new} \tag{11b}$$

The datum transformation parameters **d** as well as the parameters **p** of the finite element model are to be determined by identical points (H_{old},H_{new}) in both systems.

4. ONLINE GPS-HEIGHTING – PRODUCTION AND APPLICATION OF A DIGITAL FINITE ELEMENT HEIGHT REFERENCE SURFACE

4.1 Digital Finite Element Height Reference Surface (DFHRS) concept for an online GPS-Heighting

The profile and target of an online height positioning is easy to formulate (see fig. 6): An ellipsoidal GPS-height h, determined at a position (y,x), has to be converted directly to the height H of the standard height system. The converted height H should result online after applying a correction to h, and the resulting H should not suffer with a quality-decrease compared to the heights H resulting from a GPS height integration in postprocessing (approaches chap. 2)

In this chapter a general concept is presented, which fulfils all above requests and shows besides this even some more positive aspects. The concept is to produce in a first step in a controlled way a so called **D**igital-**F**inite-**E**lement-**H**eight-**R**eference-**S**urface (DFHRS) as a new kind of data base product (= production step). The second step is to make this data base accessible online – in an active or passive way - for DGPS heighting (= application step). That means, that either the DGPS user has the DFHRS at his disposal or the DGPS service exclusively uses the DFHRS for the evaluation of a correction Δ to transform a GPS height h to the height H of the standard height system (principle, see fig. 6).

The production step of the DFHRS reads in the system of observation equations as follows:

$$h \quad + v = H - (h+N\cdot W^2)\cdot \Delta m + NFEM(\mathbf{p},x,y) \qquad (12a)$$

$$N_G(B,L) + v = NFEM(\mathbf{p},y,x) - \partial_{h,G}(\mathbf{d}', \Delta m_G) \qquad (12b)$$

$$H \quad + v = H \qquad (12c)$$

Identical points (H, h) and if available, one or a number of geoid models $N_G(B,L)$ are used as observations to produce the DFHRS. The DFHRS on the right side is represented completely by the finite element model NFEM(**p**,x,y) of the HRS. NFEM(**p**,y,x) is modeled like in (6) with continuity conditions. The geoid model input N_G (B,L) is „mapped" to the DFHRS by removing the datum part $\partial_{h,G}(\mathbf{d}', \Delta m_G)$. An additional NFEM-refinement term may be set up in (12b). The production step of the DFHRS (12a,b,c) has to be embedded in a statistical quality control concept, e.g. of a least squares estimation, so that any component including the input of „mapped" and datum-adapted geoid-model, can be controlled.

The decisive components of the production step, which are afterwards needed in the application step - namely in an online GPS-heighting - are contained in (12a). Equation (12a) leads to the following correction scheme, which has to be applied to the GPS height h in an online application of the DFHRS data base:

$$H = h + \Delta = h + corr1 + corr2 = h - NFEM(\mathbf{p},y,x) + (h+N\cdot W^2)\cdot \Delta m \qquad (13)$$

The first correction part „corr1" is due to the DFHRS („geoid correction"), and „corr2" is due to the scale Δm between the GPS heights h and those of the standard height system H.

4.2 Example of DFHRS production

Fig. 7 shows the finite element grid of the 30km by 30km „Mosbach" area near Heidelberg, where a DFHRS was computed in the frame of a pilot project [1]. A first kind of DFHRS was produced using only identical points (H, h) in both systems, without geoid information $N_G(B,L)$ and a respective „geoid mapping" (12b). The DFHRS was evaluated for this case with a polynomial degree l=1 over a 16 mesh grid. In addition to the scale parameter Δm the complete DFHRS for the area (fig. 7) could in this case be represented by k=16 sets of each three coefficients $\mathbf{p}_k=(a_{00},a_{01},a_{10})_k$. For the special case that besides the identical points (h,H, see fig 7.) no geoid information was used for the evaluation of the DFHRS, the precision of the $\mathbf{p}_k$ of the DFHRS restricts the DGPS-based online heighting to a quality range of (1-3) cm.

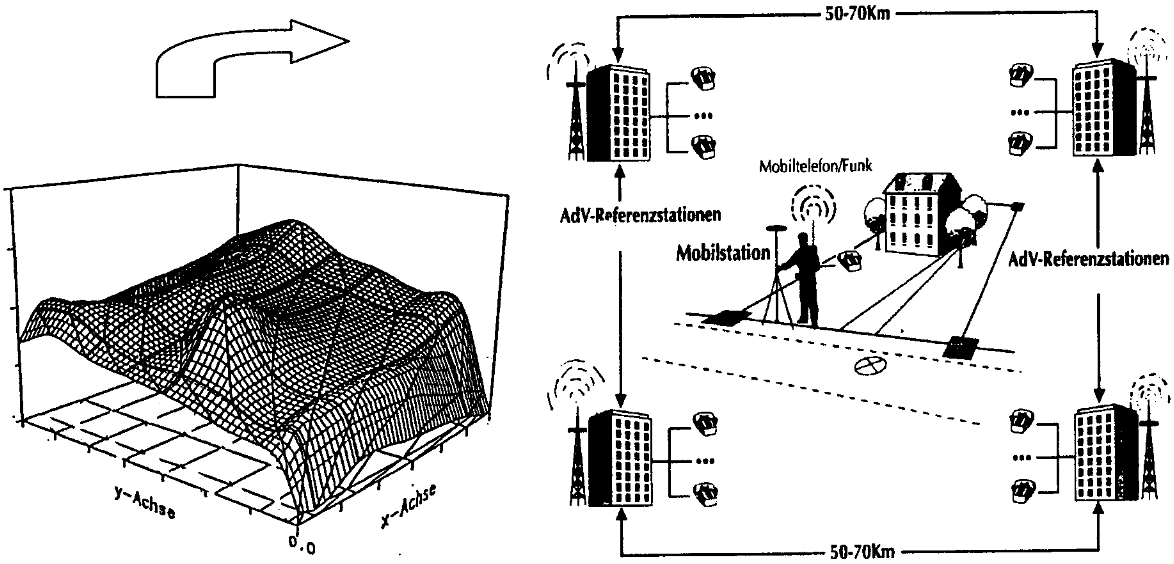


Fig 6: DFHRS (left) and its use (right) as DFHRS data base for a DGPS-based online heighting (h $\rightarrow$ H).

A quality increase and simultaneously a reduction of the number of identical points is achieved by an additional „geoid mapping“ (12b). The resulting DFHRS (12a,b,c,d) is then better than the geoid input, meaning that the additional mapping of e.g. the EGG97 cm-geoid will provide a cm or sub-cm quality for the resulting DFHRS product, as proved in respective investigations [12].

4.3 Outlook for the DFHRS concept

The DFHRS can be characterized as a new product appropriate for an online GPS-heighting with best quality and economical properties. The wellknown datum problem and individual datum calibration steps using identical points (h,H) in/before GPS heighting are completely dropped out. The DFHRS enables a direct GPS-heighting with a general usability for anybody in the frame of DGPS-applications and DGPS services.

The production of the DFHRS (12a,b,c) is performed as an overdetermined least squares adjustment, which enables a quality control of all components including the input of geoid models. The computation of the DFHRS product may be repeated at any time, as soon as new data (h, H, ΔH, Δh and N_G) arise, or even if a change of the height system type or datum is intended.

The variation of the mesh size further enables to produce on demand different DFHRS products with a different geometric quality (and prize) for an online height positioning purpose. Besides that there are two different ways for a DFHRS marketing: The first way is to keep the DFHRS on the side of the data- and DFHRS owner and transmit only the correction Δ (13) by the DGPS-service.

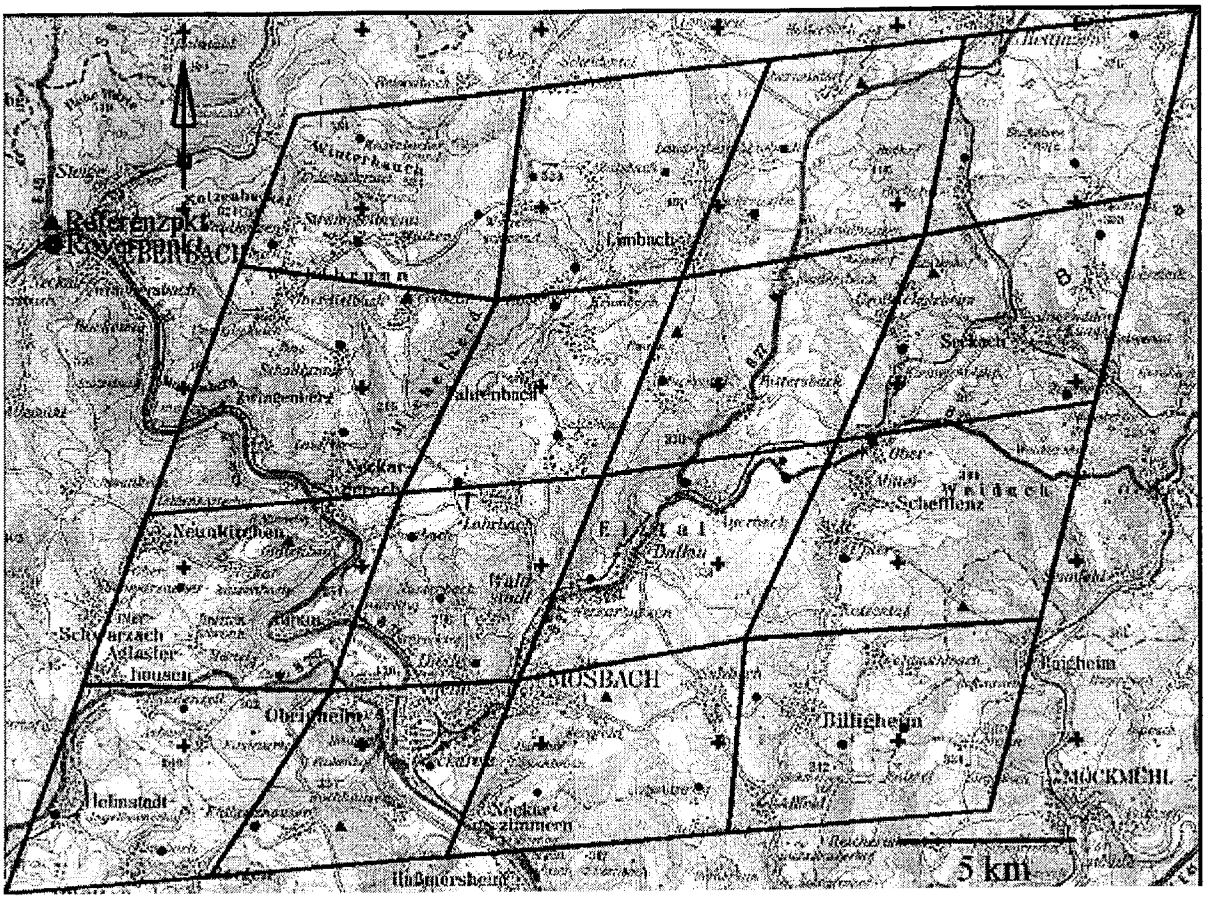


Fig. 7: Meshing and data design (H,h) of a DFHRS computation for the Mosbach region.

This requires however that the DGPS customer transmits (B,L,h) to the DGPS-service and gets back the corrected value H. The other way is of course to sell - like usual in the context with modern geoid-models [3] – the DFHRS data base directly to the DGPS user.

The first experiences with the DFHRS concept (12a,b,c) are much promising [12],[1]. As in most cases geoid information is available [3], the DFHRS evaluation may in general be set up together with a „geoid mapping“ (12b). For this complete case (12a,b,c) the best quality and control of the DFHRS will be achieved and at the same time the number of indentical points (h,H) for control and datum parameter estimation remains small even for large areas. The development of comfortable software for the production of DFHRS data bases is continued, and consequently also the implementation of DFHRS data bases in DGPS online software packages [13], [14].

REFERENCES

[1] Ackermann, C. (1999): Entwicklung von C-Software zur Berechnung Digitaler Finite Element Höhenbezugsflächen (DFHBF) und Erprobung im SAPOS-Netz als Pilotprojekt mit dem Landesvermessungsamt Hessen. Diplomarbeit am Studiengang Vermessungswesen, Fachhochschule Karlsruhe, unpublished.

[2] Bruttel, I. und C. Blattman (1997): Bestimmung präziser Landeshöhen mittels GPS - Planung, Realisierung und Bewertung des in Kooperation mit dem Landesvermessungsamt Karlsruhe

stattfindenden Baden-Württembergischen Pilotprojekts Mosbach. Diplomarbeit am Studiengang Vermessungswesen, Fachhochschule Karlsruhe, unpublished.

[3] Denker, H; Behrend, G. and W. Torge (1995): The European Gravimetric Quasigeoid EGG95. New Geoids in the World. International Association of Geodesy (IAG), Bulletin d'Information No. 77, IGS Bulletin No. 4.

[4] Dinter, G. (1997): Ein universelles Konzept zur Integration von GPS-Höhen in Gebrauchshöhensysteme. DVW-Schriftenreihe, Nr. 27. Konrad Wittwer Verlag, Stuttgart.

Internet: www.gik.uni-karlsruhe.de/~dinter/Forschung.

[5] Dinter, G; Illner, M; Jäger, R. und G. Schmitt (1997): Entwicklung und softwaremäßige Realisierung eines allgemeinen Modells zur Überführung von GPS-Höhen in Gebrauchshöhensysteme. Interner Bericht. Geodätisches Institut, Universität Karlsruhe.

[6] Dinter, G.; Illner und R. Jäger (1997): A synergetic approach for the integration of GPS into standard height systems and for the quality control and refinement of geoid models. IAG Subcommission for Europe Symposium EUREF, Ankara 1996. In: Veröffentlichungen der Internationalen Erdmessung der Bayerischen Akademie der Wissenschaften, Heft Nr. 57. Bayerische Akademie der Wissenschaften in Kommission bei der Beck'schen Verlagsbuchhandlung, München.

[7] Dinter, G.; Illner, M. and R. Jäger (1997): A general concept for the integration of GPS-heights into standard height systems comprising the quality analysis and refinement of geoid models. Submitted for Journal of Geodesy. Springer Verlag.

[8] Fellendorf, C. und M. von Wedelstädt (1998): Bestimmung von Landeshöhen mittels GPS - Planung, Realisierung und Bewertung des in Kooperation mit dem Landesvermessungsamt Mecklenburg-Vorpommern stattfindenden Projekts Schwerin. Diplomarbeit am Studiengang Vermessungswesen, Fachhochschule Karlsruhe, unpublished.

[9] Heiskanen, K. and H. Moritz (1967): Physical Geodesy. W.-H. Freeman and Company, San Francisco, London.

[10] Jäger, R. (1997): Modell- und Softwareentwicklung zur Integration von GPS-Höhen in Landessysteme und zur Qualitätsanalyse und Verfeinerung von Geoidmodellen. Horizonte 4/97 – Forschung an Fachhochschulen in Baden-Württemberg. ISSN 1432-9174.

Internet: www.fh-manheim.de:2000.

[11] Jäger, R. und H. Mengesdorf (1998): Höhenänderungen im Bereich des Pegels Mainz – Deformationsanalyse der terrestrischen Epochen 1938-1995 und Übergang auf eine GPS-basierte Höhenüberwachung. Allgemeine VermessungsNachrichten, Heft 9/98. S.272 - 283.

[12] Jäger, R. (1998): Ein Konzept zur selektiven Höhenbestimmung für SAPOS. Beitrag zum 1.SAPOS-Symposium. Hamburg 11./12. Mai 1998. Arbeitsgemeinschaft der Vermessungsverwaltungen der Länder der Bundesrepublik Deutschland (Hrsg.), Amt für Geoinformation und Vermessung, Hamburg. S. 131-142.

[13] Schwarzer, F. (1999): Entwicklung graphikfähiger C-Software zur Berechnung Digitaler Finite Element Höhenbezugsflächen (DFHBF) und DFHBF-Implementierung in GPS-Kommunikationssoftware. Diplomarbeit am Studiengang Vermessungswesen, Fachhochschule Karlsruhe, unpublished.

[14] Seiler, M. und A. Seib (1998): Entwicklung von WINDOWS-Software zur online Bedienung von DGPS-Empfängern mit NMEA-Schnittstelle für Feldaufnahme- und Navigationszwecke. Diplomarbeit am Studiengang Vermessungswesen, Fachhochschule Karlsruhe, unpublished. Internet: www.fh-karlsruhe.de/iit (see „Existenzgründer").

Accuracies achievable by GPS in practical engineering and surveying applications

PAUL CROSS

Department of Geomatic Engineering

University College London

ABSTRACT

This paper is concerned with the accuracy of GPS when used in a dynamic mode for the most common surveying and engineering applications. It highlights three issues: modelling of multipath errors, correct assignment of stochastic models for basic GPS phase data and calibration of the performance of GPS systems. It is concluded that more work is required, especially in the detailed understanding of errors sources, before the full potential of GPS can be exploited by practising surveyors and engineers.

INTRODUCTION

GPS is rapidly becoming the default measurement tool for a very large number of standard civil engineering and land surveying operations. These include classical activities such as control surveys, topographical detail collection for large scale plans, setting out, and monitoring of deformation - as well as new developments such stakeless surveying (i.e. automatic guidance for civil engineering plant) and futuristic applications such as on-site robotics. For all of these applications, knowledge of the accuracy being delivered by GPS is crucial. It is, however, only very rarely the case that the quality measures output by the GPS data processing component of the operation (either in real-time or in post-processing mode) properly describe the real quality of the results. This paper is concerned with quality of GPS - what determines it and how it is assessed.

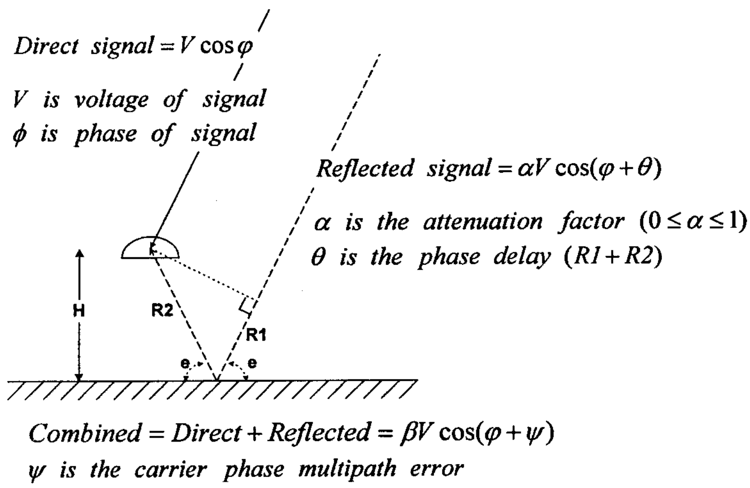
For many practical surveying and civil engineering applications the most significant error sources are those associated with multipath error estimation and analysis, as it is usually the single-most important problem in kinematic GPS applications. In this paper typical sizes and patterns of multipath errors are presented and methods for modelling it in real time are reviewed. Finally a method is discussed that can, for slow moving antennas, reduce such errors by about 50% of their normal size and some possible engineering applications of the method are highlighted. Throughout the analysis emphasis is placed on the height component of the position error. This is for two reasons: firstly it is the largest of the three components and so represents the 'worst case' scenario, and secondly to reflect the theme of this meeting (*importance of heights*). The Swedish National Land Survey has an international reputation for its excellence in the research and development of precise height measurement techniques and GPS is naturally playing an increasingly important role in its operations.

Evidence is presented for strong spatial correlation between phase measurements made to different satellites. It is shown that the fact that this correlation is ignored, which is normal practice in GPS data analysis, is a major factor in the poor performance of current quality assessment methods and real-time quality control measures.

Finally the issue of performance calibration of GPS systems is briefly discussed. This is a topic that has received little attention to date but is of increasing importance. It is easy to compare the performance of traditional surveying instruments such as EDM - where accuracy and range are usually simply related (and where the environment is of rather little importance). For GPS there are many other factors to consider, such as time to first fix (for RTK), range for guaranteed ambiguity determination, performance in partially masked environments (e.g. under light tree canopies), susceptibility to multipath and accuracy in different kinematic scenarios. Recommendations are made for some of the components of a standard testing facility to compare the performance of commercially available (and other) GPS systems.

MUTLIPATH MODELLING AND ESTIMATION

Multipath is caused by the mixing of GPS signals reflected from (usually nearby) objects with those directly received from the satellite. The basic set-up is shown below.



Detailed analysis of multipath shows that it is largely driven by the nature of the reflecting surface and that the maximum amplitude is one quarter of the wavelength of the signal being measured (i.e. 0.048m and 0.061m for L1 and L2 respectively). It is cyclic in nature (due to continuous change of distances R1 and R2), with the frequency depending of the distance of the reflecting object from the antenna (a nearby object leads to lower frequencies) and the satellite elevation angle.

Multipath causes an increase in 'noise' that leads to two related (but essentially different) problems. Firstly it is more difficult to solve for ambiguities – so the whole process fails more often that it would if multipath were not present, and secondly, even when the process succeeds, the results will be of a poorer quality. This paper is fundamentally concerned with the second of these problems.

In some applications it is possible to mitigate the amount and/or effect of multipath by adopting strategies such as:

- Siting antenna near surfaces with low reflectance
- Laying radio absorbent material around the antenna
- Using special antenna designs (with ground planes or choke rings)

But in practical engineering application these strategies are rarely applicable – the nearby surfaces are likely to be highly reflective (e.g. steel structures) and it might be essential to use lightweight antenna. Hence approaches based on modelling are often the only solution.

Multipath errors are clearly not random and are driven by basic physics. In principle they would be known if the exact geometry of the observing site was known and the reflectance properties of the reflecting surface were understood. Practical attempts to models multipath in this way have failed so most researchers in this area are trying more empirical approaches.

For instance the use of a technique based on signal to noise ration (SNR) developed by Comp & Axelrad (1996) has been investigated by Barnes, Ackroyd & Cross (1998) to assess its potential for RTK engineering surveying. What follows is a summary of the results - for more details of the mathematics and algorithm see *ibid.*

The data used for the test was collected on the roof of a building for a period of one hour with an elevation cut-off angle of zero degrees and a recording interval of 1Hz. Trimble 4000 SSE receivers were used and the baseline length was approximately 10m. Note that previous data collected on this roof had shown significant multipath. Full details of the data set and environment are given in (Barnes and Cross 1997) and a sample of the true double difference residuals (negated error) is given in Fig. 1.

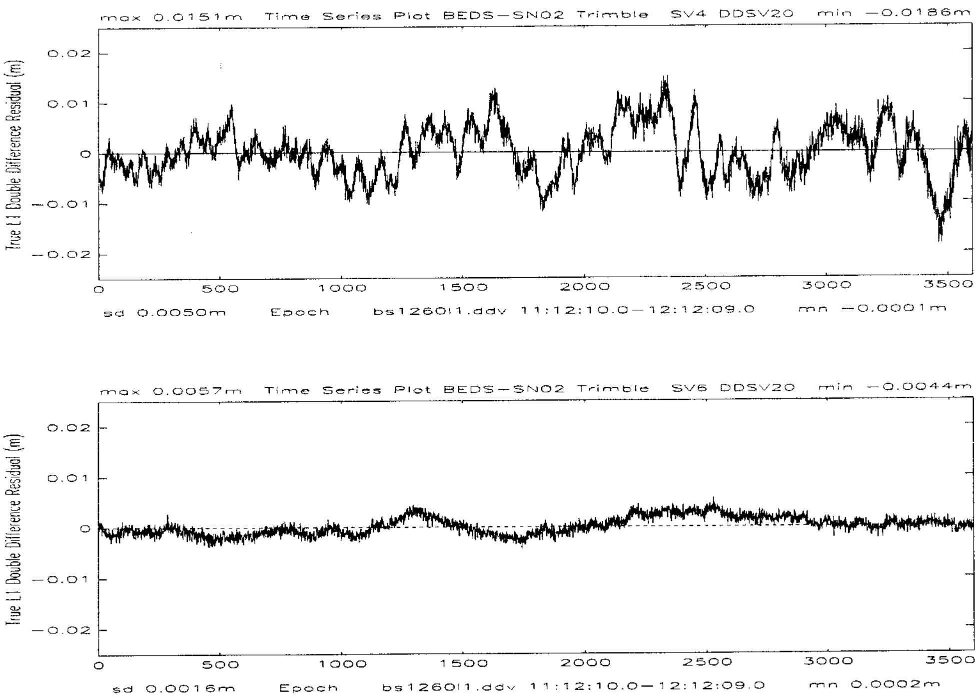


Fig. 1: True L1 double difference residuals for SV4 (20°-3°) and SV6 (50°-80°)

Several important (and well-known) trends can be seen in Fig. 1. The two most important, and most obvious, are the cyclic nature of multipath and its dependency on elevation angle. Fig. 2 shows a time series plot of the error in height following processing by the GASP software. The basic approach of GASP is described in Corbett & Cross (1995) but it is important to note that the results presented here will be 'general' and not dependent on GASP. This is because whilst different software might use completely different strategies for ambiguity determination (and so have different success/failure characteristics), when it comes to computing short vectors from a single epoch of double difference phase data (with already known ambiguities) they all use basically the same algorithm. Errors of up to 20mm can be found in Fig. 2 and a clear cyclic pattern - caused by the multipath is evident.

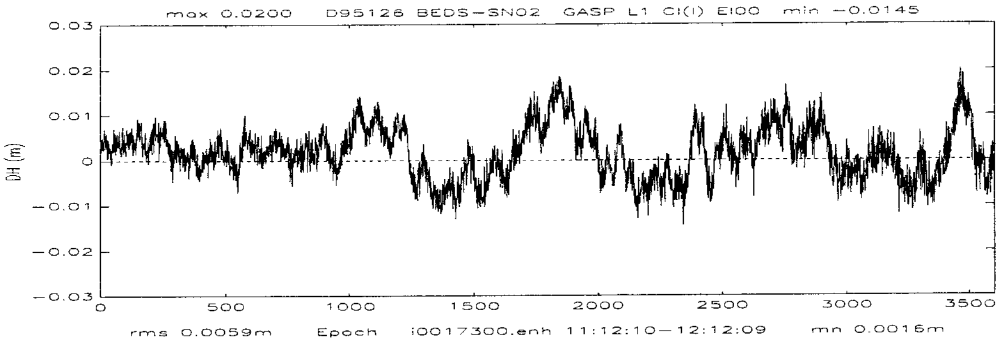


Fig. 2: GASP results for a scaled identity covariance matrix

The double difference SNR based multipath estimate was then computed for each L1 double difference phase observation and applied before again computing the positions with GASP. This time a simple elevation dependent stochastic model was used to reduce the effect of low elevation satellites. The new position error time series (height component) is shown in Fig. 3.

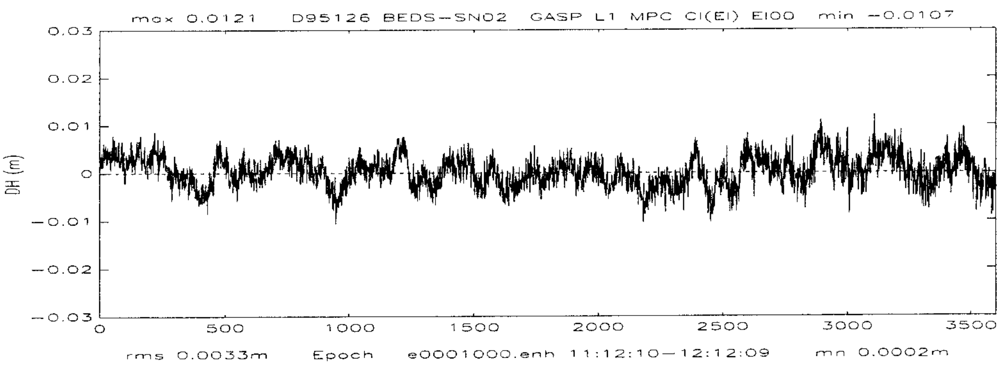


Fig. 3: GASP results following correction for multipath (SNR method) and use of an elevation dependent stochastic model

It can be seen that the RMS height error is 3.3 mm which is a 44% improvement over the standard current solution (Fig. 2, no multipath modelling and an identity matrix as a stochastic model). Medium to high frequency multipath fluctuations still remain in the series, but the low frequency multipath present in Fig. 2 are mostly removed in Fig. 3. It is important to note that the successful application of the SNR-based multipath technique relies heavily on an accurate knowledge of the antenna gain pattern in order to determine the direct SNR signal. It is possible that gain patterns might become distorted when the antenna is placed close to large conducting objects. Also routines which estimate the frequency and amplitude parameters are needed. The adaptive notch filter and adaptive least squares methods used here work sequentially and converge in typically 5 to 10 minutes, they are therefore suited to real time. However currently it is necessary to estimate manually the number of reflected multipath signals. Better spectral analysis routines would undoubtedly improve the final multipath estimates and the existing routines might work even better if the SNR had greater resolution. Hence, although the method clearly has enormous potential, considerable research and development would be needed before it could be implemented within a real-time kinematic GPS processing system.

STOCHASTIC MODELLING AND ACCURACY ASSESSMENT

Although not discussed in detail here the standard precision measures associated with the foregoing solutions do not, in general, reflect the real quality of the data. In other words the variation in the true position errors (e.g. those in Figs. 1 and 2) are not matched by similar fluctuations in the time series of the standard deviations. The formal standard deviations, as expected, simply show gradual, almost linear, changes due to changes in geometry plus occasional jumps due to new satellites entering or leaving the observable part of the constellation. Also, although the average unit variance oscillates wildly, it does not correspond to the true errors. The mean unit variance is, however, close to unity, reflecting the fact that the average values for the variances of the double difference phase measurements have been correctly estimated.

It is a fundamental goal of those working directly with GPS data and developing algorithms and software for position estimation that any formal precision measures computed from the least squares process should really reflect the quality of the solution. The currently available GPS processing software does not do this. The fundamental reason is that multipath errors vary rapidly with time – so causing the quality of the data to vary rapidly. All current solutions use very simple stochastic models, either an identity matrix (multiplied by a constant) or some sort of simple elevation based function (to force the final solution to better fit the date from higher elevation satellites).

Better understanding the multipath can lead to improvement in both the quality of the final solution and in the fidelity of the quality measures. The first part of this assertion has already been proved (by the differences between Figs. 2 and 3). That the second part is, in principle, true is shown by an experiment based on the following procedure.

- For each double difference, at each epoch, the true errors have been computed using the known station co-ordinates. Time series such as those in Fig. 1 have been obtained.
- Each double difference true error has been squared and also cross-multiplied by every other error at that epoch, time series of squared values and cross-products have hence been produced.
- Using a simple moving average (based on a one minute window) average values of these squares and cross products have been computed. These are estimates of the variances and covariances of the true double difference errors - so enabling a covariance matrix of double differences to be constructed at every epoch.
- Positions, precision measures and test statistics have been computed in the normal way.

This procedure is called here the *reverse engineered* solution – because it would not of course be possible without prior knowledge of the positions (something that would never be known in practice). The results from the application of this procedure to the data set already introduced are shown in Fig. 4

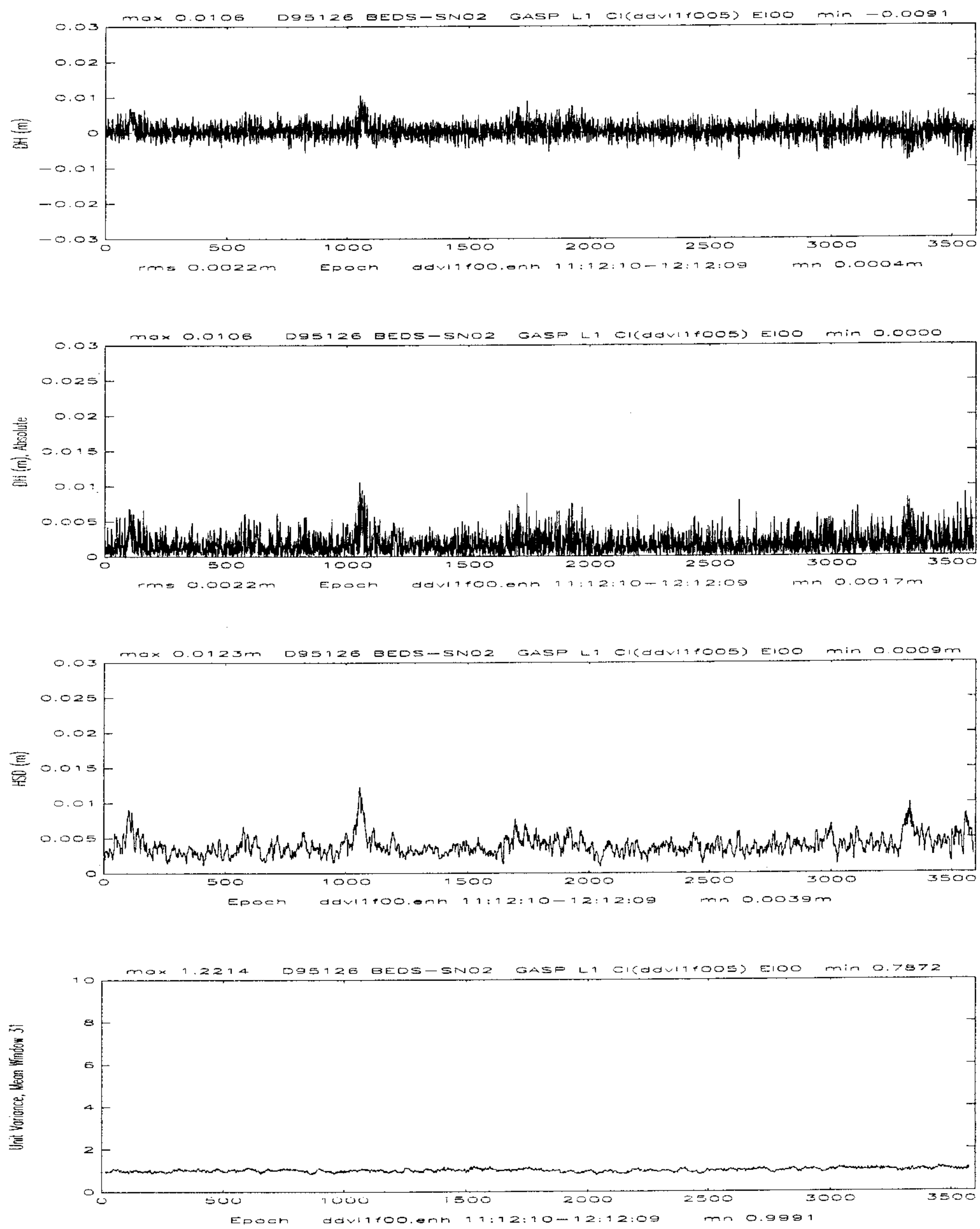


Fig. 4: *GASP results for a correct (reverse engineered) stochastic model*

The overall height RMS from the reverse engineered solution is 2.2 mm, a 63% improvement over the identity matrix model, and the maximum height error is reduced to 1.1 cm. Clearly the use of a more correct stochastic model has significantly improved the results. Perhaps more important is the fact that the series exhibits a much more random (white noise) behaviour with only occasional small increases and no low frequency multipath fluctuations. This shows that the use of the correct correlation in the stochastic model has had the same affect as modelling the multipath error as part of the functional model. This will not surprise geodesists with knowledge of estimation theory. It is well known that, in principle, systematic measurement errors can either be treated functionally or stochastically. The reason for including the experiment here is simply to emphasise the power of proper stochastic modelling and to encourage research into the derivation of transferable models for typical GPS operations and environments.

Perhaps the most valuable conclusion, however, relates to the quality measures. It can been seen clearly from Fig. 4 that the height standard deviations follow almost exactly the trends in the absolute height error and give a very high fidelity indication of position quality. The more correct models, not surprisingly, not only produce better quality results, but also produce better estimates of that quality – something that is most definitely missing from current solutions.

GPS CALIBRATION ISSUES

As has been mentioned, during the past two or three years the RTK technique has become the preferred surveying method for a wide variety of applications (especially in engineering). This trend does, however, raise a number of performance related issues worthy of serious consideration. One obvious question is how accurate is a particular RTK system? It is relatively simple to assess the precision of such systems but to examine accuracy in a truly kinematic environment is non trivial (the earlier descriptions in this paper are based on static tests), especially in environments that exhibit variable (in time and space) multipath conditions. Other questions include: how fast can the system recover initial ambiguities, how is this speed affected by dynamic environments such as multipath and foliage, how do differing satellite geometries and DOP (Dilution of Precision) values affect system performance, and how does one know that the resolved ambiguities are correct? In short, how would one go about benchmarking a RTK GPS system?

This section presents a number of tests, described in more details in Edwards et al (1999) which together might form the basis of a methodology for the assessment of the performance of such systems, together with some typical results that might be obtained. Detailed analysis of such test results may go a long way to answering some of the questions raised. It should be noted that the main purpose of raising this issue in this paper is to stimulate debate on the topic, not to discuss the performance of particular systems in detail.

Kinematic tests

One of the fundamental problems in the development of benchmark tests for RTK GPS is how to assess accuracy as well as precision. In order to achieve this in a truly kinematic environment the 'true position' of the roving GPS antenna must be recorded at the same instance that a GPS position is recorded. One place at which a facility capable of doing this has been constructed is at the Central Laboratory of Roads and Bridges, Nantes, France, Peyret (1995). The facility known as SESSYL, is essentially a mobile petrol-powered train that can carry a GPS receiver and antenna and which travels on an oval track (about 180m in length) with a known trajectory. GPS positions can be compared with 'true positions' in order to measure their accuracy (rather than just the precision). Although the track is located in a basically clean multipath environment, multipath can be induced by attaching a metal plate to the train and by parking large vehicles near to the track. Complete losses of lock can also be caused by temporarily erecting a wire mesh tunnel over part of the track. In February 1997 a number of tests were carried out using SESSYL and are reported in detail in *ibid.* The objective was to:

- assess the accuracy of an RTK system using different antenna combinations in both clean and multipath environments, and
- measure the speed of recovery of ambiguities (On-The-Fly) after complete loss of lock on all satellites.

Ambiguity resolution tests

A crucial factor in the operational use of RTK GPS is the speed with which it can initialise (i.e. solve for integer ambiguities), prior to the commencement of a survey or at some point during it following 'loss of lock' due to the masking of some or all of the satellites (for instance after passing under a bridge). In an optimum environment, free from multipath and obstructions one might reasonably expect any RTK system to resolve for integer ambiguities in around 2 minutes (120 sec) or less. Of greater interest, however, is the effect that varied dynamic environments with medium/high multipath, or partial tree cover might have on initialisation times.

Following the somewhat inconclusive results obtained from the SESSYL ambiguity resolution tests, the following experiments were designed to examine further RTK ambiguity resolution performance. Of particular interest was ambiguity resolution under harsh conditions, namely in the presence of high multipath and under partial tree cover. The overall objective was to:

- assess the speed and accuracy of an RTK system for the computation of initial integer ambiguities with different satellite geometries.

The method adopted for these experiments was to undertake a series of controlled re-initialisations of the RTK system over a period of time. This was achieved by using successive antenna pulls (removal of antenna leads) at the rover receiver forcing total loss of lock to all satellites, as though driving under a bridge. Two environments were chosen, a concrete rooftop (known to induce high levels of multipath) and an area of medium to dense tree cover. Average baseline lengths were 10m and 50m respectively for the rooftop and tree cover environments.

Signal tracking sensitivity test

The experiments outlined in the preceding section provide a method of testing the ability of an RTK system to recover ambiguities in two differing dynamic environments. However during a typical survey the rover antenna is often moving within such environments, e.g. an urban area or along a hedgerow containing mature trees. Of particular interest is the ability of an RTK system to deliver fixed ambiguity solutions whilst moving through such an environment which will cause temporary degradation/obscuring of the GPS signals. A method of quantifying system performance in just such an environment now outlined below.

The methods used for this test could range from the very complex to the very simple. One simple test would be to mount the roving RTK antenna on to the roof of a vehicle, and then drive the vehicle along an avenue of trees. The tree canopies would interfere with the incoming GPS signals to varying degrees as the vehicle moved along. The result would be degradation in signal quality and an increase in the noise within the system, which might well affect the ability of the system to deliver fixed ambiguity solutions. Clearly standard conditions need to be described so that the experiments could be (almost) replicated in different parts of the world.

General remarks

We live in a world in which it is expected that the quality and performance of almost everything we buy and/or use can be described by a set of standard parameters (albeit often empirical). Indeed such parameters (when fully understood and properly interpreted) can be extremely helpful when selecting between choices. For instance when buying a car it would be natural to compare an enormous number of them: fuel consumption, top speed, boot (trunk) capacity, length and width, safety and comfort features, etc.

In the geodetic sciences we have been used to dealing with fewer parameters, perhaps just accuracy, range and weight, but with RTK GPS the situation is far more complicated, as this paper has shown. It is important to consider, *inter alia*, accuracy, time to resolve ambiguities and ability to maintain lock, as well as others (not addressed here) such as operating distance. Unfortunately, and unlike traditional systems, all (or at least most) of these things depend very heavily of the prevailing conditions (multipath, level of masking, nature of dynamics, etc) and on a number of system features (type of antennas, satellite geometry, number of satellites, etc), so assigning suitable parameters is not a simple task.

CONCLUSIONS

Multipath is the dominant technical problem in the (even more) widespread use of GPS in kinematic operations for general surveying and engineering applications. It affects the accuracy that can be delivered, the fidelity with which that accuracy can be described and the ability to derive integer ambiguities in order to start the positioning process.

It has been shown that there are two modelling approaches that can, in principle, alleviate the problem – but much more research is needed (a) to turn them into practical real-time solutions, and (b) to understand them better in order even to begin this process.

Also this paper has indicated a number of tests that could perhaps be used to develop standard parameters to describe the performance of RTK GPS. What is now needed is for professional and scientific bodies to take the initiative in designing standard tests in standard conditions. It will then be possible for all potential users to make informed judgements between competing equipment. At the moment it is extremely difficult for anyone, even someone with a deep knowledge of GPS, to select hardware in an objective manner.

REFERENCES

ASHKENAZI V, MOORE T, HILL C., CROSS P., ROBERTS W, HAWKSBEE D, MOLLON K., WINCHESTER R., JACKSON T, (1994) Standards for Benchmarking Differential GPS. Proceedings of ION GPS-94. Salt Lake City, Utah, USA. The Institute of Navigation. p1285-1290.

BARNES J B and P A CROSS (1998) Processing models for very high accuracy GPS positioning. Journal of Navigation, Vol 51, No 2, p180-193.

BARNES J B, N ACKROYD and P A CROSS (1998) Stochastic modelling for very high precision real-time kinematic GPS in an engineering environment. XXI Congress of the International Federation of Surveyors (FIG), Part 6, Paper TS13, p61-76 (ISBN 0-85406-902-X).

BLEWITT G (1997) GPS data processing methodology: from theory to applications. In GPS for Geodesy, A Kleusberg and P J G Teunissen (Eds.), Springer-Verlag.

COMP C J and P AXELRAD (1996) An adaptive SNR-based carrier phase multipath mitigation technique. Proceedings of the 9th International Technical Meeting of the Satellite Division of the Institute of Navigation, ION GPS-96, part 1, pp. 683-697.

CORBETT S J and P A CROSS (1995) GPS Single Epoch Ambiguity Resolution. Survey Review, Vol 33, No 257, p149-160.

EDWARDS S J, P A CROSS, J B BARNES and D BETAILLE (1999) - Benchmarking of GPS RTK systems. *In press* for Survey Review.

HOFMANN-WELLENHOF, B., H. LICHTENEGGER and J. COLLINS (1993) GPS Theory and Practice. Second edition, Springer-Verlag.

GEORGIADOU Y. and A KLEUSBERG (1988) On carrier signal multipath effects in relative GPS positioning. Manuscripta Geodaetica, p172-179.

LEICK A (1995) GPS satellite surveying, John Wiley, Second edition, 1995.

PEYRET F (1995) SESSYL: the study of the positioning systems for road construction robotics. Proceedings of the 12 th International Symposium on Automation and Robotics in Construction, Warsaw, p55 - 62.

Multi-Sensor Systems for Height Determination

Yang Gao and Naser El-Sheimy

Department of Geomatics Engineering

The University of Calgary

2500 University Drive, N.W.

Calgary, Alberta, Canada T2N 1N4

ABSTRACT

Over the past several years, extraordinary achievements have been made in the use of GPS for a wide range of applications, e.g., navigation/aviation, mapping, fleet management, surveying, meteorological monitoring and precise farming. A key element in the rapid growth and success of these applications has been the integration of GPS with other enabling technologies/sensors. Such growth and success is expected to continue in the next few years as accuracy and integrity from GPS are further improved. This paper describes the basic features of several multi-sensor systems that integrate GPS with other technologies such as INS, barometer, and vision based systems. System configurations of these systems are described along with their achievable positional accuracy based on the results from the field survey. Issues related to the further improvement of these systems are also addressed.

1. INTRODUCTION

Over the past several years, extraordinary achievements have been made in the use of GPS for a wide range of applications, e.g., marine navigation, aviation, fleet management, surveying, meteorological monitoring and agriculture. A key element in the rapid growth and success of these applications has been the integration of GPS with other enabling technologies, so-called multi-sensor systems. The sensors employed in the integration are complementary in functions and in most situations only the integration approach is able to provide satisfactory solutions for the targeted applications. Presented in this paper are the descriptions of several multi-sensor systems that have been developed for several applications with varying accuracy requirements. Their system architectures and positional accuracies are analyzed with a focus on the height component. Some test results are also presented to analyze the potentials as well as difficulties in the development of multi-sensor systems.

2. GPS AND MULTI-SENSOR INTEGRATION

Measurements from GPS satellites affected by various error sources including satellite orbit errors, receiver and satellite clock errors including Selective Availability (SA), atmospheric delays and multipath (Wells et al., 1986). The autonomous positioning accuracy is in the range of 100m under

SA and 30m with SA turned off. In order to achieve higher positioning accuracy differential GPS (DGPS) techniques must be employed to provide corrections to eliminate or diminish the effects of these errors. Although there exist various DGPS methods, most of them can be classified into the following three categories according to the size of their differential coverage:

1) Local Area Differential GPS (LADGPS)
2) Regional Area Differential GPS (RADGPS)
3) Wide Area Differential GPS (WADGPS)

A LADGPS system usually is dedicated to a specified small area (baseline length from a few km to tens of km) and very high positional accuracy can be achieved when carrier phase measurements and integer ambiguity resolution technology are employed. Differential corrections using LADGPS are usually established based on data from a single reference station. On the other hand, both RADGPS and WADGPS are network type of approach that uses a network of multiple permanent reference stations to provide large area differential coverage (baseline length ranges from several hundreds for the RADGPS to thousands of kilometres for the WADGPS). In terms of positional accuracy, WADGPS has been commonly used to provide metre-level positioning accuracy based on the pseudorange measurements while RADGPS is aimed at providing decimetre/centimetre accuracy based on carrier phase measurements. A combination of the above three system approaches is capable of providing a complete differential GPS service with accuracy from metre-level to cm-level.

Although DGPS system have been applied in a variety of applications as an independent system, integrating GPS with other sensors have been important for two major reasons, namely

1. GPS-only approach has been often found difficult providing satisfactory solutions for many applications in terms of accuracy and reliability. For instance, cm-level DGPS positioning relies on the correct determination of the integer carrier phase ambiguities which is difficult in high dynamic environments with frequent cycle slips. Auxiliary measurements are often helpful improving the obtainable accuracy and reliability from GPS.
2. Integrating GPS with enabling technologies can enhance the capability of GPS. e.g., allowing GPS to be applied to applications previously found difficult or impractical. For instance, signal blockage and attenuation have prevented the effective use of GPS in the areas such as forestry and city centre surrounded with tall buildings. An integrated multi-sensor system might be the only choice to provide a satisfactory solution to applications within those areas.

In the following sections, the above issues will be discussed with the description of several multi-sensor systems and their innovative use of GPS to different applications. The impact of the integration on the height component will be highlighted in the analysis.

3. DGPS/BAROMETER SEISMIC SURVEY SYSTEM

In this section, a GPS/Barometer seismic survey system is described which is one excellent example of extending the use of GPS technology to areas previously found difficult or impractical through multi-sensor integration.

GPS has found increased applications in the seismic industry when the Real-Time Kinematic (RTK) systems became available. However, the system is only effective in open areas as signal blockage and attenuation including multipath have prevented its effective use in forested areas. Because of this, GPS seismic survey crews only replace conventional crews in open areas, with the conventional crews being shifted to forested and swamp areas as precise GPS positioning is difficult in these areas without significantly increased occupation times, clearing of trees or raising the GPS antenna above the canopy. Meanwhile, natural resource and geophysical companies are encountering increased pressures to reduce tree cutting for the reasons of high cost and environmental impact.

Since it is the height component from GPS that is most severely affected in the forested areas, an efficient solution to the above problem is to integrate a sensor that is capable of providing satisfactory vertical control in those areas. Shown in Figure 1 is an DGPS/Barometer Real-Time Kinematic (RTK) system developed by Eagle Survey Ltd. By integrating GPS with a digital barometer, the system is able to provide horizontal positioning accuracy at the 20cm level and vertical positioning accuracy at the sub-meter level in both open and tree covered areas. Shown in Figure 2 were the positional results for the system through a heavily treed area. This multi-sensor system has been successfully used in a number of seismic survey projects (Gao et al., 1996).

The obtainable height accuracy from barometers depends on the barometer accuracy, the geographic terrain and the barometric data processing method. Traditionally, barometric heighting has been long considered only as a lower accuracy (several meters) height determination approach and has been used primarily for reconnaissance, aircraft navigation and mapping. With the presence of digital barometers with accuracies at the level of 0.02 mbar with a drift at the level of about 0.01 mbar (Copley, 1994), however, differential barometric height accuracy can be well controlled within the one metre range. Further accuracy improvement, however, requires the enhancement of the barometer performance and the development of more sophisticated data processing method. Using multiple base barometric stations can also help minimize the errors in barometric height measurements.

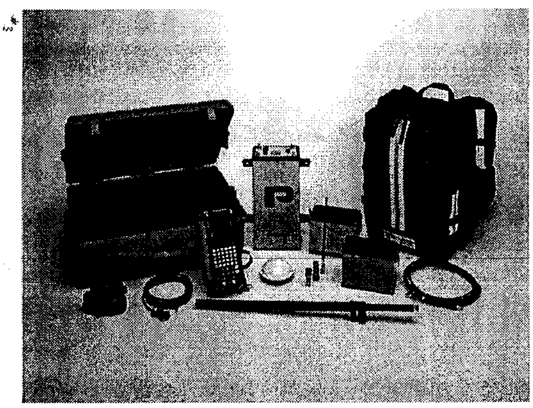


Figure 1 – DGPS/Barometer Seismic Survey System

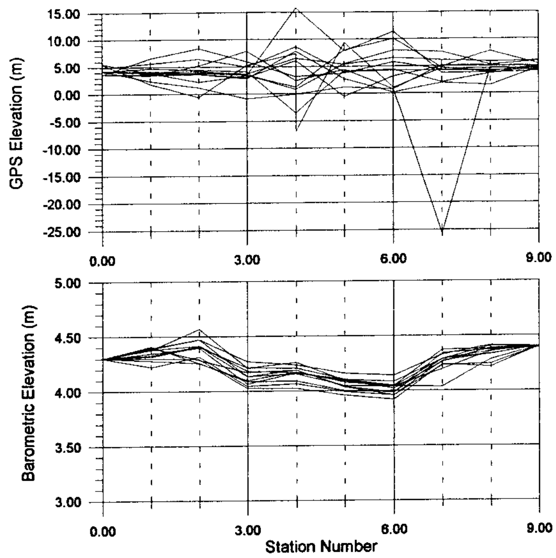


Figure 2 - GPS and Barometer Height Profiles

4. DGPS/INS Bridging METHODS

In this section, the complementary roles of GPS and INS and the so-called DGPS/INS bridging techniques are described.

In terms of system performance, INS is able to provide extremely accurate velocity, position, and attitude with low noise and negligible time lag for short time intervals. However, because INS position and velocity are obtained by integration, they are subject to adverse effects by low frequency errors. Thus, INS position and velocity errors grow with time. Figure 3 depicts a typical error behavior of the positional solution from an INS system, generated based data from one of the test runs at the University of Calgary Research Park. To obtain accurate outputs at all frequencies, the INS therefore must be updated periodically using external measurements with good long-term position accuracy. In this respect, GPS positions are ideal update measurements.

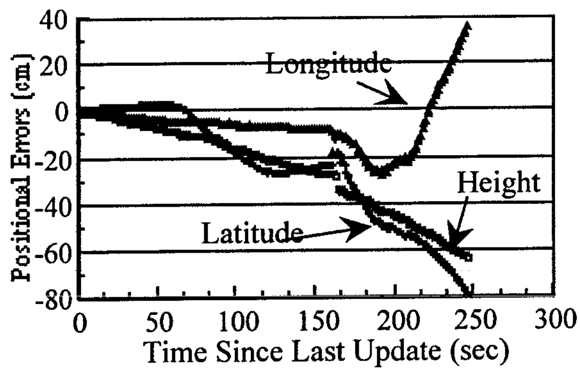


Figure 3: INS Accuracy in Stand Alone Mode

GPS on the other hand, can deliver excellent position accuracy, but has the problem of cycle slips and signal outages. Cycle slips are in essence gross errors leading to a discontinuity in the trajectory. GPS therefore requires precise external position and velocity measurements for fast signal reacquisition after outages and for fast cycle slip detection and recovery.

The integration of GPS and INS, therefore, provides a survey system that has superior performance in comparison with either a GPS or an INS stand-alone system. For instance, GPS derived positions have approximately white noise characteristics over the whole frequency range. The GPS-derived positions and velocities are therefore excellent external measurements for updating the INS, thus improving its long term accuracy. Similarly, the INS can provide precise position and velocity data for GPS signal acquisition and reacquisition after outages. This reduces the time and the search domain required for detecting and correcting cycle slips (El-Sheimy et al, 1995)

The complementary roles of GPS and INS in an integration scheme can be applied to significantly improve the efficiency, flexibility and reliability for a navigation system. Bridging using INS position is one typical example of the improvement that can be achieved in GPS/INS integrated systems.

The basic idea behind INS bridging is the use of the INS predicted coordinates to reset the position of the GPS after loss of lock and signal reacquisition. The bottom line, in INS bridging, is that any errors incurred during unaided INS operation should be below half a cycle in order to prevent a bias to the GPS computation following reacquisition of lock. The reason for this INS induced bias is that the inertially derived position is used in re-computing the phase ambiguities. Since they are considered as constant biases, until the next loss of lock, errors in the ambiguity determination, due to proportionate errors in the INS coordinates, will show up as a constant bias. Should there be a large number of satellites available, the GPS filter will attempt to reestablish precise ambiguity terms in kinematic mode. Although this is not uniformly reliable, since it is geometry dependent, it works well if there is a longer period of uninterrupted GPS observations with at least 4 satellites.

In general, the GPS/INS bridging consists of forward bridging and backward bridging. Forward bridging is used as a prediction method and can be employed to real-time applications while the backward bridging is essentially a smoothing process as a post-mission method. For non real-time applications, GPS/INS backward smoothing can considerably extend the bridging intervals after signal reacquisition. The principle is shown in Figure 4.

In the forward prediction process, GPS information up to the time epoch T_1, at which the GPS signal is lost, is used to predict the trajectory during the period T_1 to T_2, in which no GPS data is available. This results in the error curve E_1. Due to the double integration of errors at time T_1, the error curve E_1 increases rapidly. Using the information in time sections T_2 to T_3 after GPS signal reacquisition, a backward prediction can be used for the T_1-T_2 section, resulting in the error curve E_2. It is the mirror image of the error behavior of E_1. A smoothing procedure combines both the forward prediction and the backward prediction optimally, and gives an improved estimation of the trajectory with the error curve E_3. As can be seen, the bridging interval is greatly extended by using this procedure. For more details about INS bridging, see Schwarz at al (1994) and El-Sheimy (1996b).

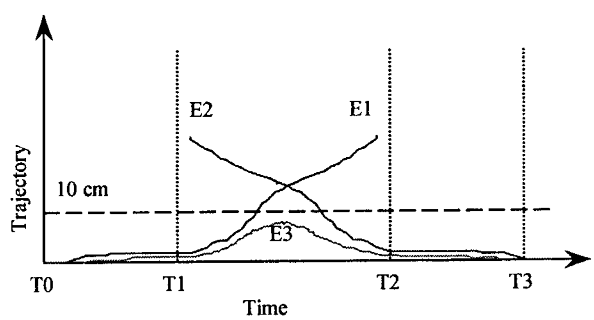


Figure 4: GPS/INS bridging with forward-backward predictions and smoothing

Figure 5 depicts the error behavior of the INS in stand-alone mode based on data from a test run. GPS observations were removed from the data for a 100 s period and the INS data was processed in stand-alone mode. The truth model of this diagram was obtained by using the trajectory computed from the original GPS/INS measurement with accuracy ranging from 2-5 cm. The INS stand-alone positioning results stay below the half cycle level (10 cm for L1 and 43 cm for wide-lane) for about 30 s for L1 and 52 s for the wide-lane.

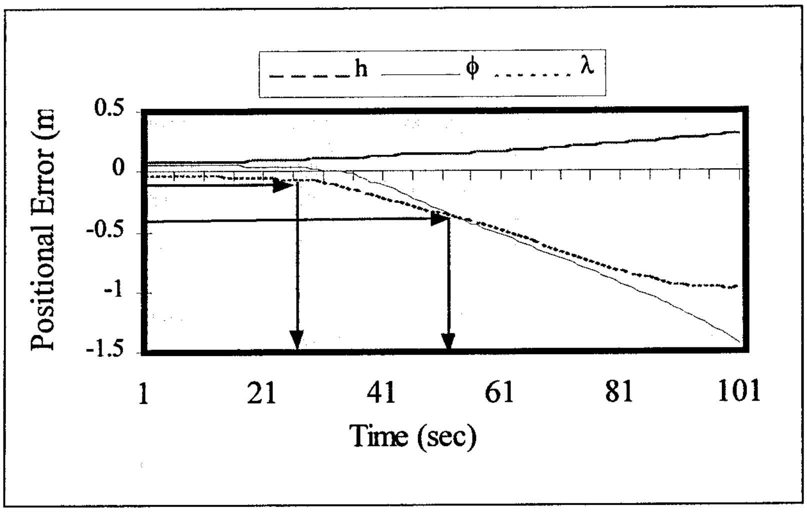


Figure 5: INS Error Behavior in Stand-alone Mode.

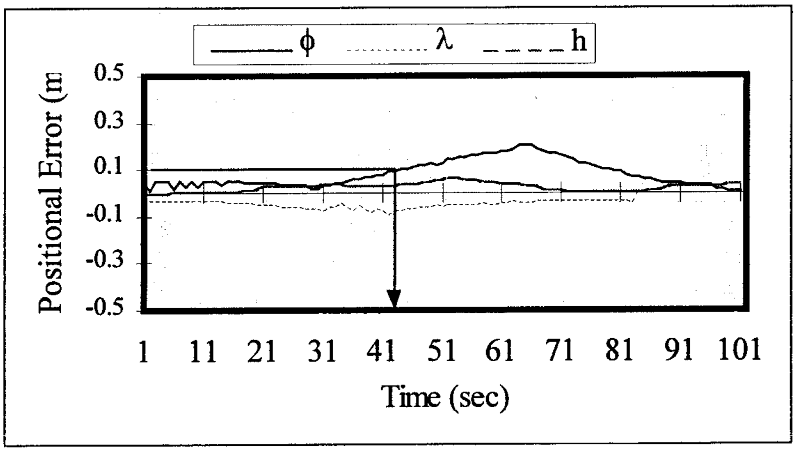


Figure 6: INS Error Behavior in Stand-alone Mode after Backward Smoothing

The results of the smoothing are plotted in Figure 6. It has been assumed that there is a GPS position update at the end of the 100 sec period of INS stand alone mode. The truth model of this diagram was obtained by using the trajectory computed from the original GPS/INS measurements. Results show that the smoothing process generally improves the accuracy and that the INS bridging interval has been extended to 42 seconds for L1. For wide-laning the positional errors stays below the half cycle level all the time.

5. DGPS/INS/CCD MObile mapping system

In this section, a GPS/INS/CCD Mobile Mapping System (MMS) e.g. VISAT (Figure 7) developed at The University of Calgary is described (El-Sheimy, 1996a). Mobile Mapping Systems (MMS) have become an emerging trend in mapping applications for fast and cost-effective data acquisition because they allow a task-oriented implementation of geodetic concepts at the measurement level.

VISAT consists of two major components especially important in a mobile mapping system: digital imaging and precise navigation. The navigation component of the VISAT system consists of two ASTECH Z12 GPS receivers and a Honeywell Laser-Ref III strapdown INS (1 nmi/h). The GPS receivers are operated in differential mode, with one antenna stationary on a known control point and the other mounted on the survey van. By using the carrier phase observable, the antenna center of the receiver is accurately positioned and its velocity is obtained as a time-dependent function. The imaging component of the VISAT system consists of eight digital cameras with a resolution of 640 x 480 pixels. The cameras are housed in a pressurized case and mounted inside two towers, which are attached to a fixed base on top of the van, thus eliminating differential motion of the cameras during the survey. Six of these cameras are arranged in such way that they provide a 230-degree horizontal and a 40-degree vertical field of view (FOV). The two other cameras are dedicated to imaging features such as power lines, requiring a different vertical FOV. All cameras are externally synchronized, so that every image record (8 images) refers to one position of the van.

Combining these two components, the concept of the georeferenced image as the basic photogrammetric unit emerges (El-Sheimy, 1996b). This means that each image is stamped with its georeferencing parameters, namely three positions and three orientations, and can be combined with any other georeferenced image of the same scene.

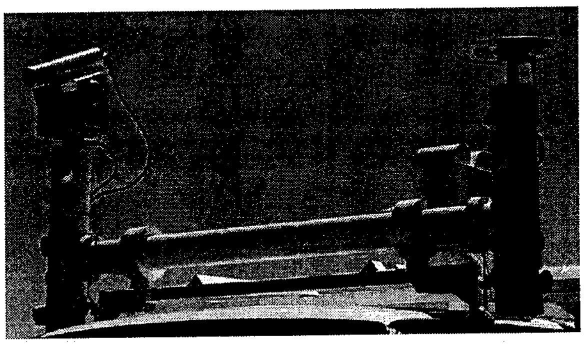


Figure 7: The VISAT hardware

The final accuracy of the 3-D coordinates for all objects within the video cameras' field of view is a function of the complete processing chain which involves GPS positions, INS position/attitude, target localization in the images, and system calibration. Figure 8 shows the errors in the computed 3-D coordinates of 14 control points located along one of the test sectors. The 3-D coordinates are computed from 2-images. The distance between the control points and the cameras was approximately 10-30 m. The figure shows clearly that an RMS of 16 cm in the horizontal coordinates and 7 cm in height are achievable for distances up to 30 m away from the van. The results in height indicate that the GPS/INS positioning component is working at the centimeter level. Since the height component in GPS is the weakest, it can be expected that the horizontal components are at least of the same accuracy. The increase in errors for the horizontal components must therefore be due to the camera array. The most likely explanation is that the increase in horizontal RMS as compared to height RMS is due to the along track error. Along track error could be due to imaging geometry (one pixel error in the x image coordinate could introduce about 40 cm for objects at 30 m from the van) and poor synchronization or constant offset error between the GPS/INS component and the camera component although unlikely.

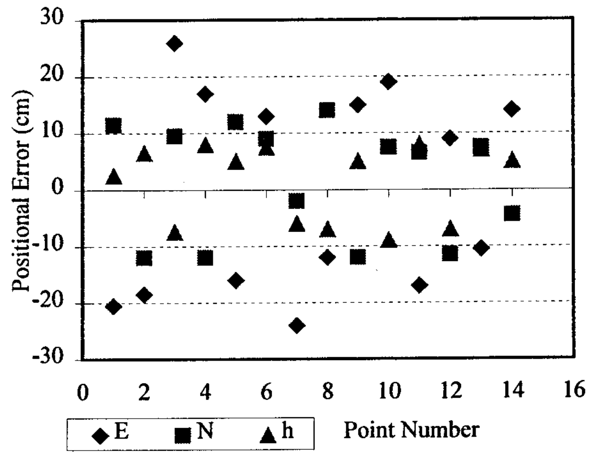


Figure 8. The VISAT System Positional Accuracy

6. SUMMARY

Several multi-sensor systems have been described in this paper including their system architecture and the obtainable positional accuracy. The analysis indicated that integrating GPS with other enabling sensors such as barometers, INS and CCD camera has enhanced the system's overall positioning performance. Although height component is the weakest component compared to the horizontal components using GPS, the multi-sensor systems described in this paper have provided satisfactory height solutions through system integration.

REFERENCES

Copley, D.W. (1994), The Application of Vibrating Cylinder Pressure Transducers to Non-Military Applications, Weston Aerospace Limited, Farnborough, Hampshire, UK.

Gao, Y., McLellan, J.F. and Schleppe, J.B. (1996). "An integrated DGPS/Barometry RTK System", GIM, Vol.10, No.12.

El-Sheimy, N., Schwarz, K.P., and Gravel, M (1995)., Mobile 3-D Positioning Using GPS/INS/Video Cameras, The Mobile Mapping Symposium, Ohio State, USA, May 24-26, 1995, pp. 236-249.

El-Sheimy , (1996a) ,A Mobile Multi-Sensor System For GIS Applications in Urban Centers, ISPRS'96, Commission II, WG 1, Vienna, Austria, July 9-19, 1996.

El-Sheimy , (1996b) , The development of VISAT: A Mobile Survey System for GIS Applications, Ph.D. Thesis, Department of Geomatics Engineering, the University of Calgary, Canada.

Schwarz, K. P., El-Sheimy, N., Liu, Z. (1994): Fixing GPS cycle Slips By INS/GPS: Methods and Experience, KIS94, Banff, Canada, September 1-2, 1994, pp. 265-275.

Schwarz, K.P (1995) : Integrated Airborne Navigation Systems for Photogrammetry, The Wichmann Verlag, ISBN 3-87907-277-9, Photogrammetric week'95, Stuttgart, pp. 139-153.

Wells, D.E., N. Beck, D. Delikaraoglou, A. Kleusberg, E.J. Krakiwsky, G. Lachapelle, R.B. Langley, M. Nakiboglu, K.P. Schwarz, J.M. Tranquilla and P. Vanicek (1986). Guide to GPS Positioning. Canadian GPS Association, Fredericton, N.B., Canada.

Height Control of Construction Plant by GPS and GLONASS

G. W. Roberts; A. H. Dodson; V. Ashkenazi

Institute of Engineering Surveying and Space Geodesy (IESSG), The University of Nottingham.
Tel: +44 (0) 115 951 3880, Fax: +44 (0) 115 951 3881, E-mail: iessg@nottingham.ac.uk

ABSTRACT

Construction plant control and guidance is a very hot topic. Already laser levelling has been used to enable automated construction plant height control. RTK GPS is an alternative method of controlling the blade's height in real time.

The paper details trials conducted by the IESSG whereby GPS and GPS/GLONASS receivers were situated upon a bulldozer, allowing kinematic carrier phase positioning of multiple locations upon the plant. The kinematic positioning was then compared to the on board laser guidance system as well as a digital level. The results show that precision in the order of a few millimetres is possible.

INTRODUCTION

Laser levels are frequently used to monitor and control the heighting of construction plant and, due to the continuous readings taken, automated heighting is possible. However, such systems only enable height control, with a crude distance measurement often provided by the use of a measuring wheel attached to the plant. The use of Real Time Kinematic (RTK) Global Positioning System (GPS) for plant control and monitoring has been investigated as a useful addition to the laser level system [Roberts, 1997]. Furthermore it may provide an alternative solution, offering both precise height and plan position information.

GPS technology is currently being used for applications in many diverse fields of civilian activity. Principal among these are geodesy, geophysics and general navigation. However, its impact on mainstream civil engineering has been limited. So far, the only applications of GPS in civil engineering have been for surveying, deformation monitoring and some setting-out operations, yet the potential of GPS as a measurement system for high precision three dimensional (3D) dynamic positioning and navigation is largely untapped. The development of reliable kinematic GPS technology has seen its inclusion in automated aircraft landing systems and vessel docking systems. Today RTK GPS is being used to aid the docking of a high speed ship (HSS) Catamaran ferry operated by Stena Line between Holyhead and Dun Laoghaire, seen in Figure 1. The adoption of GPS technology by the civil aviation and maritime communities has paved the way for developing similar on-line control and guidance systems for applications within the civil engineering industry. This technology can be used to provide the 3D position and control of, for example, the blade of earth moving equipment in real time.

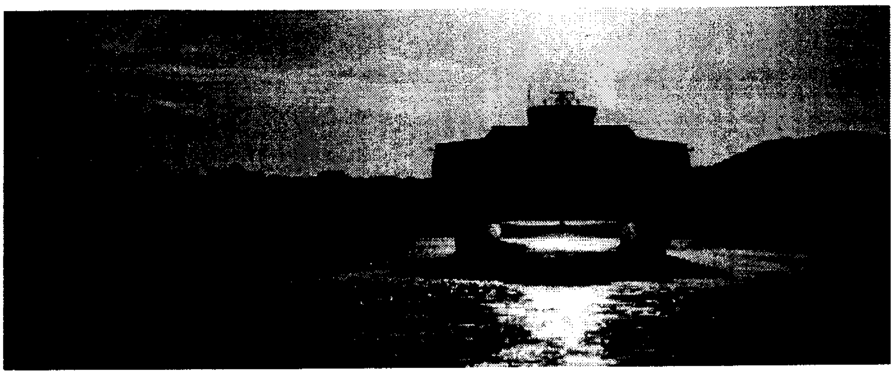


Figure 1, The Stena Line HSS 1500 Catamaran docking at Holyhead. The use of real time kinematic GPS allows this procedure to be accelerated

RTK GPS has been made possible through recent advances in GPS receiver technology and software/processing techniques and in particular, the increased measurement resolution of the GPS satellite signals. For example, a modern GPS receiver's carrier phase observable now typically has a resolution of the order of 1 mm or better [Ashkenazi *et al*, 1996]. Civilian dual frequency receivers are now capable of accessing both L1 and L2 pseudorange observables, even when the encryption of the precise codes is switched on. The US degradation of the signal is not a problem as this effect is differenced away through relative positioning. The major remaining constraint affecting the use of the carrier phase observable is that posed by *cycle slips*. Cycle slips occur when the satellite signals are obscured from the antenna for a period of time, no matter how short a period, and interfere with the continuity of the carrier phase data. It is of fundamental importance when positioning using carrier phase GPS that cycle slips are both detected and corrected by the processing software.

This paper outlines the recent advances in GPS technology and processing, which have enabled RTK GPS to be used for autonomous control operations within the construction industry. Trials have been carried out at the Institute of Engineering Surveying and Space Geodesy (IESSG), at the University of Nottingham, using the GPS satellites for guidance and control of earth moving machinery. The results demonstrate that precisions of the order of a few millimetres are possible. Comparing the results of GPS with results obtained independently using a laser-based control system and a digital automatic level substantiates these claims.

Laser Controlled Construction Plant

The use of conventional terrestrial surveying techniques, such as optical levelling, can provide accurate height control of construction plant operations. However, this is both a lengthy and labour intensive procedure, which may result in reduced production. A faster alternative exists in the use of laser levels.

The use of lasers in civil engineering operations is becoming common place [Uren and Price, 1997]. Two types are used, notably *fixed-beam* and *rotating-beam* lasers. The rotating-beam laser consists of a fixed beam laser fitted with a rotating pentaprism. The prism turns the beam through a right angle and spins it through 360° at speeds of rotation of typically either 300 or 600 rpm. A beam of light is produced which continuously sweeps across a site. These instruments are capable of generating both horizontal and sloping planes, which can be used to provide a height datum. Electronic laser receptors are subsequently used to detect the centre of the plane. Laser levels typically have a working radius of up to 300 m and an accuracy of better than 1 cm per 100 m.

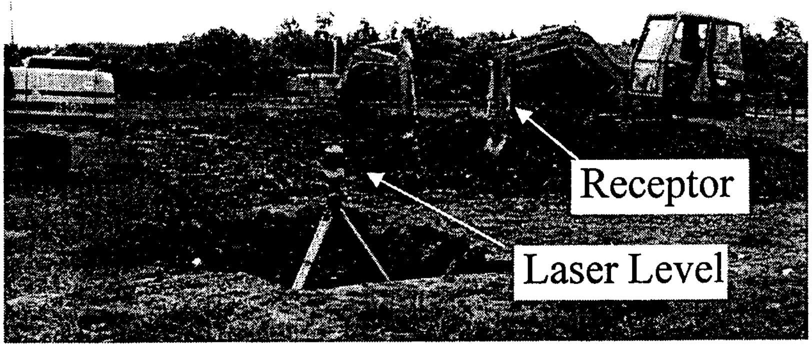


Figure 2, a laser level being used to assist with the excavation of a trench

Applications of laser levels include general site levelling, controlling the position of horizontal and vertical features and the control of construction plant.

A laser level is typically set up on top of a tripod, and once set level, it is possible to determine the relative heights of the laser receptors. The receptor may be used in a manual procedure, such as placing it on a conventional levelling staff. The receptor will make a sound when it is in the laser plane, allowing the operator to take the corresponding reading.

A further step to the manual technique is to place a laser receptor onto a piece of construction plant. Such an example is shown in Figure 2. Here the digger's arm is equipped with a laser receptor. The laser level is set up on a tripod, and the required depth of the trench related to the laser level calculated. As the arm digs deeper, the sensor seen in Figure 2 will eventually cross the laser plane. A signal is then given to the operator, telling him that the trench has reached the required depth.

Yet another step is to use the laser levelling data to automate the plant's operation. As an example, automated height control of construction plant is achieved by placing a laser receptor onto the blade of a bulldozer via a telescopic mast, Figure 3. The height of the blade is then adjusted until the laser plane is detected by the receptor. An alternative example for an asphalt paver is shown in Figure 4. Other control devices are also used such as ultrasonic devices which measure heights from the top of curbs adjacent to the road or from the top of string lines. Both the laser and ultrasonic systems are integrated into the machinery allowing automatic height control.

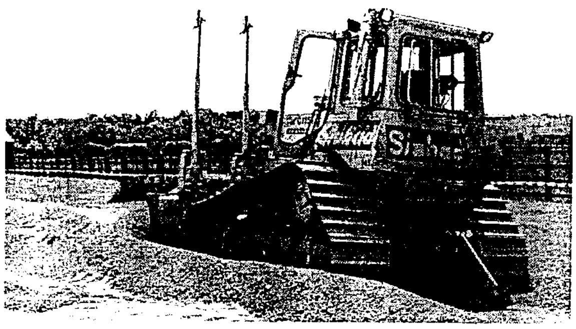


Figure 3, a bulldozer's blade being controlled by a laser levelling system

Figure 4, a laser level being used to control an asphalt paver

Further examples include the use of a laser system on a grader, thus allowing height and tilt control of the blade. This is seen in Figure 5, where a grader is equipped with two sensors, one on each side of the blade. Knowing the relative height of each side of the blade allows the tilt of the blade to be calculated. This type of system allows the driver to simply move his vehicle backwards and forwards whilst the blade's height is automatically adjusted to correspond with a pre-programmed level. The instrumentation within the drivers cab is very simple, whereby he is able to adjust the height of either side of the blade manually, being able to measure the change using the laser. Also, he is able to set a required height into the system, and the laser will adjust the hydraulics accordingly, keeping the blade at that height. Figure 5 illustrates the grader in operation as well as a paver in operation, both using the laser system. The laser itself may be seen on a tripod located in the background in between the two pieces of plant.

Figure 5, a laser levelling system being used to control the height of a grader and paving machine

Work has been conducted into the use of lasers to control vehicles' 3D position [Beliveau *et al*, 1996]. RTK GPS also lends itself to the task of automated heighting. The use of RTK GPS not only results in height measurements, similar to the laser level system but also horizontal position. This allows instantaneous quality control of the levelling to be achieved, since the plan coordinates of every level can be recorded, for checking later. The raw GPS data, which is used to compute the real time generated position, can also be recorded for post processing purposes.

Guidance from above

The IESSG, in conjunction with Sinbad Plant Hire Ltd, recently conducted feasibility trials using RTK GPS with an automated blade laser control system. Three *mobile* GPS receivers were attached to the bulldozer (Figure 6) and a fourth was used as the *reference* receiver, situated locally at a site whose coordinates had previously been determined to centimetre level. The GPS receivers were all low-powered Ashtech ZXII receivers. Two antennas were located on the blade itself using the laser level receptor masts, and the third placed on top of the operator's cab. During the trial, all four receivers collected data at a rate of 2Hz. The positions of the three *mobile* receivers were calculated using the RTK technique through using Ashtech commercial real time and post processing software, and in-house software developed at the IESSG.

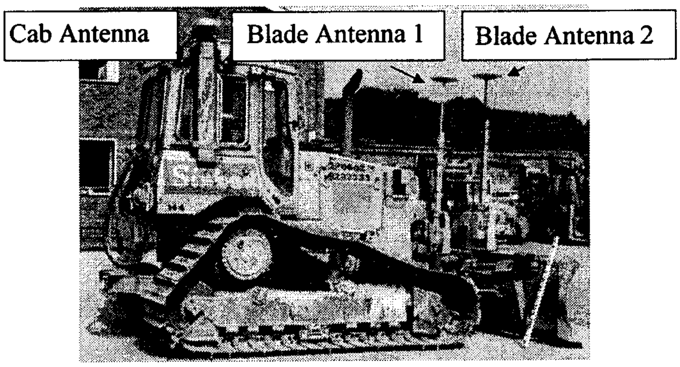


Figure 6, the GPS equipped bulldozer [Roberts, 1997]

In addition to the GPS receivers, the bulldozer was also equipped with a state-of-the-art laser-based on-line control system. The height of the blade, computed using the laser-based control system, provided the *ground truth* from which the performance of the GPS equipment could be assessed.

The objectives of the feasibility trials were to investigate the precision to which GPS could provide control in both height and plan and subsequently to compare the heights with laser and digital levelling instruments.

Height Precision

Figure 7 shows the RTK GPS heights of the three antennas located on the bulldozer. During the trial, the bulldozer was taken through a series of manoeuvres. Initially, it was at rest, indicated in Figure 7 as the period during which the heights of the three receivers remain constant. Following this initial static period, the bulldozer crawled forward down a slight gradient which can be identified in Figure 7 as the period (around 14:24) where the three line plots are seen to decrease at the same rate.

It can be seen from Figure 7 that at approximately 14:30 the blade antennas move relative to each other whilst the cab antenna remains almost at a constant height. During this period, the bulldozer's blade was tilted, altering the heights of both blade antennas. Similar manoeuvres are again seen at about 14:32 and 14:33.

It can clearly be seen from the Figure that the RTK GPS technique enables the monitoring of the movements of the blade and bulldozer to the order of a few millimetres. The presence and magnitude of the system vibration can be seen to be of a similar order, from the noise levels in the results during the static phase of the test.

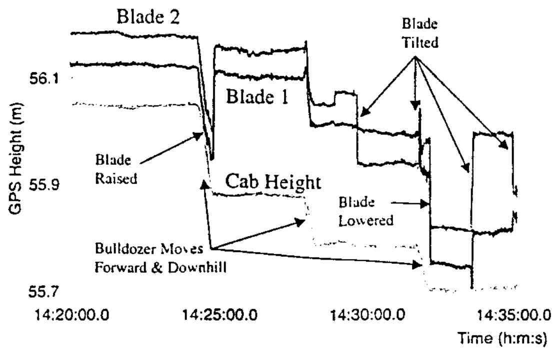


Figure 7, antenna height results [Roberts, 1997]

Validation of Results

RTK GPS has been shown to provide a plan precision of the order of ± 2 mm over a baseline length of about 1 km [Ashkenazi and Roberts, 1997]. Validation of the height precision was obtained during the above trials by comparing the GPS derived heights to those obtained from the on board laser heighting system and a digital level.

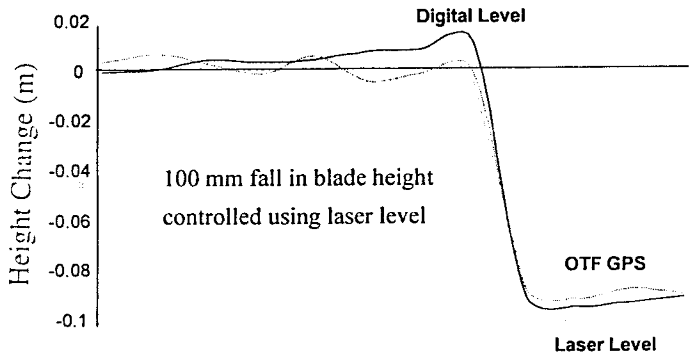


Figure 8, comparison of height changes using GPS, laser and digital levelling

A Leica NA2000 digital level was used to check the RTK GPS height. The digital levelling staff was placed upon the bulldozer's blade and levels taken at specific times. The digital level derived height was then compared to that derived from RTK GPS at the same epochs, in order to determine the GPS precision. Figure 8 illustrates the changes in digital levelling and GPS derived heights compared to the laser levelling.

From Figure 8, it can be seen that the agreement between the three systems is in the order of a few millimetres, with a maximum of 1.5 cm. The digital level agrees with the GPS more so than the laser system, especially true after the blade is lowered by 100 mm. This is thought to be due to the tilting affect of the blade, which slightly alters the height component discussed later.

GPS and GLONASS on Construction Plant

Further trials were conducted at the IESSG in which a single frequency Ashtech GG24 GPS/GLONASS receiver was located upon the plant in conjunction with dual frequency Z-XII receivers. Two Z-XII receivers were placed upon the bulldozer's blade and a third Z-XII and a GG24 upon the driver's cab. All receivers recorded data at a 5Hz frequency, with the reference receivers for both systems located closely. Again, the bulldozer was maneuvered, varying the blade's height under the control of the laser level system. All data was recorded to enable post processing, thus obtaining as high a precision as possible. The trial was conducted to identify any problems encountered using such a GPS/GLONASS system upon the plant. In addition, the trial was conducted to assess the precisions obtained by the GG24 upon the construction plant as well as to assess the comparison in speed of ambiguity resolution with the ZXII. Ideally, a combined GPS/GLONASS receiver would compare the coordinates obtained by the two satellite systems, thus enabling a quality control of the results. A combined system would enable plant control in a built up area to be less of a problem due to the increased number of satellites. The resulting GG24 results were acceptable. Hardly any loss of data was experienced. Two of the ZXII receivers, however, did not track satellites for a large part of the trial, thought to be due to the antenna cables being worn or damaged.

During the trial, the bulldozer remained static for 10 minutes. The engine remained switched on, introducing some vibration to the results. Due to the damaged antenna cables, data for only the GG24 and one of the ZXII receivers on the blade was available. It can be seen from Figure 9 and 10, a sample of the results, that again precisions of the order of ± 2 mm were obtained in plan for the 10-minute period. There is obviously a phenomenon experienced by both receivers at around 11:43, as the coordinate moves slightly.

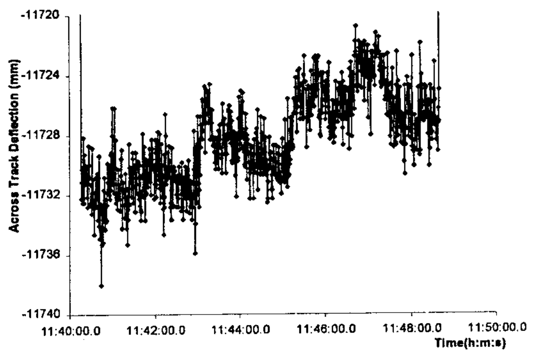


Fig 9, across track movement of the GG24 antenna located on the driver's cab

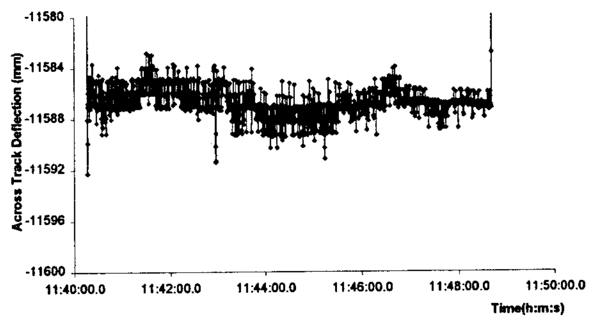


Fig 10, across track movement of the ZXII antenna located on the bulldozer's blade

The height component, however, is seen to be less precise than first expected. The GG24 and ZXII height components can be seen in Figures 11 and 12 respectively. Here the GG24 height is seen to vary by ± 15 mm and the ZXII by ± 25 mm. This, however, is thought to contain vibration noise of the running bulldozer's engine.

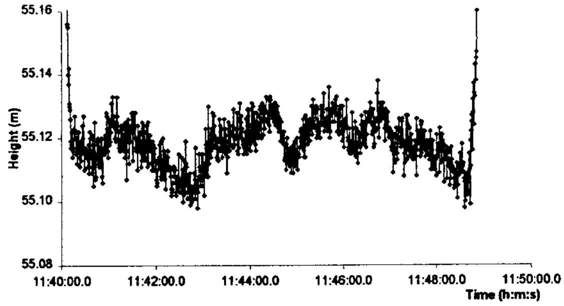


Fig 11, height movement of the GG24 antenna located on the driver's cab

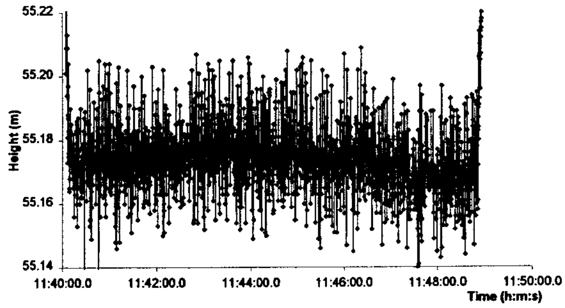


Fig 12, height movement of the ZXII antenna located on the bulldozer's blade

The height component of the GG24 receiver, located upon the driver's cab (Figure 11), is seen to be less noisy that that of the ZXII located upon the blade (Figure 12). This is clearly due to the vibration, as the blade would be expected to vibrate more than the cab. It is also obvious that there is true movement experienced in the results and that the antennas are not static. At 11:43 and 11:46 there is movement seen in both the GG24 results (Figures 9 and 11).

Further Results

Figures 13, 14 and 15 illustrate changes in height, distance along and distance across the track respectively during the same period of time for the same blade antenna. These illustrate instances when the blade's height was firstly lowered and then raised by two 100 mm increments but whilst the bulldozer was stationary. The 100 mm height adjustments were controlled by the laser-based system already operational on the bulldozer. It can be seen that during these manoeuvres, the GPS antenna, located approximately 3.33 metres above the pivot point of the blade, not only moved in height, but also in plan. This is due to the fact that the blade tilts back and sideways when raised. Errors can be introduced into the results due to this phenomenon, and have to be corrected in order to increase the precision. This phenomenon is thought to be the reason why the three height results disagree in Figure 8. It can be seen from figures 14 and 15 that vibrations of ± 1 cm are evident. These are due to the temporary pole attaching the GPS antenna to the blade being pliable. The vibration in the pole can be seen when the pole is in the process of being raised and immediately afterwards, as there is a whipping effect on the pole. The bulldozer's vibration is seen during the movement, although this may be partly caused by the laser level system, used to control the height during the trial, continuously adjusting the height. The results do show that plan precision is such that these small movements are detectable using GPS, but any future commercial system would need to address these issues.

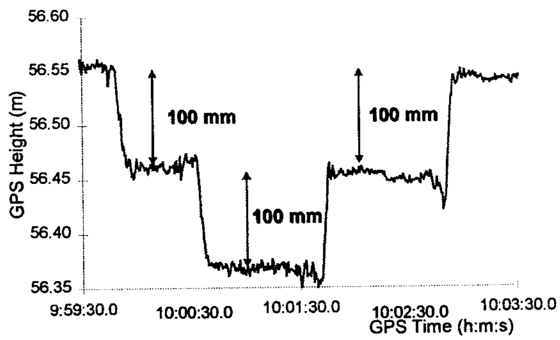


Figure 13, height plot during trial 1, during which the blade was moved by 100 mm height increments [Roberts, 1997]

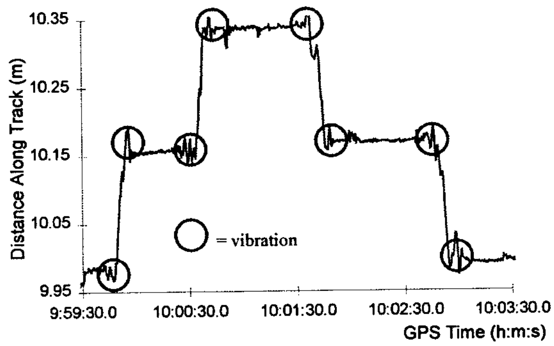


Figure 14, along track plot during trial 1, during which the blade was moved by 100 mm height increments [Roberts, 1997]

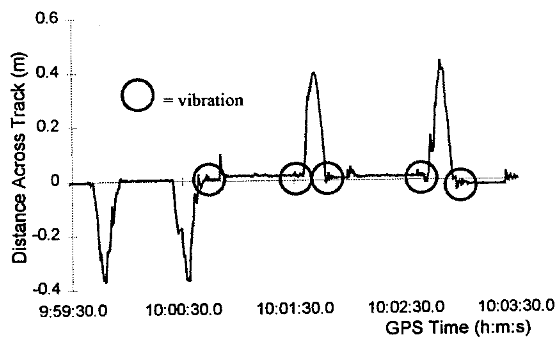


Figure 15, across track plot during trial 1, during which the blade was moved by 100 mm height increments [Roberts, 1997]

Conclusions

Many construction processes will benefit from the introduction of automated operations, such as laser guidance and GPS aided control. These include repetitive and labour intensive operations (e.g. grading and compacting), precision operations (e.g. piling and excavating) and operations in hazardous and hostile environments (e.g. decommissioning sites). The trials are a first stage in the development of a fully automated system. The initial trials, performed and analysed at the IESSG, have successfully demonstrated that RTK GPS for control systems on board construction plant can provide coordinate precisions of a few millimetres, when a reference receiver is located close by (<1 km) at a point of well-defined coordinates.

The real time communication aspect of this system is the main limitation, as the UHF telemetry link requires line of sight. Alternative data communications, e.g. the proposed use of the cellular phone network, may overcome this problem and prove a valuable tool for the successful use of RTK GPS over baseline lengths of tens of kilometres.

A GPS-based system offers the potential to provide continuous 3D control, irrespective of the terrain, over large areas and at the same level of centimetre accuracy as current state-of-the-art laser systems, with the only limitation being that of line of sight to the satellites. This is a major area of research at the IESSG. In particular research is focused on the combined use of GPS with the Russian "Global'naya Navigatsionnaya Sputnikovaya Sistema" (GLONASS) system. Combining the two constellations potentially doubles the number of satellites available with which to position and, hence reduces the likelihood of a shortage of satellites for the RTK positioning. It also provides integrity to the results by obtaining two sets of coordinates from two different satellite systems, which one would hope to agree with each other.

Further enhancements would include the integration of the RTK GPS results with other sensors such as tilt sensors, attitude sensors, the hydraulics of the machine and the laser system itself.

Acknowledgements

The field trials, which led to this paper, were carried out as part of a much wider research project, sponsored by the EPSRC (The UK Engineering and Physical Sciences Research Council) on "Real Time Positioning Using GPS for Civil Engineering".

The authors are grateful to Messrs Sinbad Plant Hire Ltd of Stapleford for providing the construction and laser equipment to carry out the field trials. Thanks are also due to Stephen Simmonds, of Sinbad Plant Hire Ltd, for his enthusiastic help and to Roghailan Al-Shammari and Ben Sansome, MSc students at the IESSG, who helped with the fieldwork and the analysis of the results.

REFERENCES

Ashkenazi V. Dodson A. H. Moore T. Roberts G. W., 1996, Real Time Monitoring of the Humber Bridge, Surveying World May/June 1996. Vil 4 Iss 4. ISSN 0927-7900

Ashkenazi V. and Roberts G. W., 1997, Experimental Monitoring of the Humber Bridge Using GPS, Proceedings of the Institution of Civil Engineers - Civil Engineering, Volume 120 Issue 4, November 1997. ISSN 0965 089 X. pp 177 - 182. Printed and bound in Great Britain by BPC Magazines Ltd, Milton Keynes.

Roberts. G. W., 1997, Real Time On-The-Fly Kinematic GPS, PhD Thesis, University of Nottingham.

Uren, J., and Price, W. F. 1997, Lasers in Construction - the Evolution Continues. Proceedings of the Institution of Civil Engineers - Civil Engineering, Volume 120 Issue 1, February 1997. ISSN 0965 089 X. pp 27 - 35. Printed and bound in Great Britain by BPC Magazines Ltd, Milton Keynes.

Beliveau, Y. J., Fithian, E. E. and Deisenroth, M. P. 1996, Autonomous vehicle navigation with real-time 3D laser based positioning for construction. Automation in Construction, Volume 5 Number 4 October 1996. ISSN 0926-5805. PP 261 – 272. Elsevier Science BV.

Some questions of the precise height measurement

Dr. Szabolcs Csepregi

College for Surveying and Land Management

1-3. Pirosalma street, P.O.B. 52. 8002 Szekesfehervar, Hungary

Fax: +36 22 327697

1. Polygon around Szekesfehervar

The works on developing the crustal movement levelling network in Hungary started at the beginning of the 1970s. The works on precise height measurement started also at this time in our college. We built up a levelling cornice polygon around the town Szekesfehervar and in the municipal area in the year 1973. Actually the exercise was to analyse the concept of Vincze Vilmos in practise, that the levelling line directed across the town is weaker than a line directed in two different directions around the town.

Proving it look as Fig.1. The line leading across the town is the diameter of the circle. The length of it is 2R. The half circle keeping away from the town means πR long line. The reliability of the line measured along the diameter $m_d = m_o\sqrt{2R}$.. The reliability of the line directed outside the town along the half circle $m_{halfcircle} = m_o\sqrt{\pi R}$. The reliability of the point, which is determined as the average value of the two half circles: $m_{circle} = m_o\sqrt{\pi R/2}$.. The quotient of the two values is $\sqrt{\pi/4} = 0,89$, it means, that the lines directed outside the town secure 10% better accuracy.

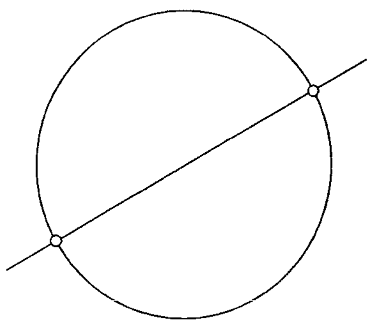


Fig.1.

Further advantage of the solution is, that we have a chance to build up a new town-network which is necessary inside the city. Another advantage that we can produce the more precise measurements outside the town than inside.

In the first year we measured the levelling circle which is situated outside Szekesfehervar. In the second year we measured two diameters. The levelling lines were leading across different kind of fields. The national first and second order lines are running along roadways. Our lines were built out along dirt roads.

We used Wild N3 1 cm spacing instrument and Wild invar levelling rod. The maximum distance between the instrument and the levelling rod was estimated in 40 m-s according to the national standards of that time. The measuring time for one station was 5 minutes in the average. The levelling rods were calibrated before the measuring and after the measuring. The measurements were done strictly at the convenient time in the small hours of the morning and in the evening. The standard deviation of the levelling was 0.3 mm. Analysing the standard deviations of the measurements we estimated that the standard error of 1 km line depends on the circumstances of the measurement. We got the most suitable result on those lines, which were carrying a little deal of traffic. Measurements along roads which were carrying a great deal of traffic produced worse results with 0.35-0.40 mm standard error in one km. We got the same kind of results inside the town when measuring along the thoroughfare of the town.

2. LeveLling through the river Tisza

Continuing the work we developed a same kind of cornice polygon around Szeged. As a part of the exercise we were levelling across the river Tisza at Szeged. The width of the river is approximately 300 meters. We were levelling with the help of a home-made table. This table was similar to the table used in Hungary earlier. There were two striped graduations, both of them 2 cm wide with white and black stripes. The second graduation was painted just like the first one, but it was moved 1 cm away from that. During the measurement we used a 1 cm micrometer spacing instrument so one graduation either on the left or the right side could be intercepted with the wedge of the instrument. During the measurement we could notice the disadvantages of this table. The scales were essentially 2 cm wide stripes by 1 cm moving on the other side of the table. The disadvantage of this was that we had to intercept a line which continued with a 1 cm moving away. This was really hard to intercept because we didn't get a symmetrical picture. And ruined the precision of the measurement because we shouldn't carry a symmetrical picture into the middle. Another influence of this was, that we had to pay special attention for the coarse reading.

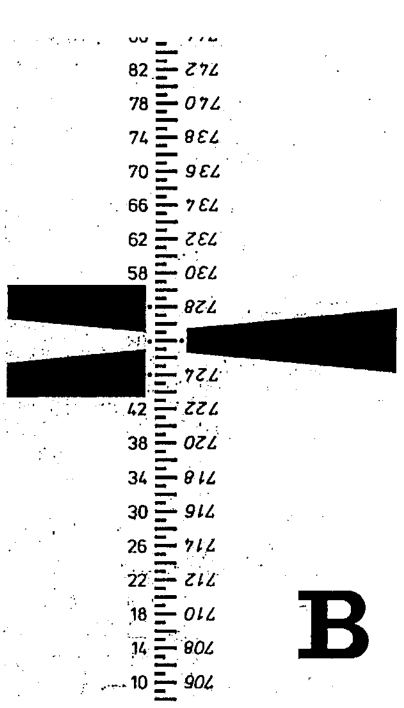


Fig. 2.

We created a new „reciprocal-levelling" table drawing a lesion from the cross-levelling at Szeged and a new method for levelling on longer distances. There is only one big graduation (Fig. 2.) This wedge shaped dividing has to be intercepted the way we do it when levelling. There are a lot of options about the shapes of the levelling scales, and there are many kind of graduations on the different rods. These can be angled shaped, rounded shaped, trapezoid shaped. We created a trapezoid shaped graduation. We created half cm graduation like the original levelling rods, but these are invisible from higher distances. The graduations were counted differently according to the cm and half-cm instruments.

The adopted the following procedure: levelling rods were put on stone benchmark which had been marked earlier. The back side of the rods were furnished with tables that were moveable up and down. At the beginning of the measurement, after setting up the instruments the tables have been set in order to be able to read suitably using the micrometer of the instrument. After setting the table in the position we red the rods. This means, that first we red on the graduation of the table, and after turning the rod we red on the rod graduation. The next step was actually the "levelling-across". This time we red on the graduation of the table that was put on the other side of the bank. This observation could be done well we could intercept the single graduation on the other side of the river. After the measurement we did the rod measure again. We managed to level across the river with the accuracy of 0.2-0.3 mm according to the experiences. We tried out the table around the lake as a subject of a diploma work. This time we could compare the results of the levelling across the lake to the results of another line levelling around the lake.

3. Bodajk Csokako crustal levelling line

A levelling line was built out in 1992 between Bodajk and Csókakő. This line crosses an existing geological fault line.

Levelling lines in Hungary are usually directed along roads. Roads are many times built next to lines of faults so levelling lines rarely crosses geological fault lines. The length of the completed line is 12 km and it is crossing the fault line called Mor graben.

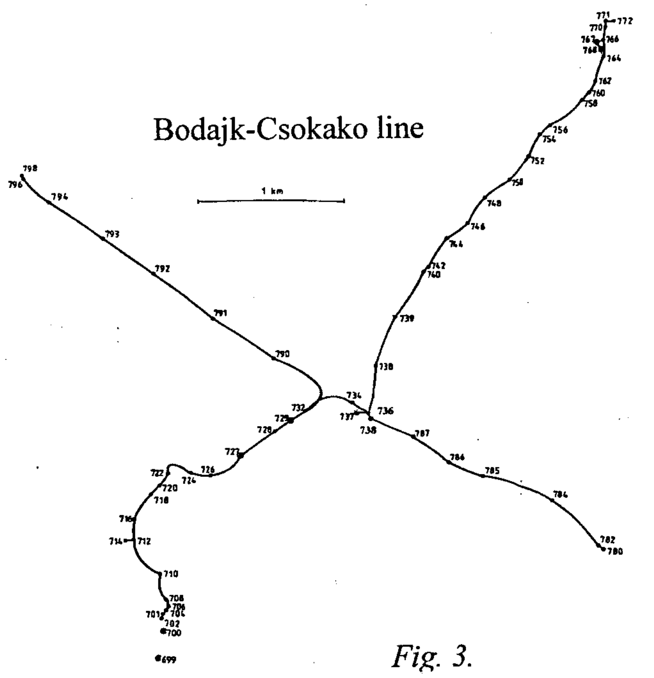


Fig. 3.

On the eastern side of the fault you can find the Vertes mountain, and the western side is surrounded by the mountains of the Bakony. The road 81 has been built along the fault line. Move-ment analysing standpoints were primarily taken into account when building out the lines. The endpoint of the line was placed at Bodajk in the side of the Kalvaria Hill in a concrete block. At the endpoint of Csókakő three rivet benchmarks were used. The intermediate points of the line were placed closer to each other than they are placed in the national network. The distances between the intermediate points are under 300 meters in the average.

We wanted to achieve more precise determination of the movements by placing more points into the network. The points were primary placed into the walls of buildings built directly on rock, but we always selected rather old buildings that had been finished for many years. The points have been built in culverts and in other concrete buildings. Many times the marking was made of stone.

Movements between 1991-1992 and 1991-1993

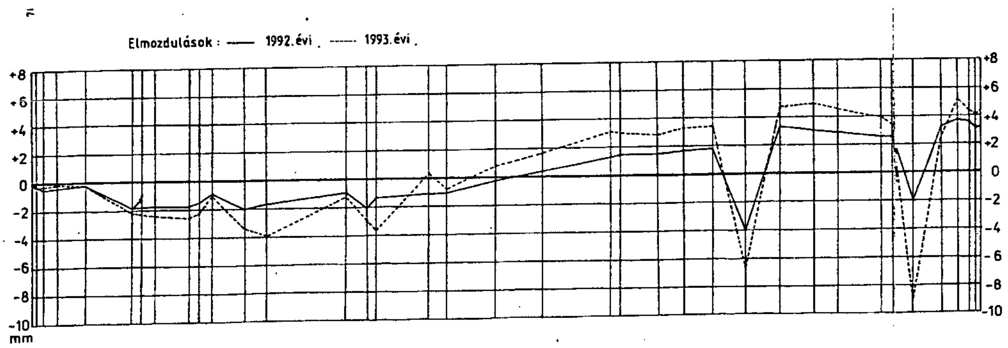


Fig 4.

The measurements were achieved by the Hungarian Optical Works (MOM) Ni A 31 precise level, but we used Zeiss Ni 002 instrument too. The longest distance between instruments and rods were 30 meters. But many times decreased this distance to 10-15 meters on the steep rising sections. The results of the measurements were directly stored in a hand calculator. The data storing program made us possible to store different kind of data that we hadn't stored before. Measurements were processed further on computers. The first program produced field sheet following the traditional noteform. The second program put the line together. This made possible to skip the wrong measurement results.

Movements: 1991-92 and 1991-93

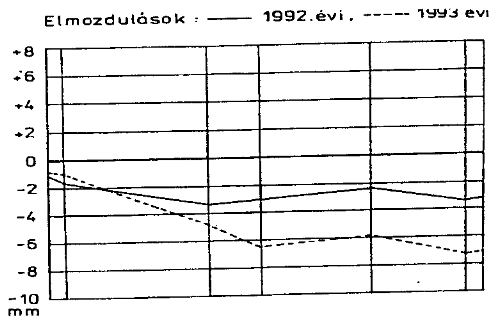


Fig 5.

The measurements were carried out by experienced teachers during high school exercise. The students had to exercise to hold the levelling rods and handle the computers. One levelling section was measured more times during the exercise. Besides the usual indexes there has been calculated the average measuring time of one station, the value of the average instrument sinking, and the standard deviation of the measurement of one station. We calculated the standard deviation for one km section of course. The modern processing system made us possible to calculate more indexes and results than in traditional way.

We managed to obtain a Leica NA 3000 digital levelling instrument to make our work easier After working out the measuring program, this instrument has been put into use. According to experiences the speed of the measurement rised. The precision of it wasn't weaker than the precision of the former optical instrument. By reason of 3 years measurements, the movements of the two mountains occured continuously. The movements up till now prove this. According to Fig. 4., 5. Csokako is rising 2-3 mm, Sored is sinking 2-3 mm per year compare to Bodajk. The most important features of the levelling:

measuring time of one station in the average	3.2 min
S.dev. for one instrument:	0.14 mm
average instrument sinking:	-0.003 mm
S.dev. of levelling per km :	0.27 mm

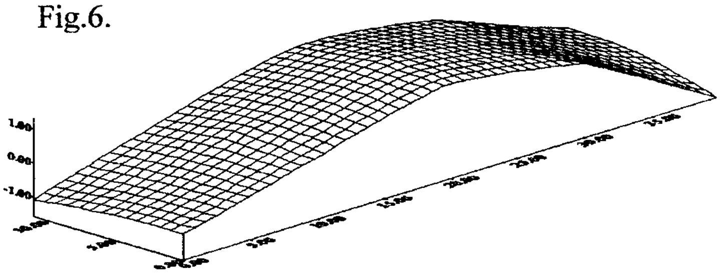
We hope it will be possible to continue the measuring.

4. Deformations of buildings

There were built a lot of housing estates at Szekesfehervar and in Hungary in the 1970-s and 80-s. The upper layer of the ground of Szekesfehervar is a 3-4 meters thick gray slit ground. Ten-stored houses need to be supplied by special foundations. Earlier the territory was flooded with water in order to protect the fortress of Szekesfehervar. The movements of all ten-stored houses had to be analysed by height measuring. Analysing was achieved by repeated precise measuring on rivets that were fixed on the buildings. In order to determine the deformation of the foundation base we also fixed rivets inside of the building. The sinking of the building was determined by repeated precise levelling. During the building operations the measurements were repeated in 4-7 days. By the time a building was completed there had been 8-10 measurements done.

In the following sentences we'd like to introduce the experience of measuring one building. The analysed building is 60 meters long and 11.6 meters wide. It consists of three different blocks that are only once separated by dilatation gap. The blocks were not built at the same time. The prescriptions were neglected. By the time the block 2 and 3 started to be built the third level of the block 3 had already been completed.

Fig.6.



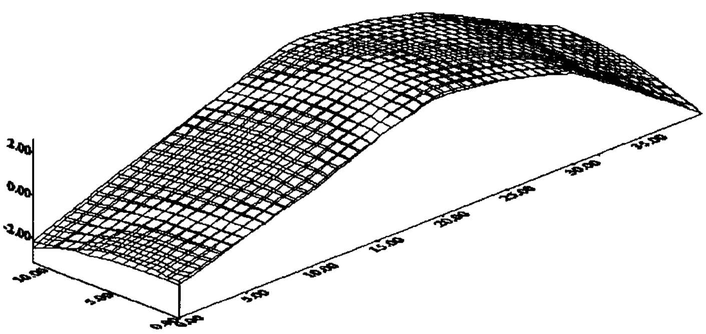
The foundation of the building has been put on 97 pieces of 1.4 meter diametrial long and 7.5 meter deep borings. The ground floor was made of monolit ferro-concrete. The fitting work of the prefabricated panels started from the beginning of the second level.

It is important, that the examination network of the building shall be an independent one and this network shall be connected to the motionless reference points when determining the movement of the building. In the examination network the movements of the single points show the deformation of the foundation base of the building. According to our experiences: measurements can be done more precisely between examination points of building because we don't need base plane points. The reference points are situated rather far away from the building and it demands to create more turning points.

From the point of view of stability of the building the sinking of it is not so much important. Maybe it is sinking but it doesn't ruin the foundation base of the building.

The stability of the building is determined by its leaning and by the deformation of the foundation base. These were that we wanted to determine by the analysing. During the processing we determined the height of all points and the experienced movements. Using the deformation results we calculated the leaning and the deformation of the foundation base by taking an adjusting plain.

Fig. 7.



To determine the balancing plain we used the analytic equation of the plain

$$(Ax + By + Cz + D = 0)$$

as observation equation. For the determination we considered only the measured z movements as wrong values, and this way we wrote the

$$-v = ax + by + d + z$$

observation equation.

After the measurements completing the adjustment we experienced that the sum of residual squares (Σv^2) was groving continuously and so did the corrections. Analysing the corrections we can display regularities and this is why we constructed the isoline map of the corrections (Fig. 6., 7.). This shows expressively a continuously developing deformation of the foundation base of the building which may be dangerous for it. The continuously rising tilting values determined by the adjustment showed well the movement of the building. It is not dangerous for the building under special limits. The degree of the deformation is signed by the regular increase of the corrections, and this may damage the structure of the building.

5. Computations of torsion surfaces

The movement analysing networks are usually created in this way: after creating a network, every height of the single points are going to be determined. Next time a new network will be created and so we can draw a conclusion from the changing of the heights of the points. The national networks were originally not established for crustal movement analysis. Only later they decided to use them for crustal movement examination. When developing the national levelling network none of the lines crossed the crustal movement fault lines. They often lead parallel with them because the roads are also parallel with them.

When developing a movement analysis network the lines should cross the fault lines in order to determine the movements.

The computations were also developed so that they shall supply creating independent networks. These methods didn't take into account that the standpoints of the movement analysis. The originally measured height differences were considered as measurement results, only when engeniers determined the network. When calculating the movements the changes of the heights have already been considered as measurement results.

The first aim of the movement analysis is to determine the movements so we have to take this into account. Let's consider the points are moving when we are adjusting the network. Let's write the parameter of the height of a point that is moving at a standard v speed. This time the observation equation of the height difference:

$$v = M_E + tv_E - M_S - tv_S - \Delta m, \quad where$$

$M_S ; M_E$:	height of the standpoint ; height of the endpoint
t:	time counted from the beginning of the basetime
$v_S ; v_E$:	speed of the startpoint; speed of the endpoint
Δm:	measured height difference.

The measurements on larger areas take many years. Using this solution we can take the time difference into account. We can complete the computations with local measurements that are in connection only with some points or with little areas.

The other solution is if we give the function of the speed of the points in a form like this: $v = f(x,y)$, where $x; y$ are coordinates of the points. Conclusion: We can write the next observation equation:

$$h_{12}/t = f(x_2,y_2) - f(x_1,y_1)$$

where (h_{12}) is the height difference changing between 1 and 2 points.

We express the (h_{12}) changing of the height difference as a function of the $f(x,y)$, *where* $(x_2;y_2)$ and $(x_1;y_1)$ are the coordinates of the start and the end points.

It's important how we define the equation of the moving surface. If we choose a simple equation this will not show the real movements by a suitable precision. If the equation is a little bit elaborated it will be difficult to solve. These are the reasons why we are allowed to use this solution only in the case of simple movements.

By mentioning the questions of computation I wanted to point out the importance of original value which is considered to be the basic data for network analysing. The method of the analysing also depends on it.

Error sources in high precise levelling

-How to minimise their effects on the heights

By

Michel Kasser (ESGT, France) and Jean-Marie Becker (NLS, Sweden)

High precise levelling techniques are basically extremely simple, and easy to understand. But they present two specific aspects that are without any equivalent among any other topographic/geodetic/surveying operations: (I) their perform extremely *high accuracy*, and (ii) they are based on *reiterated and repeated operations* so that the smallest systematic error should be well known and carefully handled, and (iii) contrary to many other geodetic processes, levelling is *poorly overdetermined* so that the internal consistency (loop misclosures) is not a very efficient indication about the quality of measurements.

The errors that affect the results of levelling (the heights of the benchmarks) are generated during the following production steps:

⇒ Network configuration

⇒ Benchmarks

⇒ Instrumentation + Equipment

⇒ Levelling procedures

⇒ The operators

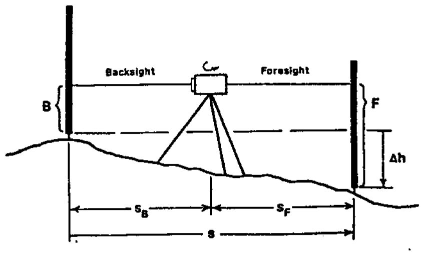
⇒ External factors

⇒ Calculation + Adjustment

Each one of these steps contributes to the *final error-budget* with its specific errors coming from different error sources. We present here a synthesis about some errors in levelling

1. INSTRUMENTAL ERRORS IN LEVELLING

The basic theoretical figure of high precision levelling is extremely simple (see figure below):



—Setup of leveling, $\Delta h = B - F$ and $s = s_B + s_F$

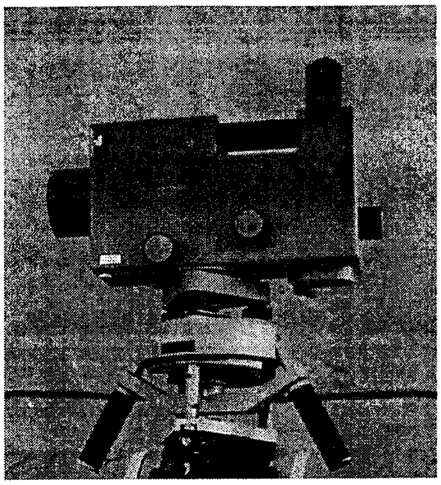
- The equipment (the level) provides an optical axis that is perfectly horizontal,
- The observer reads perfectly the graduations on the rod in perfectly vertical position, rod whose origin is exactly the lowest end formed by a base-plate perfectly perpendicular to the rod length, and whose graduations are perfectly known (e. g. each centimetre).
- During the observations, the backward and the forward rods are in vertical positions over the two different benchmarks
- The observer reads on the backward rod, then he knows what is the elevation of his optical axis relatively to the backward benchmark, thereafter he reads on the forward rod, these operations provides the elevation of the forward benchmark relatively to the backward one. During these observations, the benchmarks and the level are supposed to be stable and not submitted to any vertical movement, and the instrumental optical axis gives an altitude completely independent from the direction of observation and in a horizontal plan.

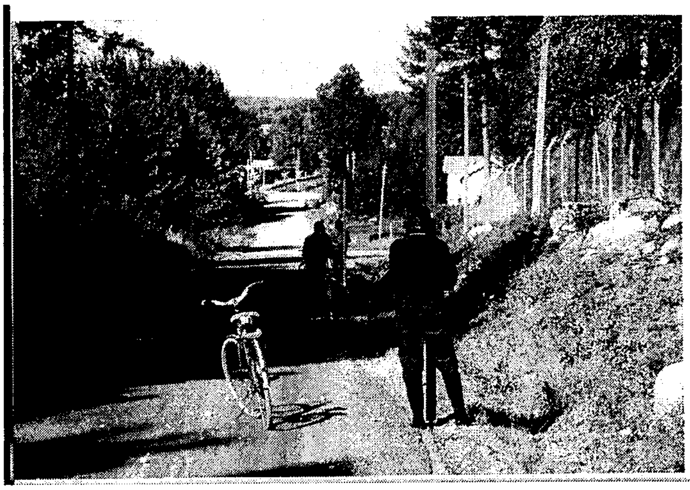
Of course, the perfection is not accessible in our world, and we shall analyse the instrumental problems that arise from the necessary imperfections within this figure. But as most of these points are very classical for surveyors, we shall point out the aspects that are not well documented.

1.1 The level

In the level, a mobile part of the optics is designed so that the direction of the optical axis is independent from the position of the whole box of the level, within given limits. In order to damp correctly this mobile part, and considering the possible movements of a level during its lifetime, the only possibility is to use the air itself to perform this damping. Ribbons, instead of wires, must guide the mobile part, as ribbons may be very flexible in one direction and keep a good rigidity in the other one. This feature is necessary for the guidance of the damper, where any mechanical contact between mobile and fixed parts is prohibited. The damping movement requires that these two parts move inside a free space less than 1mm due to the extreme fluidity of the air.

When the ribbons have been folds in one direction, they keep a small residual flexion, so that the equilibrium of the moving part is slightly different from its theoretical position: also called "*compensator residual error*". It is recommended to limit the systematic effect at the set ups by levelling the instrument alternatively pointing in different direction (Forward and Backward) according to the "red pantaloon " method.





Of course, the optical axis is not perfectly horizontal, but it is expected to describe a quite flat cone whose axis is perfectly vertical, and as long as the two rods are at the same distance from the level, these collimation errors cancel.

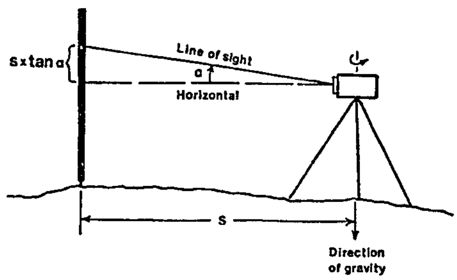
Only the Zeiss Jena Ni 002 with its two symmetric compensator positions have been able to provide a fully collimation free line of sight (see figure above left), correcting quite well these aspects.

It has been shown that even the *magnetic fields* from the earth and also from power lines or industrial installations as well were responsible of some asymmetric effects on the compensator position of some automatic levels due to the use of magnetic materials (like iron screws) in their mobile part. The earth magnetic field has its maximum systematic effect in the North-South levelling lines and is minimum in East-West lines. Today most of the automatic levels seem to be free of magnetic materials, but *any new instrument should be certified regarding this sort of behaviour.*

1.2 Specific problems for digital levels

These levels provide a much more easy and fast operation than classical automatic ones. The risk of blunders has disappeared, thus the operations may be very simple without danger: one reading backward, one forward, which allows an efficient work. But new problems arise:

- The ***illumination of the rod*** must be kept at a much higher level than necessary for the human eye, and *differences in illumination* between rods or on one rod can create systematic errors (perhaps it is the case for human eye too, but it has not been proved yet).



–Effect of collimation error, α.

- ***Shadows*** on the rod can create strong operational difficulties and in some case stop all levelling operations.
- The problems *of **refraction*** are quite different for the different type of fabrications (manufacturers), and often larger, than with human eye observations (see "refraction" paragraph). For example also when the optical axis physically is very close to the ground or even under the rod base-plate (pointing in the ground) the instrument still automatically registered observations. These are fully affected by the refraction effects that are

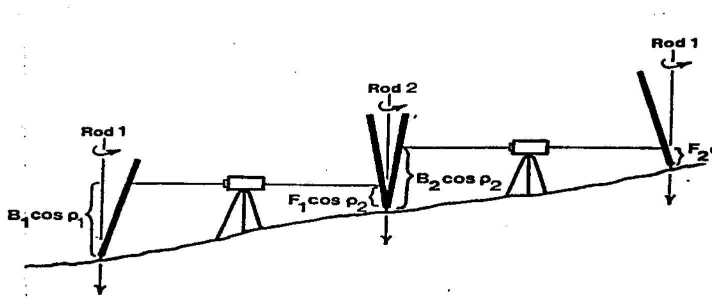
extremely strong in the low air layers.

- The instrument adjustments for collimation are generally very sensible for ***temperature variations*** and can create important systematic errors. For high precise levelling it is recommended to check the collimation each time that the air temperature changes from more than 5° C.

1.3 The rods or staffs

Verticality of the rods

The rods should be perfectly vertical. If inclined with an angle ρ, it would imply that the height difference would be affected with units



—Effect of plumbing error (and scale errors) accumulates on sloping terrain, since $B_1 \neq F_2$ and $F_1 \neq B_2$.

of 1/cosρ cm instead of 1 cm.

We recommend that several circular eye-bubbles (2 or 3) be mounted on each staff instead of one who never alone can guarantee the verticality of the staff. The staffmen can continuously verify that all bubbles are inside their circle, notice a discrepancy between them and adjust them. A small mirror at 45° over the bubbles can avoid optical parallax when checking the centring of the bubble.

We insist on the fact that the loop misclosure will not indicate any large value for a systematic non-vertical rod especially in flat terrain.

Scale of the rod

The requested precision for the graduation is directly correlated with the specifications of the levelling operation. High precision (better than 4 mm.km$^{-1/2}$) is generally impossible with non-invar foldable rods. As soon as the scale of an invar rod receives a shock, they must be checked. A major problem appeared in 1991 with the new digital levels, whose rods (with bar code) cannot be tested on the classical existing interferometric installations. Today only few laboratories can made such a calibration and that also is very expensive to perform. As a result, the surveyors have been driven into a complete dependency to the equipment manufacturers, and they often consider now that any checking of the rods create problems and has to be avoided. *We urge the Universities, the National Institutions in charge of levelling networks or in charge of the metrology to offer services of invar-rod checking at a reasonable cost.*

Base plate of the rod

It must be perfectly flat and horizontal when the rod is vertical. If it is not the case, an error will arise if different parts of the base plate are used, a "*centre ring* "may be used to avoid these errors (several 1/10 of mm). At the connections to some benchmarks especially on walls, bridges it can be impossible to use these centre ring and special attention has to be paid on the stationing of the rod.

To test the base plate is very easy, the surveyor can do it by comparing the readings from the rod positioning (5) over a fixed point (bolt) using the four corners of the base plate and at its centre. The discrepancies should not be in excess of 0.1 mm for high precise levelling without centre ring.

Rod calibration

The temperature and the humidity (for rods with a wooden frame) are important factors for the rod calibration. For high precise levelling it can be useful to register the temperature values from the invar during the field operations. Laboratory calibrations are still exceptions and not representative for the various field conditions existing during the levelling operations. We consider that an effort should be performed by laboratories providing calibration facilities, so that the calibration may *be done in conditions close to the field ones*, and not only with a fixed temperature and humidity.

2. USE OF THE EQUIPMENT UNDER FIELD CONDITIONS

Long field experience has shown that many different factors can introduce errors in the results of levelling operations and therefore it is good to remain some of these external and operational factors:

- topography or terrain
- meteorological: refraction + temperature + humidity
- vegetation: forest, corn, grass, etc
- proximity of water, smoke
- underground: asphalt, gravel, etc
- traffic, circulation
- magnetic fields
- vibrations

We shall not deal with all of these factors but mainly with the specific aspects of the behaviour of the ground, and of the surveying procedures.

2.1 The ground surface

The common sense experience is of no use to anticipate the very infinitesimal movements of the ground. When analysed with the proper instrumentation, any concrete moves even under the pressure of a finger, any non-crystalline material flows, the wind in a tree even at more than 10 metres deforms the ground, which also affect the level.

The same situation occurs with the ***surveyor moving around the tripod or the rod*** from one position to another one.

The rod at its stations, often on temporary pivots, moves certainly also during the set up time. If the levelling line follows *summertime* a road with a macadam surface the pivot (with the rod over) will regularly sink during the set up time (high temperatures). In contrary at cold *wintertime* there will be more or less a rebound due to the elasticity of this material.

In order to minimise such systematic disturbing (settlement) movements, the temporary pivot and the tripod must be understood as floating on the ground, with the largest possible interaction area. A large spherical surface with a nail in the centre to avoid any slip, or multiple diamond shaped flat surfaces are recommended for the rod support. The same considerations are pertinent for the tripod feet's, where large contact surfaces provide an excellent answer to the problems due to the movements (consistence) of the ground during observations.

If precision levelling was not so precise, these difficulties would never have appeared, but we have now the possibility to observe far under the 0.1 mm level, and then we may see how are generated all these small movements, that in turn will be responsible of systematic errors.

2.2 Field procedures

The usual measuring proceedings in high precise levelling have different origins. Some field procedures aim at removing any possibility of unrecoverable blunders, some tries to minimise the ground movements. All of them consider that refraction is unavoidable. We can distinguish several of them as follow:

- detection and elimination of the errors effects *directly at the source*: equipment , field procedures
- minimisation of the *error propagation*: field procedures with checks
- detection and minimisation *afterward* at the office by the "correct" application of known corrections using "properly" collected and " representative" information.

Let us look at a few examples:

- A traverse should never be referenced on the national network through only one benchmark. At least ***connections to two benchmarks*** have to be used to check their relative *stability*
- The use of *stadimetric readings* helps to recover blunders.
- The use of ***double graduations*** on each rod helps to recover blunders too. But let us point out a frequent wrong statement: in no way will these procedures improve the accuracy.
- The ***"tour d'horizon"*** methodology (1 reading backward, 2 forward, 3 forward, 4 backward) aims at minimising the consequences of the vertical movements of the tripod: if its movements were perfectly steady, they would be perfectly removed.
- The old method of ***double traversing*** with at each station 2 temporary pivots helps to provide a guarantee that during the way of the level from one station to another, the pivots have not moved. This procedure also is in no way an improvement of the precision, and its economic utility is questionable.
- The ***"red pantaloon "*** methodology (at one station the reading starts with the forward rod, at the next one by the backward rod, etc.starting always by a reading on the same rod if two rods are used) slows down the production; its aim is to mitigate traversing systematic errors.
- For the measurements (double runs each one in opposite directions) of high precise levelling lines, it is recommended to ***vary as much as possible the conditions*** existing at the two measuring times (change of observer if possible, have a meteorological situation as different as

possible, etc.). Therefore avoid performing the two runs immediately one after the other. *The goal is not to get the smallest misclosure, but to get a mean value as close as possible to the true one: these two goals are often conflicting in levelling...*

With today equipment and a special attention paid to the temporary pivots and the feet of the tripod, we consider that the best precision will be attained with 2 rods and the "tour d'horizon" procedure. And with digital levels, the procedures formerly used just against blunders should not be used any more.

We shall finally note that if digital levels have a clear sensibility to the level of illumination of the rod and the levelling is in a steady direction for hours, the systematic differences in the illumination for the forward and backward rods for hours could create systematic effects. Tests at IGN-France 1993 with the Leica NA3000 have shown systematic errors amounting to over 0.1 mm for each station, i. e. around 1 mm/km in the North-South direction.

3. WHAT ABOUT THE REFRACTION IN HIGH PRECISE LEVELLING?

Many researchers have worked on refraction, improving our knowledge on this phenomenon. But a large part of the refraction in field conditions is obviously random. We shall summarise here some important points:

The refraction in the first metres is a pure ***question of temperature gradients*** (nearly no effect of pressure, direct effect of humidity completely negligible). Thus it changes with the presence of trees, of passes of clouds, the presence of wind and water. The curvature of the rays is more and more important when it passes *closer to the ground*, which induces a slightly different reading on the rod as what would have been observed without refraction. This effect is known to be very dependent from the height of the ray over the ground. It must be taken care of special situations occurring in urban areas, where the temperature gradients may be quite unpredictable because of the contrasts from houses in the shadow to walls exposed to the sun. *One should carefully avoid long ranges when such anomalous situations appear.*

The ***humidity*** of the ground has an indirect influence. All temperature effects are due to the *exchanges of energy* between the sun, the ground and the atmosphere. Concerning the ground-atmosphere exchanges, they exploit mainly (I) the evaporation of available ground moisture and (II) the heating of air. So the presence or the absence of evaporable moisture in the ground changes also the thermal gradient, which is responsible of the refraction.

The ***turbulence*** in the low layers creates conditions where the noise measurement is high, but where the mean curvature of the beam is often stable, so that induced systematic errors are modest. On the other hand, conditions of very low turbulence and very rapidly varying refraction happen often at the beginning and the end of the day, and they are extremely unfavourable for good measurements.

In flat areas, the refraction is statistically equivalent on the back and fore observations, with a negligible mean effect provided the environment is uniform. When slopes are strong, the sight length becomes quite short, and the refraction (i. e. curvature) difference between back and fore sights will also be small (although not negligible), as curvature effects are proportional to the square of the sight length. So that the main systematic refraction errors will happen when the slopes are moderate, allowing for long sight lines, where the back-fore difference of refraction due to the difference of height between the two sight lines and the ground.

In any case it should be avoided to observe with an optical axis going closer to the ground that as for example 50 cm. In this section of the atmosphere the refraction is by far too important in many common meteorological situations. The best way to decrease the refraction effects is to have a high observation line

The digital levels use in general a large part of the rod for each reading of the graduation with exception for Zeiss DiNi10 & 11 who use only 20 cm. Leica observe within an angle of 2° which means a very large part of the rod so that the effect of the refraction is averaged between zones where it is important and zones where it is smaller.

This situation does not mean that it is an improvement over the previous situation with human eye observation. It means that it is a new situation that is not yet well documented. In some cases, the dependency to refraction is probably worse: when the line of sight goes close to ground, the quite non-linear increase of the refraction effects with regard to the distance to the ground is increasing the effects.

4. THE MOTORISED LEVELLING

After this description of a large part of high precise levelling problems, we shall just point out the important benefits from our experience with the motorised levelling technique compared with classical foot levelling and with respect to the previous analyses:

- The *movements of the rods and the tripods* during observations are extremely small, as the observer does not move around the instrument (the Ni 002 has a mobile eyepiece).
- In any case the pressure from the weight of the observer on the ground surface is distributed through the wheels of the car reasonably far away from the tripod.
- The feet's of the tripod and the temporary pivots are heavy and have a very large contact area with the ground, so that the relative movements between the equipment and the ground are nearly impossible or minimised.
- The collimation of the Ni 002 due to its two symmetric pendulum positions is extremely stable and close to zero, which implies no visible error due to unequal sight lengths.(Up to 5% difference in range).
- The optical axis is around 2.2 m high over the ground surface and in an area where the refraction is much more stable and does not reach so important values.

- Readings on the rods are impossible for the first 60 cm where refraction effects are often extremely strong. Normally the use of 3,5m rods and equal sight lengths implicate that readings under 1m level are avoided.
- The observations are performed very fast, so that the temperature is quite the same for the instrument between the first and the last observations at a given station.

Thus it is easy to understand why, apart from any economical considerations, the motorised levelling has reached so high figures in terms of precision. Many examples from France, Denmark, Sweden and other countries confirm it. The South-North traversing of France between Marseille and Dunkerque in 1983 has been quite interesting, as for very long sections a rms under $0.3\ \mathrm{mm.km}^{-1/2}$ has been achieved, with systematic errors close to zero, as the recent high precision geoidal computations show it now.

5. CONCLUSIONS

Among all the measurement techniques, high precise levelling is in a very old and known technique. Levelling is one of the most precise methodologies and thus the most sensitive to quite infinitesimal phenomenon's whose repetition is the revealing. So our intuition is often useless to anticipate what may be the main error sources in a given situation. Nevertheless, the enormous amount of data acquired through national levelling network observations provides good guidelines to understand the question. But the unsolved questions are numerous enough to keep even the best specialists of levelling very modest. Reality is by far too complex to be fully understood, and only an engineer-level approach has a chance to be reasonably productive...

The picture below shows the participants at the FIG- WG 5.2 meeting in Gävle 1999-03-17

Manufacturing of High Precision Leveling Rods

Fischer T., Fischer W.

NEDO GmbH & Co. KG, Hochgerichtstr. 39-43, D-72280 Dornstetten, Germany,

phone +49 7443 2401 0, fax +49 7443 2401 45, http://www.nedo.de

{tfischer, fischer} @nedo.de

Summary

In this paper we describe the process of manufacturing and calibration of high precision leveling rods.

At NEDO in cooperation with the University of Karlsruhe, Germany, a new method of manufacturing high precision leveling rods has been developed. The basic idea is to use a high energy pulsed CO_2 laser source to generate markings on the surface of a steel tape. Therefore, the tapes are first sprayed black , then afterwards sprayed yellow so that only yellow is visible on the surface. The high energy laser is used to remove yellow color in those regions which are expected to be marked as black, either for barcode or standard scales. To enable precise positioning of the rod we use a high precision laser based calibration system. This new manufacturing method results in very small random errors of less than ±0.007 mm. Permanent calibration of our leveling rods at the Technical University of Munich, Germany, certifies a very small thermal length extension coefficient.

High precision scales

History

Until the late 70's, leveling rods were manufactured in two different ways: (1) using a mask made of invar steel to spray a scale on a steel tape and (2) using a milling machine to engrave the scale on the rod. Both methods were not able to fulfill increasing precision requirements. That was the starting point for NEDO to establish a new method of producing high precision leveling rods.

Overview

Based on a calibration technique described in [3], NEDO developed in cooperation with the University of Karlsruhe a manufacturing process for high precision leveling rods. A tape made of invar steel with a small thermal length extension coefficient ($\alpha \leq 1 \cdot 10^{-6} \cdot K^{-1}$) is first sprayed black and then yellow on top. This tape is moved along a fixed unit which consists of a pulsed high energy CO_2 laser and an optical system consisting of a lens and a mask (see fig. 1). With the high energy laser, parts of the thin top color layer (yellow) can be removed, so that the underlaying layer (black) becomes visible. Therefore the laser is formed by a mask and focused by a lens. To control the CO_2 laser pulses, a high precision laser based calibration system [3] is used. This allows a high accuracy of the manufactured scales. A flexible software interface implemented in the PC based control system allows flexible production of both, standard and barcode scales. With the help of an electro-optical microscope [3] we are able to calibrate our scales for the purpose of a quality control system based on ISO 9001. NEDO is the only company worldwide manufacturing leveling rods of such a high accuracy and quality.

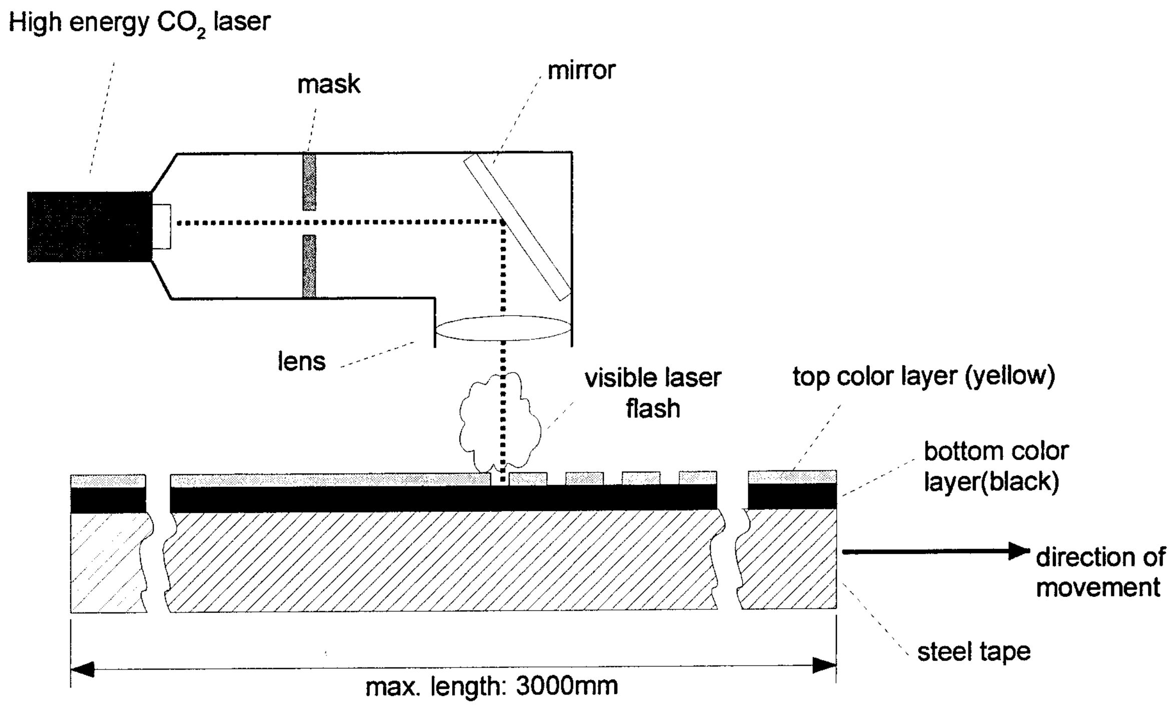


fig 1: principle of the manufacturing process

Production method

During the process of manufacturing (see fig. 2), a steel tape is fixed on a movable unit which passes along a high energy CO_2 laser at a constant speed *v*. The distance *d*, measured by a very high precision laser based calibrator is sent via the interferometer control unit to the PC based control system. The computer compares this information with formerly calculated positions where the CO_2 laser has to be activated. If both values match, the computer sends one puls to the CO_2 laser.

Basic idea of this new method was a procedure known from simple marking tasks used in the production of electronic components and food packaging. In these applications, a high energy CO_2 laser is used to mark the products with their type or the 'best before' date. Instead of using masks for simply writing text on a surface NEDO analyzed these laser systems with the question if it is also possible to use such as laser for high precision marking tasks. First trials showed, that the lens used in the optical path of these lasers caused a distortion which required much more complex masks than initially planned because these masks had to correct the distortions. Another difficulty was to find best parameters for the CO_2 laser like energy and high voltage level. Lots of experiments were necessary to reach this goal. This problem had to be discussed in conjunction with the type of colors used on the steel tape (see fig. 1). We found, that the yellow color on top showed best optical characteristics. Finally we had to decide, whether to stop the movement of the rod at each position where a laser pulse has to remove yellow color or if it is also possible to keep the steel tape moving at a constant speed. The first possibility would require a very precise positioning of the steel tape and the movable unit on which it is mounted. This seemed to be very difficult because we would have to take in account all the dynamic aspects of the whole system. On the other hand, the timing aspects of the computer based control system seemed to be solvable with a standard computer available in the early 80's. The second possibility (keeping the unit moving at a constant speed) required a very powerful control system which has to interact under realtime conditions. But on the other hand, a precise positioning of the moving unit wouldn't be necessary. A lot of experiments showed us, that a solution based on the second version brought better results e.g. a higher precision. The powerful control system which is necessary for this solution is described in the next paragraph.

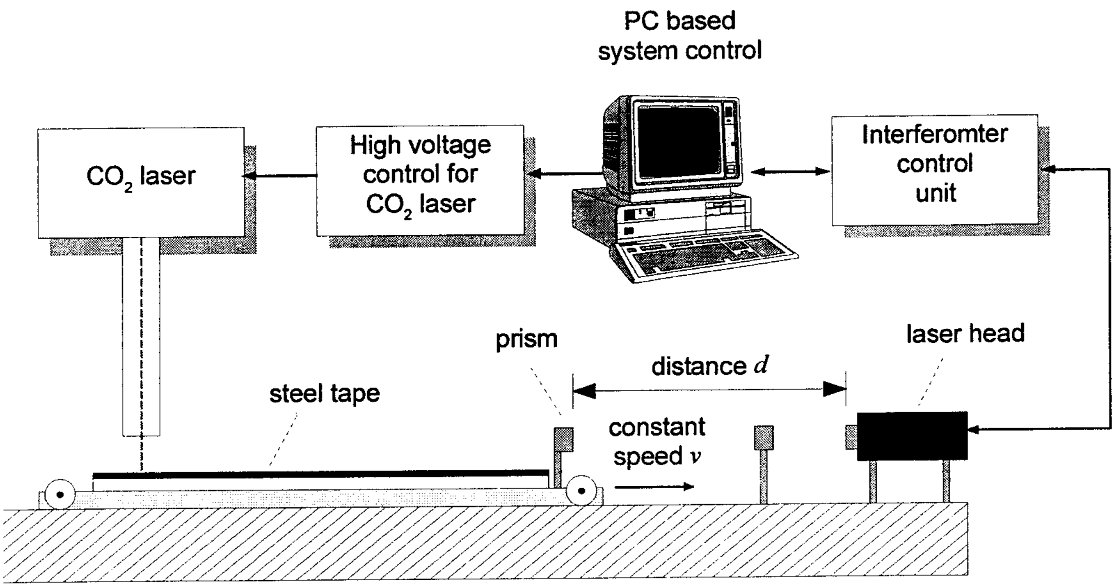


fig. 2: integration of the control structure in the manufacturing process

Control system

Control of such a complex system described above requires a powerful control system. In the beginning of this manufacturing process, we started with a Commodore CBM4000 - a 6502 processor based system running at 1 MHz using a very simple and small operating system. The poor computing power of this system required complex additional control hardware which was designed in cooperation with university of Karlsruhe.

Meanwhile, processing power of today's computers is high enough and we have a control system based on a Pentium processor. Most of the tasks which were implemented initially in hardware are now implemented in software and allow a flexible adaptation to new scaletypes. The fact that the CO_2 laser is a pulsed type requires a little bit more complex software. In case of manufacturing standard scales (1cm), the software calculates those positions where a single pulse must be generated to trigger the CO_2 laser. Much more effort is necessary to generate trigger pulses for barcode scales. Here we have the problem, that a large black area has to be divided into several single pulses. Based on a fixed width of a single shot, this results in a sequence of shots which have to overlap a little bit to guarantee that no yellow color is left (see fig. 3). At a first glance this seems to be a rather simple task but taking in account all requirements, the solution gets a bit more complex. The most important fact is, that we have to ensure a minimum recovery time between two laser pulses to ensure a perfect function of the CO_2 laser system (see fig .3). If the overlapping of two single shots is too large (based on a constant speed v of the steel tape), we will probably get a violation of the recovery time. Another problem we have to focus on is the fact, that we have to work during a manufacturing cycle with only one mask having a fixed width. This requires that all black areas of a barcode scale have to be partitioned into several elements having a width of this single mask. To fulfill this requirement, we get different overlappings of the single shots depending on the width of a black area. To fulfill the requirement of the minimum recovery time we have two possibilities: (1) adjusting the width of the mask and (2) choosing a speed v which ensures no violation of the recovery time. Both parameters can only be varied in small amounts. (1) Increasing the size of our mask will result in an inadequate energy density of the laser flash which wouldn't be high enough to remove the yellow color from the top layer. Decreasing the size of our mask will result in an energy density which is probably too high thus destroying underlaying black color layers. (2) Decreasing the speed v will probably result in a discontinuous movement thus adding errors to the scales. Finally for each type of barcode we are producing we had to optimize all of these parameters to get best results.

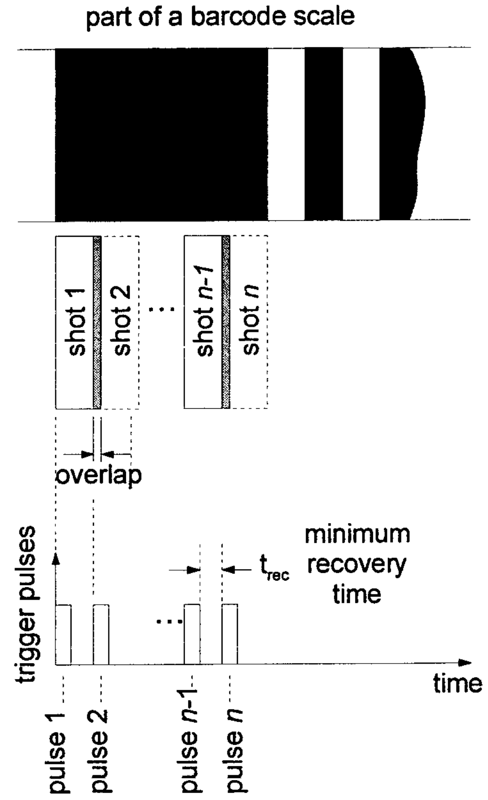


fig 3: dividing a barcode scale into several single shots

Software

To allow production of different types of scales, we developed a control software which divides generating of control data in two steps. In a first step, a barcode scale e.g. is taken and divided into several single shots fulfilling all requirements as discussed in the previous chapter. As a result a list is generated containing all positions where the CO_2 laser has to be activated. This step allows us to generate a scale-independent data format which allows handling of any kind of scale in an easy to use style. A list as described above can contain production data of both, barcode and standard scales. In a second step this list is read by the software which controls the entire production process, and generates trigger pulses for the CO_2 laser. Also the calibration process is based on this software structure: estimated marking positions are read from a list and compared with the real positions detected by the electro-optical microscope.

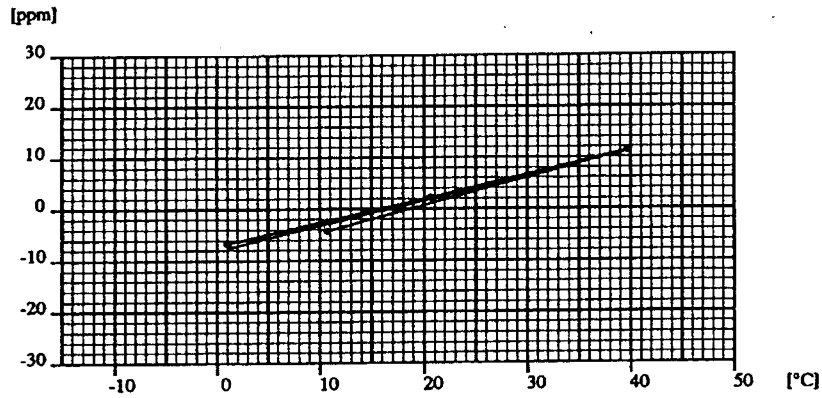
Accuracy

One of the most important questions concerning the manufacturing of high precision scales is their accuracy. Based on the new production method described above, we have random errors of less than ±0.007 mm and a scale factor error of less than 1.2 ppm. To ensure a high level of quality we use an electro-optical microscope [3] for the calibration of our tapes. With its help we are able to detect error sources immediately.

In the last few years, calibration of complete systems (e.g. a leveling rod consisting of an invar steel tape and an aluminum body) has become more and more important [4, 1]. Therefore at Technical University of Munich (TUM) a calibration system was developed which allows calibration of complete leveling rods under changing atmospheric conditions [2]. By permanent calibration of our leveling rods at TUM, we are able to ensure a perfect thermal behavior of our leveling rods. Fig. 4 shows the thermal length extension coefficient of one of our leveling rods. The graph shows the behavior of the rod by applying different temperatures (30° → 0° → 20° → 40° → 10°) to it.

Determination of the coefficient of expansion **Horizontal calibration position**

Measurement cycle: 30 → 0 → 20 → 40 → 10 [°C]



Coefficient of expansion: $\alpha_T =$ 0.48 ± 0.02 *ppm/°C*

fig. 4: thermal behavior of a leveling rod

Assembling of the rods

In the previous chapter we described the manufacturing of high precision scales based on a new method using a pulsed high energy CO_2 laser and a high precision calibration system. After this manufacturing step, the tapes and the aluminum body of the leveling rods are assembled together. To stabilize the steel tape within the body of the rod, springs are used which fix the steel tapes at a force of 30N. This requires that also during manufacturing of the scales the steel tapes are fixed at the same force onto the moving unit of our new CO_2 laser based engraving machine. For the overall accuracy of our leveling rods it is also important to ensure a right angle between the foot and the body of the rods. To fulfill this requirement we developed a set of tools which allows perfect assembling of foot and body.

Future Trends

At the beginning of 1998 we established a quality control system based on IOS9001 which requires permanent quality control of our products. As mentioned in chapter 1.6, we use an electro-optical microscope [3]. It was initially designed for calibration of standard scales (1 and ½ cm scales). By adding new scales to our production line, we found that this microscope is not flexible enough because it only allows detection of edges. For the purpose of permanent quality control we are forced to establish a more powerful image processing tool. Currently we are working on this new system which will also be used to detect misplaced and inadequate laser shots.

Summary

In cooperation with external partners, NEDO developed a manufacturing process for high precision leveling rods using a pulsed high energy CO_2 laser and a high precision calibration system. Based on a powerful control system we are able to produce both, standard and barcode scales at a very high accuracy with random errors less than ±0.007mm. Permanent calibration of our rods certify an excellent thermal behavior. Currently we are working on a CCD based image processing tool, for further improvement of our quality.

References

[1] Ingensand, H.: "Neue Verfahren zur Zertifizierung geodätischer Meßinstrumente", Proc. of 'Geodätische Woche in Obergurgel, Austria', 1997

[2] Maurer, W., Schnädelbach K.: "First Experiences with a Vertical Comparator for the Calibration of Invar Rods. Precise Leveling", Dümmler Verlag Bonn, 1983

[3] Schlemmer, H.: "Laser-Intereferenzkomparator zur Prüfung von Präzisionsnivellierlatten", Verlag der Bayrischen Akademie der Wissenschaften, ISBN 3 7696 9266 7, 1975

[4] Schmid, C.: "Automatische Nivellierlattenkomparierung für Strich- und Codeteilungen", Institut für Geodäsie und Photogrammetrie, Bericht Nr. 244, ETH Zürich, 1995

On behaviour of invar rods with aluminium frame used in Third Levelling of Finland

Mikko Takalo

Finnish Geodetic Institute

FIN-02431 MASALA

Email: Mikko.Takalo@fgi.fi

Abstract

The vertical rod comparator of the Finnish Geodetic Institute (FGI), automated in 1996-1997, applies the HP Laser Interferometer 5529A with the HP5519A Zeeman stabilized HeNe laser as a length standard and the Cohu CCD-camera with an area sensor for determining the position of the rod lines. The comparator is housed in the FGI's laboratory in Masala, especially designed for rod calibrations. The atmospheric conditions in the room can be regulated by an airconditioner for temperature from *5°C* to *50°C* and for humidity from *5%* to *100%*.

Since 1988 the behaviour of the invar rods with aluminium frame was examined by determining their linear thermal expansion and analyzing the results. Several irregularities in length of invar rods were found, mostly indicating malfunction of rod compensators.

Introduction

The Finnish Geodetic Institute (FGI) has a vertical, totally automated laser rod comparator (Fig. 1) housed in the modern laboratory in Masala. Comparator, automated in 1996-1997,

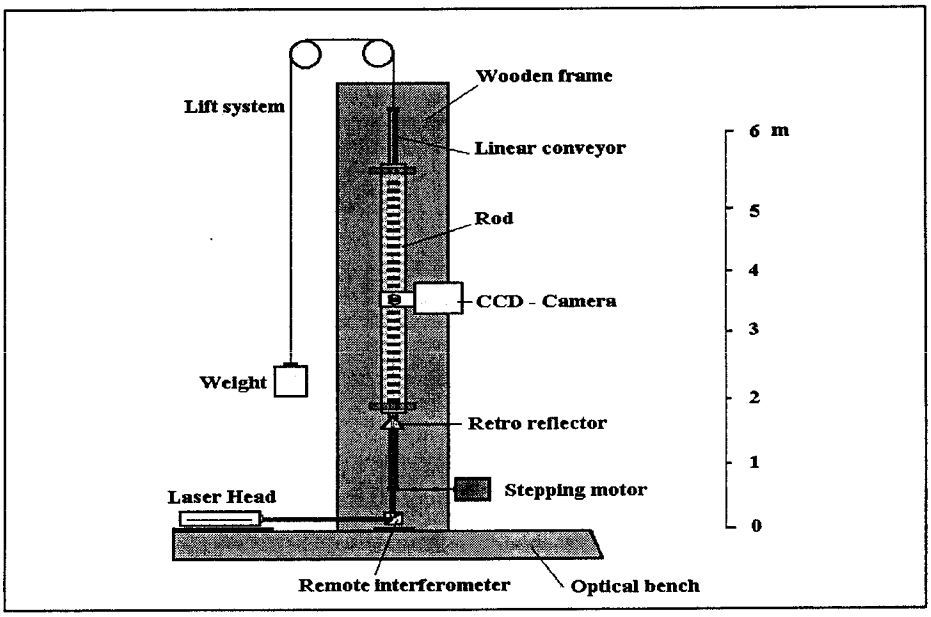


Fig. 1. *Vertical laser rod comparator of the Finnish Geodetic Institute.*

allows the examination of rod scales in vertical position, more precisely and in more details than the previous FGI's comparators (TAKALO 1985) in Finland.

Thus, we are able to calibrate rod scales and determine their thermal expansion with high accuracy, but we are, however, not able to monitore the unpredicted behaviour of rod scales during field measurements.

Behaviour of rod scale

The precise levelling rod is constructed so that the tension of invar band is always constant. This is done with the help of the rod compensator (Fig. 2), which is one of the most important operational parts of invar rod.

In the field circumstances the invar rod is always affected by air temperature. Owing to the large difference between thermal linear expansion of invar band and aluminium frame, an unexpected change in length of invar band can occur in case of the malfunction of the rod compensator.

According to the study by TAKALO (1990) the linear thermal expansion coefficient of aluminium frame is appr. *24 ppm/℃*, whereas that of invar band is less than *1 ppm/℃*. During the field work, lasting normally *2-3* months, the aluminium frame can change by even *2 mm*. Therefore the malfunction of the rod compensator as well as the errors in observing rod temperature can complicate the monitoring of true length of invar band (Fig. 3).

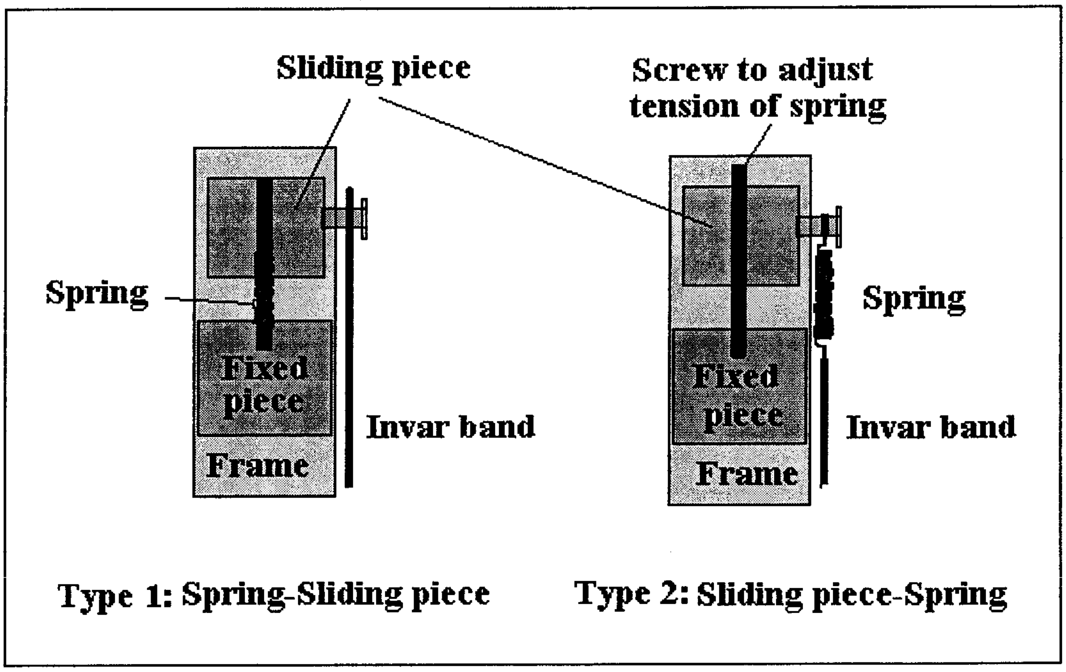


Fig. 2. *Compensator types of invar rod with aluminium frame. Type 1 is typical in rods manufactured in the 80's and type 2 in the 90's, respectively. The previously mentioned is typical construction, e.g., for bar code rods.*

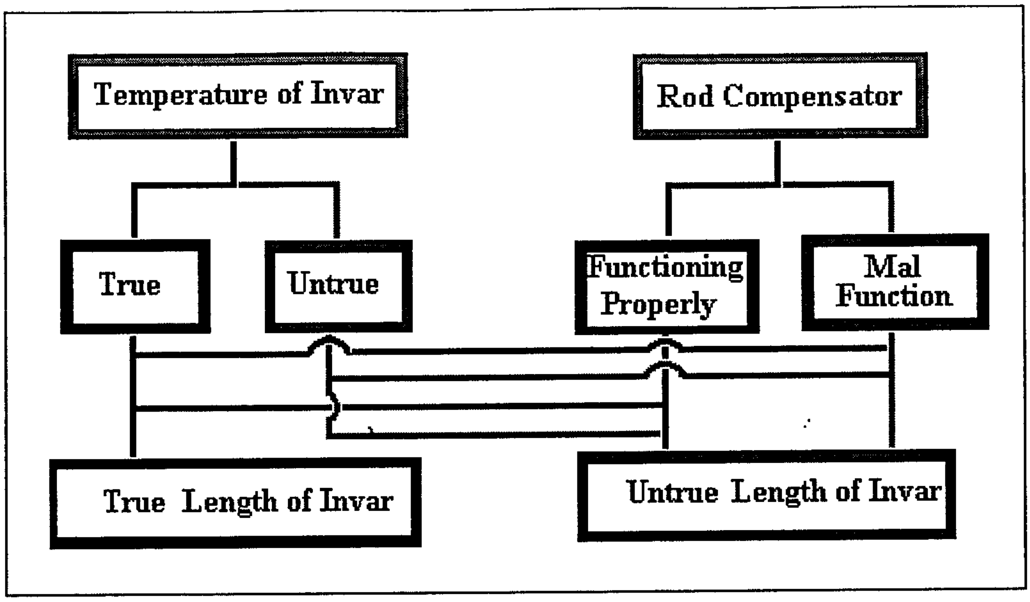


Fig. 3. *Significance of rod temperature and compensator in monitoring true length of invar band.*

There are two methods to test malfuction of the rod compensator:

1) Determination of linear thermal expansion or

2) To measure directly tension of invar band.

The latter technique has been applied by MAURER and ROSSMEIER (1986). Also in Finland the same subject has been examined (TAKALO 1988). A linear dependence between the rod length and the tension of invar band is evident as illustrated in Fig. 4. To achieve a length stability of ±*1 ppm*, the variation in tension must not exceed ±*0.3 kp*.

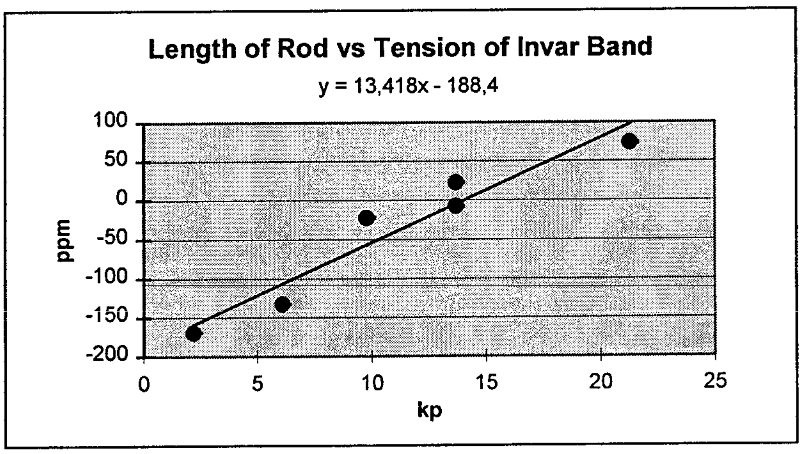


Fig. 4. *Influence of tension to the length of invar band for the rod 369B by Zeiss Jena.*

The result above can be generalized with the formula

$$\Delta L_m = \alpha_F \, (F - F_0) \, 10^{-6}, \qquad (1)$$

where F = actual tension of invar band,

F_0 = constant tension , e.g., *10 kp* and

α_F = mechanical expansion coefficient,e.g., in Fig. 4:

$\alpha_F = +3.3 \pm 0.7\ ppm/kp$.

When applying the former method, where a certain length of invar band is measured at different temperatures, the big difference between temperatures as well as the large length ensure the detection of a possible malfunction of the rod compensator.

The length of invar rod at temperature T is

$$L_T = L_0 + \alpha_T L_0(T - T_0), \tag{2}$$

where L_T = length of rod at temperature T in °C

L_0 = length of rod at temperature T_0 in °C and

α_T = linear thermal expansion coefficient in *ppm/°C*.

The α_T is obtained from the regression for observations (T, L_T).

By denoting $\Delta L = L_T - L_0$ and $\Delta T = T - T_0$ in (2) we obtain the formula for linear thermal expansion coefficient

$$\alpha_T = (1/L_0)\ \Delta L/\Delta T \tag{3}$$

Hence by applying the error propagation law we can derive the formula of standard deviation for α_T.

$$m_{\alpha 0} = (1/L_0)\sqrt{\{ (1/\Delta T)^2\, m_{\Delta L}^2 + (1/\Delta L)^2\, m_{\Delta T}^2\}}, \tag{4}$$

where

$m_{\Delta L}^2$ = a priori variance of length measurement and

$m_{\Delta T}^2$ = a priori variance of temperature measurement.

By substituting the following real values

$L_0 = 2.885\ m$,

$\Delta L = 32\ \mu m$

$\Delta T = 14\,°C$,

$m_{\Delta L} = \pm 0.4\ \mu m$ and

$m_{\Delta T} = \pm 0.2\,°C$,

to (4) we obtain

$$\mathbf{m_{\alpha 0} = \pm 0.008\ ppm/°C},$$

which accuracy can be achieved when the rod compensator is supposed to work properly and observed temperatures are true.

Thermal expansion

The Finnish Geodetic Institute has three pairs of invar rods with aluminium frames used in the Third Levelling of Finland. Two of them are from the 80's with the compensator type 1 and one from the 90's with the type 2 (Fig. 2)

Since 1988, the linear thermal expansion of the rods nos. 8617-8620 were examined four times and nos. 9404 and 9405 three times.

The determinations in years 1988-1989, described by TAKALO (1990) were done at the laboratory of Helsinki University of Technology using the horizontal rod comparator with the HP Laser Interferometer 5526A (TAKALO 1985). The determinations since 1996 are done using the vertical laser rod comparator at the FGI's laboratory in Masala with the HP Laser Interferometer 5529A (TAKALO 1997).

For all determinations the measuring range was *2.885 m* and the measurements in 1996-98 consisted of 10 successive length measurements, which realized the estimation of the stadard deviation for a single length measurement $\mathbf{m_L}$.

The results of linear thermal dependence are given in Appendix 1: Figs. 1-14. The time series of thermal expansion coefficient α_T and standard deviation m_α are given in Table 1 and also in Appendix 2: Figs.1-6.

Table 1. *Thermal expansion coefficients (ppm/°C) in 1988-1998.*

Rod	1988 5-12.4	1989 9.2-14.3	1996 7-9.5	1998 11-18.3	1998 7-15.10
9404			.72 ±.04	.79 ±.04	.65 ±.05
9405			.68 ±.03	.78 ±.02	.72 ±.07
8617	.72 ±.19	.61 ±.09		.62 ±.07	.89 ±.33
8618	.82 ±.25	1.08 ±.10		.87 ±.06	.37 ±.37
8619	1.30 ±.20	1.08 ±.16		.37 ±.40	.95 ±.07
				.66 ±.12*	
8620	.63 ±.10	.34 ±.14		.65 ±.05	.74 ±.09

* Determined 8-14.4.1998 after lubrication of the rod compensator.

When measuring the linear thermal expansion there are three indicators to detect possible malfunction of a rod compensator:

1) The change of α_T ($\Delta\alpha$) indicates a long term malfuction (Table 1 and App. 2),

2) The standard deviation of α_T (m_α)(Table 1), which exceeds a priori value $m_{\alpha 0}$ (See Eq. 3), indicates a short term malfunction and

3) The standard deviation of a single length measurement (m_L) indicates very short term malfunction, i.e., appr. *5-10* minutes corresponding well the time used at a setup in precise levelling (See App.1).

In Fig. 5 the representative values of all three indicators are collected. Because they must be compatible among themselves and characterize well a change of α_T, the maximum values of the standard deviations were selected.

There are two cases, which clearly prove the malfunction of the rod compensator, viz the expansion coefficient of rod no. 8619 in spring 1998 (App. 2: Fig. 5) and no. 8618 in autumn 1998 (App. 2: Fig. 4). The values deviate remarkably from the main trend of α_T and simultaneously the standard deviations are large. The former indicates a long term and the latter a short term malfuction. The expansion coefficient of rod no. 8617 in autumn (App. 2: Fig. 3) does not deviate the main trend of α_T, but its large standard deviation indicates a remarkable short term malfuction of rod compensator. The changes of the successive expansion coefficients of the rod nos. 9404 and 9405 (App. 2: Figs. 1-2) are vanishingly small as well as the values of standard deviations. This indicates that the rod compensators of these rods behave properly even though the standard deviations of α_T exceed the optimal value of $m_{\alpha 0} = \pm 0.008$ *ppm/℃*.

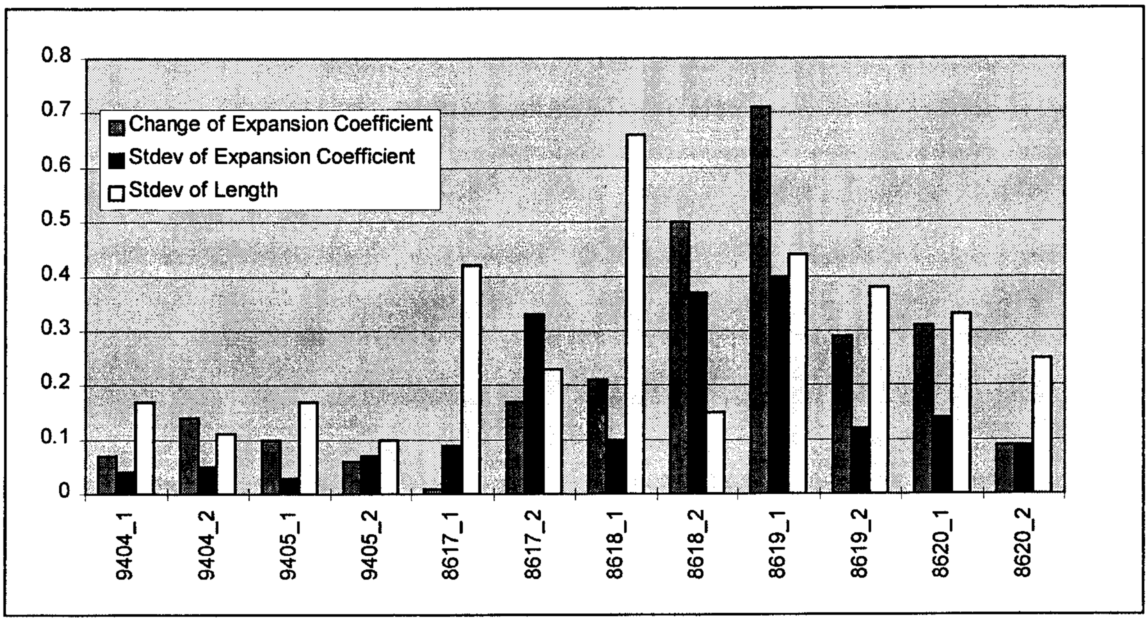


Fig. 5. *Three indicators to detect malfunction of rod compensator.*

In addition the cases 8617_1, 8618_1 and 8619_1 in Fig. 5 indicate a very short term malfunction of the rod compensator. In the first two the change of expansion coefficient is small, whereas the case 8618_2 indicates a strong long term malfuction, but a diminishing small short term one. An explanation to this may be the getting stack of the sliding piece in the rod compensator (Fig. 2).

The principal and real reason for malfunction of the rod compensator is a friction between the sliding piece of the compensator and the surface of the housing for the aluminium frame (Fig. 2: Type 1). In practice this friction can sometimes be so large that a general valid function (Eq.1) length vs tension (Fig. 4) cannot be given. A nice proof of this is the rod no. 8619. The standard deviation of α_T and deviation from the main trend of α_T were ± 0.40 *ppm/℃* and *-0.71 ppm/℃*(App. 1: Fig. 11) and after lubrication and cleaning the corresponding values became significantly smaller: ± 0.12 *ppm/℃* and *-0.42 ppm/℃* (App. 1: Fig. 12).

An example of the influence of a real malfunction may shed light on this serious problem. By substituting the values

$\Delta H = \mathit{300\ m}$ (in Lapland),

$T = \mathit{5\ ^\circ C}$ (Autumn),

$T_0 = \mathit{20\ ^\circ C}$ (Reduction temperature) and

$\Delta\alpha = \mathit{0.93\ ppm/^\circ C}$ (Table 1), assumed to be undetected,

to the formula

$$e_{\Delta\alpha} = \Delta\alpha\ \Delta T\ \Delta H \qquad (5)$$

we obtain the value

$$e_{\Delta\alpha} = \boldsymbol{0.93x15x300 = 4.2\ mm},$$

in observed height difference.

Conclusion

As conclusion we can state that so far we write the length of invar rod as the combination of Eqs. 1 and 2

$$\mathbf{L_{T,F} = L_0 + \alpha_T L_0(T - T_0) + \alpha_F L_0(F - F_0)}, \qquad (6)$$

it does not include any malfunction of the rod compensator and we have to be aware of tension of invar band all the times.

References

MAURER, W., ROSSMEIER, F., 1986, Calibration of Invar Rods-Scale Behaviour, Influence on Precise Levellings. In: Pelzer, H., Niemeier, W. (Eds): Determination of Heights and Height Changes, Hannover, pp. 141-157, Dümler, Bonn.

TAKALO, M., 1985, Horizontal-Vertical Laser Rod Comparator. Reports of the Finnsh Geodetic Institute, 85:2, Helsinki.

TAKALO, M., 1988, Invarmitan pituuteen vaikuttavista tekijöistä. Unpublished. Found in library of the Helsinki University.

TAKALO, M., 1990, The effect of Varying Temperature on the Invar Rod with Aluminium Frame. Paper presented in the XI General Meeting of Nordic Geodetic Commission, May 7-11, 1990 in Copenhagen Danmark.

TAKALO, M., 1997, Automated Calibration of Precise Levelling Rods in Finland. Reports of the Finnsh Geodetic Institute, 97:3, Helsinki.

TAKALO, M., 1998, Experiences of Automated Calibration of Levelling Rods in Finland. 11 pages. Will be published in Contribution of NKG General Meeting 25-29.5.1998 in Gävle, Sverige.

Appendix 1. Thermal dependencies.

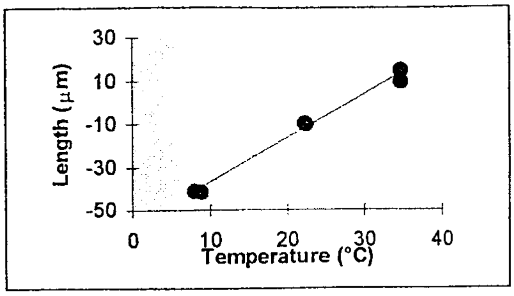


Fig. 1. Thermal dependence of rod 9404 May 1996.

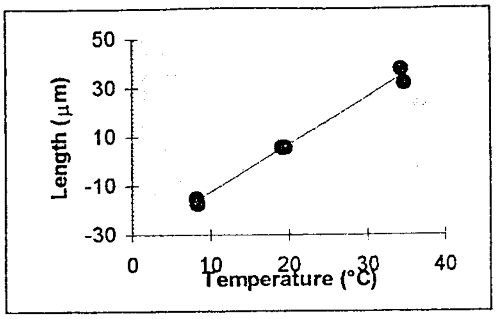


Fig. 2. Thermal dependence of rod 9405 May 1996.

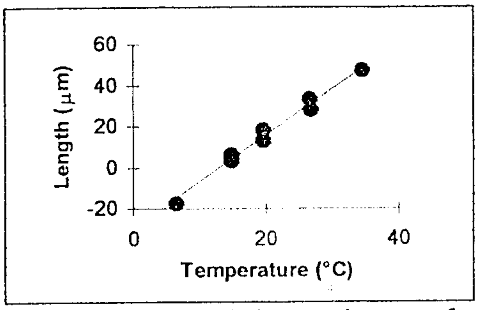


Fig.3. Thermal dependence of rod 9404 March 1998.

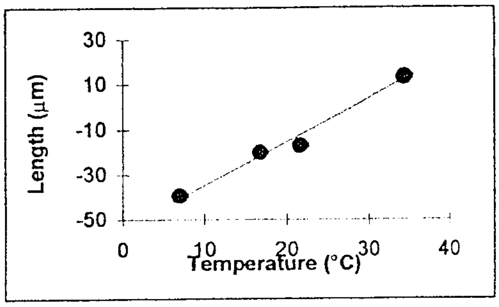


Fig. 4. Thermal dependence of rod 9404 October 1998.

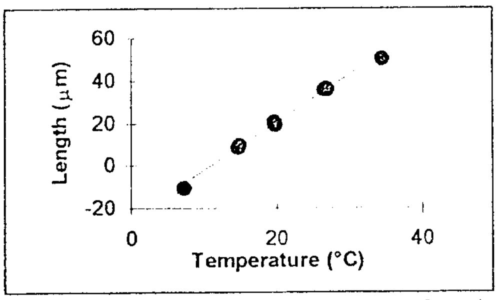


Fig.5. Thermal dependence of rod 9405 March 1998.

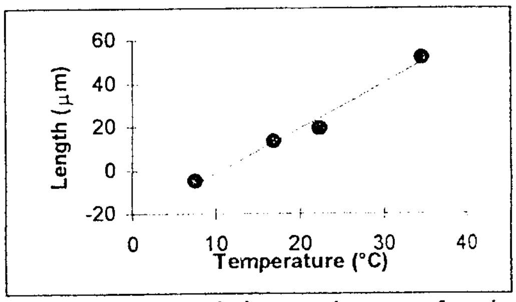


Fig. 6. Thermal dependence of rod 9405 October 1998.

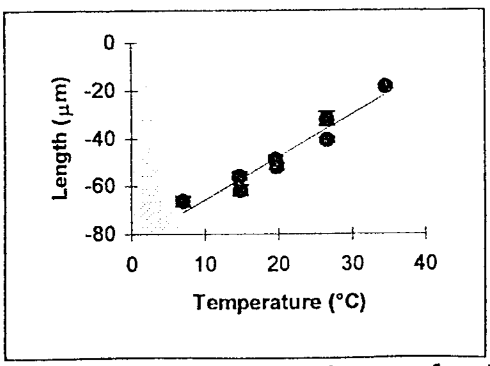


Fig. 7. Thermal dependence of rod 8617 March 1998.

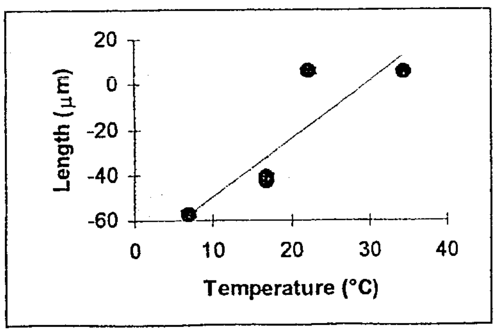


Fig. 8. Thermal dependence of rod 8617 October 1998

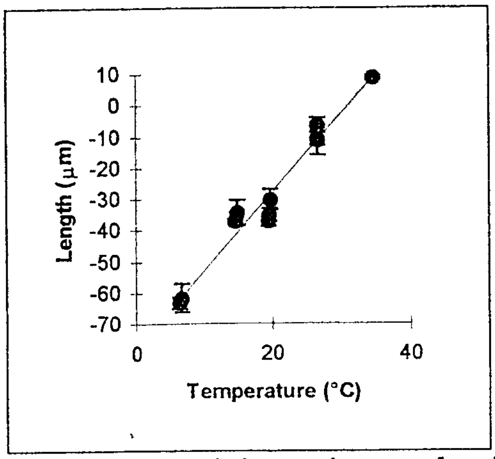


Fig. 9. Thermal dependence of rod 8618 March 1998.

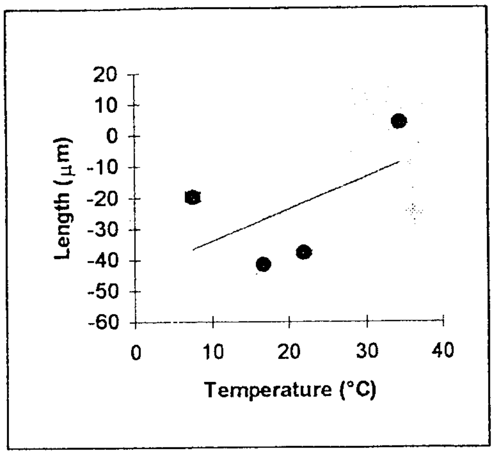


Fig. 10. Thermal dependence of rod 8618 October 1998

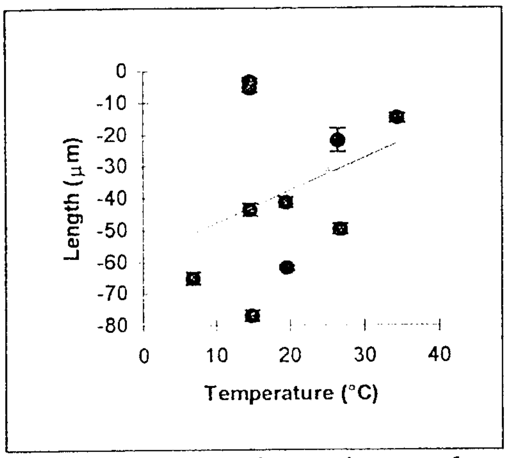


Fig. 11. Thermal dependence of rod 8619 March 1998.

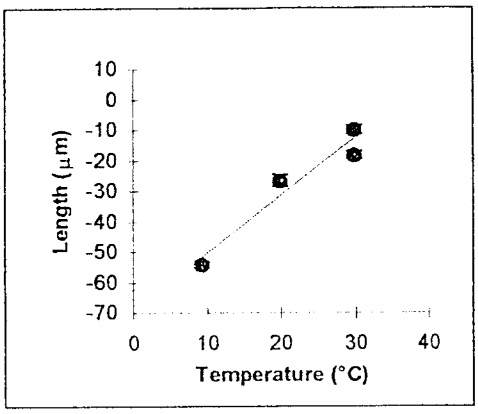


Fig. 12. Thermal dependence of rod 8619 April 1998.

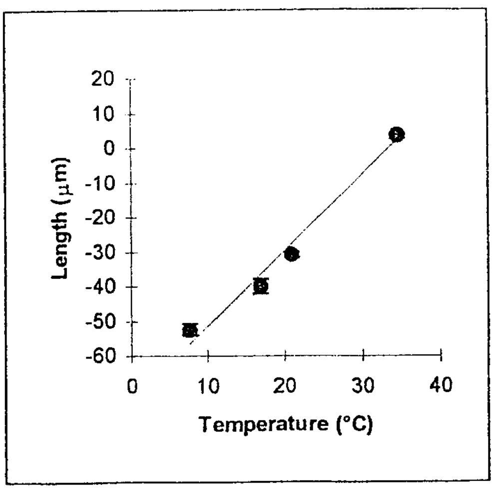


Fig. 13. Thermal dependence of rod 8620 March 1998.

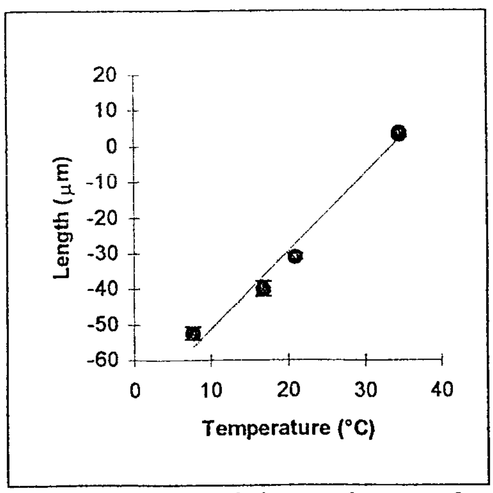


Fig. 14. Thermal dependence of rod 8620 October 1998.

Appendix 2. Thermal expansion coefficients.

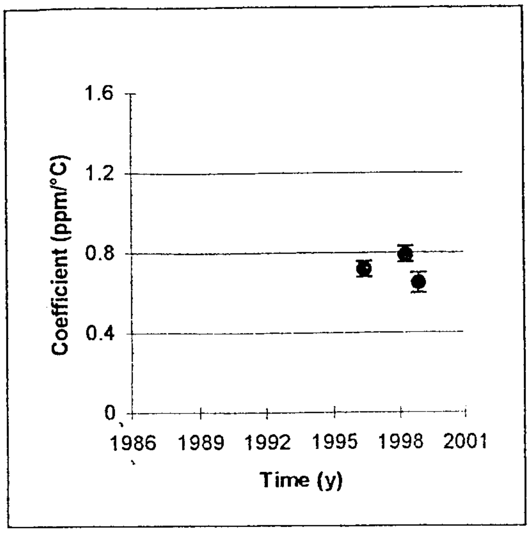


Fig. 1. Expansion coefficient of rod 9404

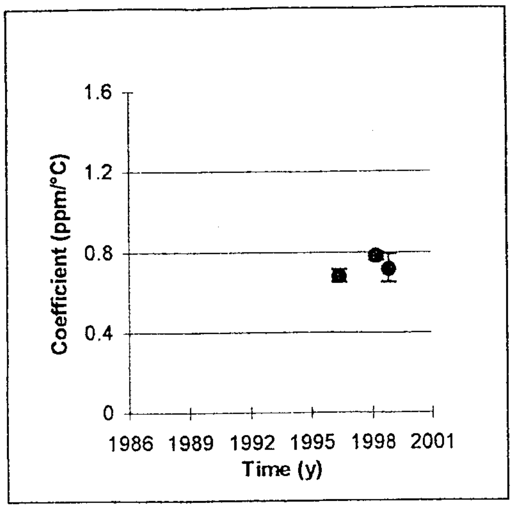


Fig. 2. Expansion coefficient of rod 9405

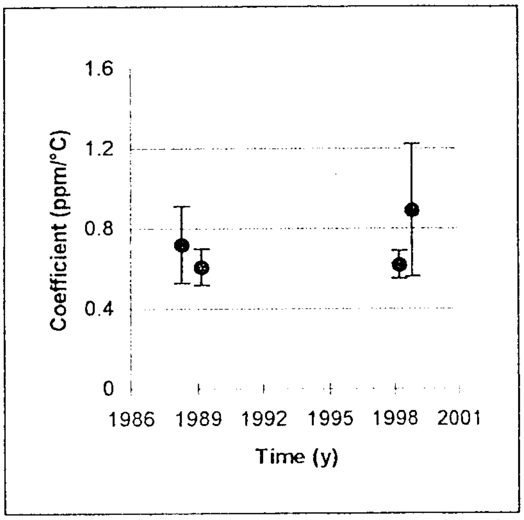


Fig. 3. Expansion coefficient of rod 8617

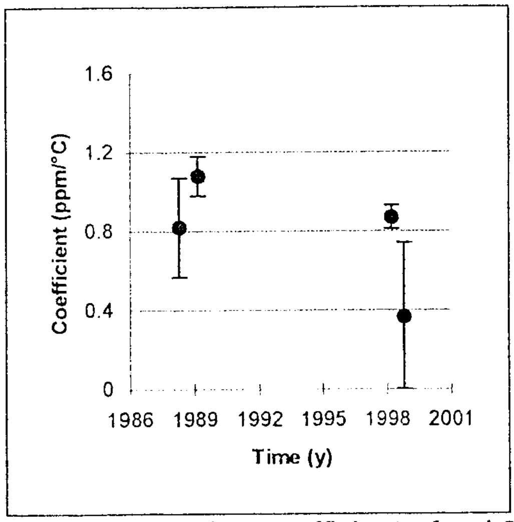


Fig. 4. Expansion coefficient of rod 8618

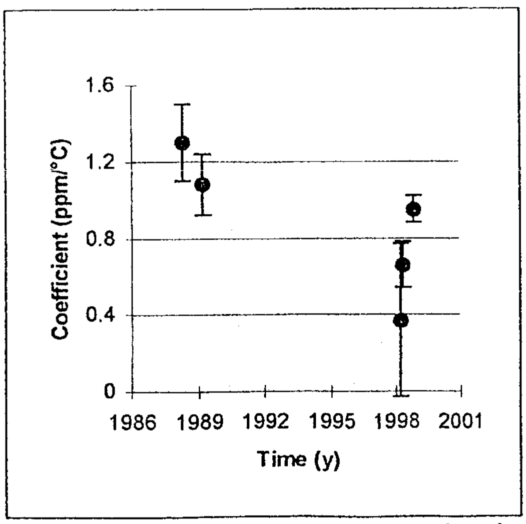


Fig. 5. Expansion coefficient of rod 8619

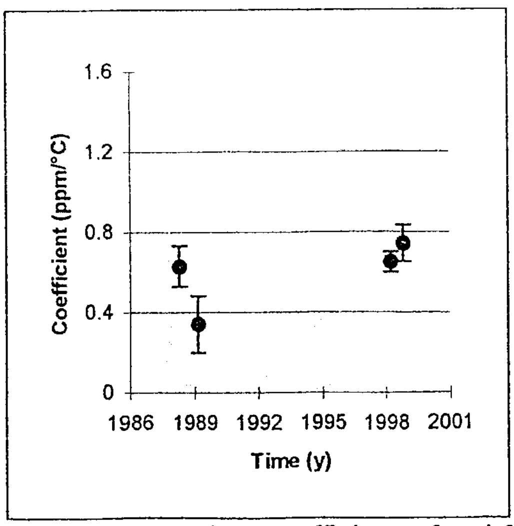


Fig. 6. Expansion coefficient of rod 8620

Checking, Testing and Calibrating of Geodetic Instruments

- Some remarks with respect to recent developments in this field-

Hansbert Heister

UniBw M - Institut für Geodäsie

D - 85577 Neubiberg, Germany

E-mail: heister@glabse.bauv.unibw-muenchen.de

SUMMARY

Checking, testing and calibrating of measuring instruments are traditional activities of all engineers engaged in metrological processes. The necessity of these investigations is justified by the liability to produce measuring results of best accuracy, according to the measuring task. The surveyor as metrologist has as well the responsibility to follow these procedures. But in the last years new technologies and increasing complexity of measuring systems demand completely new methods for investigating or checking these instruments. Additionally new aspects concerning the quality management system (ISO 9000 ff.) have to be considered. The paper proposes some general recommendations and concepts of this special field, which will consider not only technical and economical constraints but also the requirements of an international recognized quality system.

ZUSAMMENFASSUNG

Die Überprüfung und Kalibrierung der Messmittel sind traditionelle Aufgaben aller messtechnisch orientierten Ingenieure. Die Notwendigkeit hierfür liegt darin begründet, dass alle Messprozesse durch zufällige aber auch systematische Messabweichungen beeinflußt werden. Auch der Vermessungsingenieur hat die Aufgabe, zuverlässige Messergebnisse und sinnvolle Genauigkeitsangaben zu verantworten. In den letzten Jahren haben jedoch neue Technologien und steigende Komplexität der Messsysteme die Einführung neuer Prüfverfahren erschwert, in einigen Bereichen sogar unmöglich gemacht. Zusätzlich verlangt ein modernes Qualitätsmanagement System (ISO 9000 ff.) ein Bestätigungssystem für Messmittel. Dieses beinhaltet hierarchisch gegliederte, dokumentierte Prüfverfahren. Im folgenden werden unter Beachtung von wirtschaftlichen und technischen Rahmenbedingungen hierfür Anregungen und Konzepte vorgeschlagen.

INTRODUCTION

The irresistible trend to automated, microprocessor controlled measuring instruments on to totally "black-boxes" and the increased use of these sensor systems by surveyors or by - in this field (geodesy or surveying) - even less educated people demands for new, structured procedures to check, test or calibrate these instruments, in order to guarantee furthermore reliable and accurate results to all customers. The situation has changed so much in the last decade due to very rapid innovation cycles. The old methods for investigating will no longer fulfill all the requirements, which come up with the use of automated measuring equipment. Most of the experts often feel helpless in answering all the questions referring to suitable tests. Additionally more and more companies and legal institutions demand for quality management system according to EN ISO 9000 ff. With its quality management element 11 the international standard ISO 9001 demands *the control of inspection, measuring and test equipment: The supplier shall establish and maintain documented procedures to control, calibrate and maintain inspection, measuring and test equipment (including software)*. More details of this process are documented in ISO 10012 *Quality assurance requirements for measurement equipment.* This standard implies a metrological confirmation system, which each supplier has to establish ensuring that all measuring equipment performs as intended. All these circumstances require new or adapted geodetic test methods especially under consideration of practical realisation under technical and economical constraints. Pursuing the literature in this field (FISCHER, 1998, GOTTWALD, 1998, STAIGER, 1998) the uncertainty becomes even more evident: the necessity therefore is being heavily discussed, but suitable methods are reported only very rudimental. Before continuing, it is necessary first to define the notions *check, test* and *calibration* with respect to metrological applications:

- *Check*: Specified or documented technical procedure with the objective to ascertain, that the instrument continues to measure correctly (within the specifications). In general are checks simple and robust field procedures applied by the user, which have to be defined in a confirmation system and should be performed daily, weekly or on actual demand.
- ***Test***: A comprehensive investigation of the measuring equipment with regard to a defined objective target e.g. to confirm the manufacturers specifications or the correct operations by following the instructions of the manuals. It can encompass single sensor components, sensor systems, software or firmware features, measuring methods etc.
- ***Calibration***: The comparison under specified conditions of sensor outputs against the outputs of a reference standard. The result of a calibration may be expressed as a *calibration constant, factor* or as a series of these quantities, sometimes in the form of a *calibration curve.*

Close connected with these activities is the term

- ***Adjustment***: Operation intended to bring a measuring instrument into a state of performance and freedom from bias suitable for its use (ISO 10 012 / 1).

All these different test methods listed in an hierarchic order are not only useful for correct instruments handling but also are they essential elements of a metrological confirmation system according to ISO 9001.

A GEODETIC METROLOGICAL CONFIRMATION SYSTEM

The term metrological confirmation system is defined in ISO 10 012 / 1 and indicates the set of operations required to ensure that an item of measuring equipment is in a state of compliance with requirements for its intended use. Such a system encompasses the organization, procedures and responsibilities for all activities concerning checking, testing and calibrating. Figure 1 shows you the way such a system is embedded in the hierarchy of different qualified laboratories and defines the traceability, the unbroken chain of comparisons. Additionally it states the structure within a quality management system with references to the appropriate international standards (ISO).

In general we can split all procedures for investigating measuring instruments into two levels:

1. Procedures, which can only be performed by accredited calibration laboratories, which have the technical competence and the high grade of technical equipment.
2. Test methods, which can be realized by qualified technical staff of the company respectively establishment by applying documented procedures.

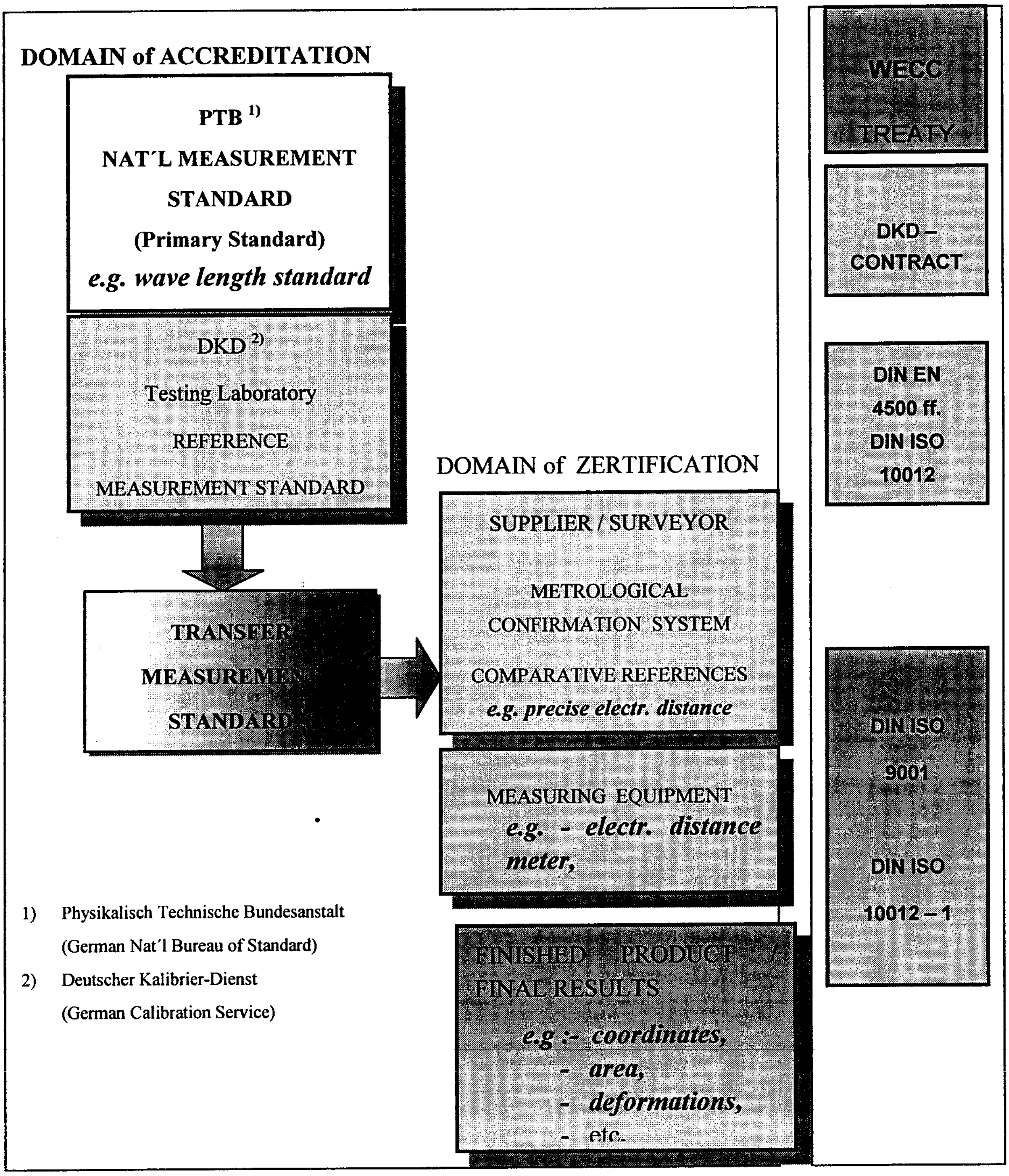


Figure 1: Calibration hierarchy and traceability (in Germany)

Both levels are very important to maintain the conformance of a high precise surveying instrument to specified requirements, but for the surveyor it is more important to focus on the 2nd level - the internal metrological confirmation system. Certainly he should be able to decide, what *he* can check and what has to be done by a *specialized laboratory*.

The internal confirmation system should guarantee that for each surveying instrument respectively each measuring equipment a competent member of the staff is designated as authorized officer, ensuring confirmations to be carried out in accordance with documented procedures and equipment to be in a satisfactory condition (s. Fig. 2). For analyzing and valuing measuring results of calibrations or other tests statistical methods are to utilize allowing the most objective estimation of the uncertainty of measurements and the prediction of appropriate intervals of calibration, checks etc. At this it is indispensable to maintain records of all events and informations concerning the functioning of the instruments. Figure 2 structures exemplary tasks and operations of an internal confirmation system.

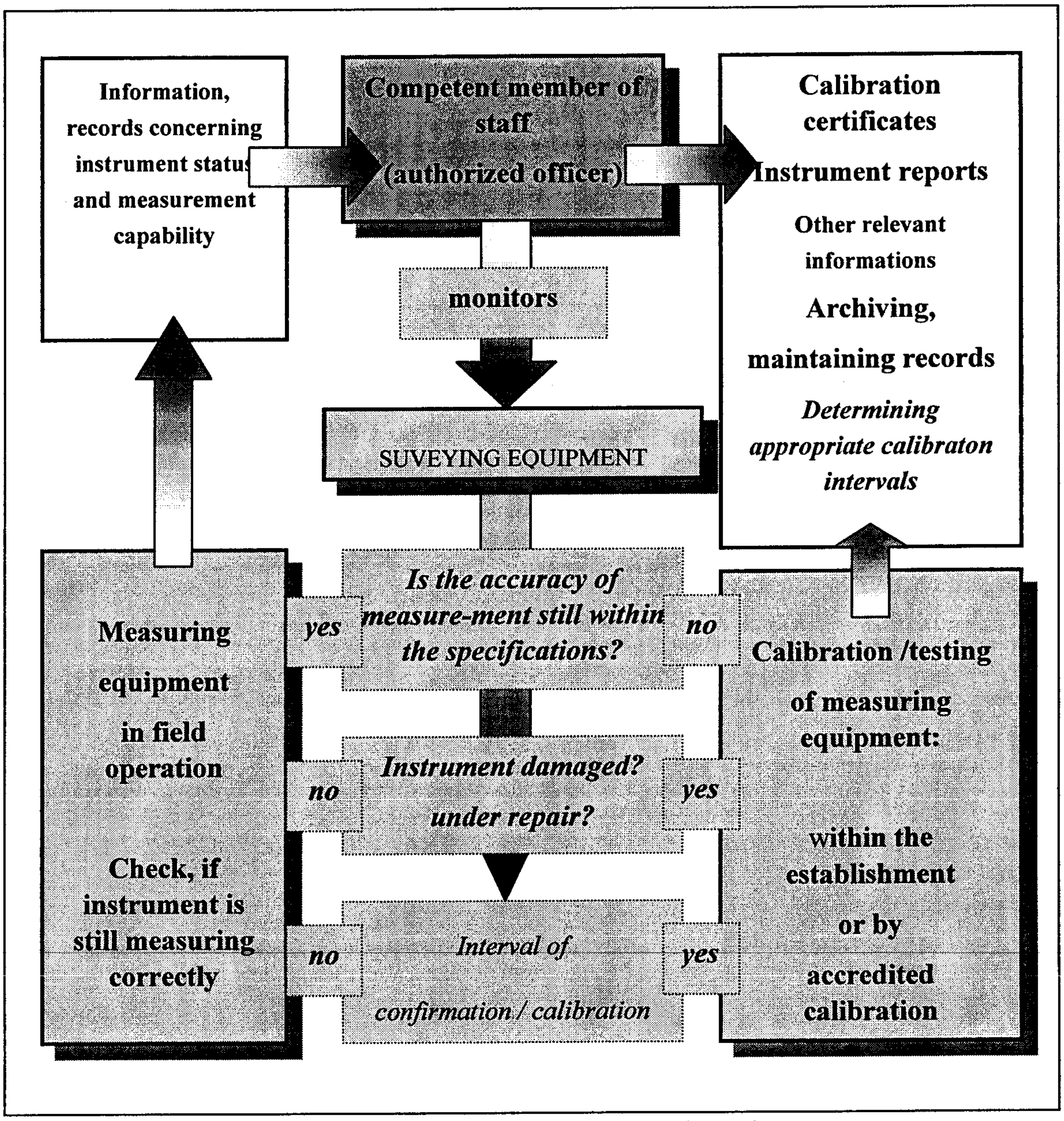


Figure 2: Structure of an internal confirmation system

GENERAL CONSIDERATIONS OF TESTING GEODETIC INSTRUMENTS

The old notions concerning the external structure of e.g. a theodolite, from which you could derive well defined procedures for handling, checking, adjusting or calibrating, are partly totally out-dated.

Modern surveying instruments are better structured on the base of sensor components or functionality (s. Fig. 3).

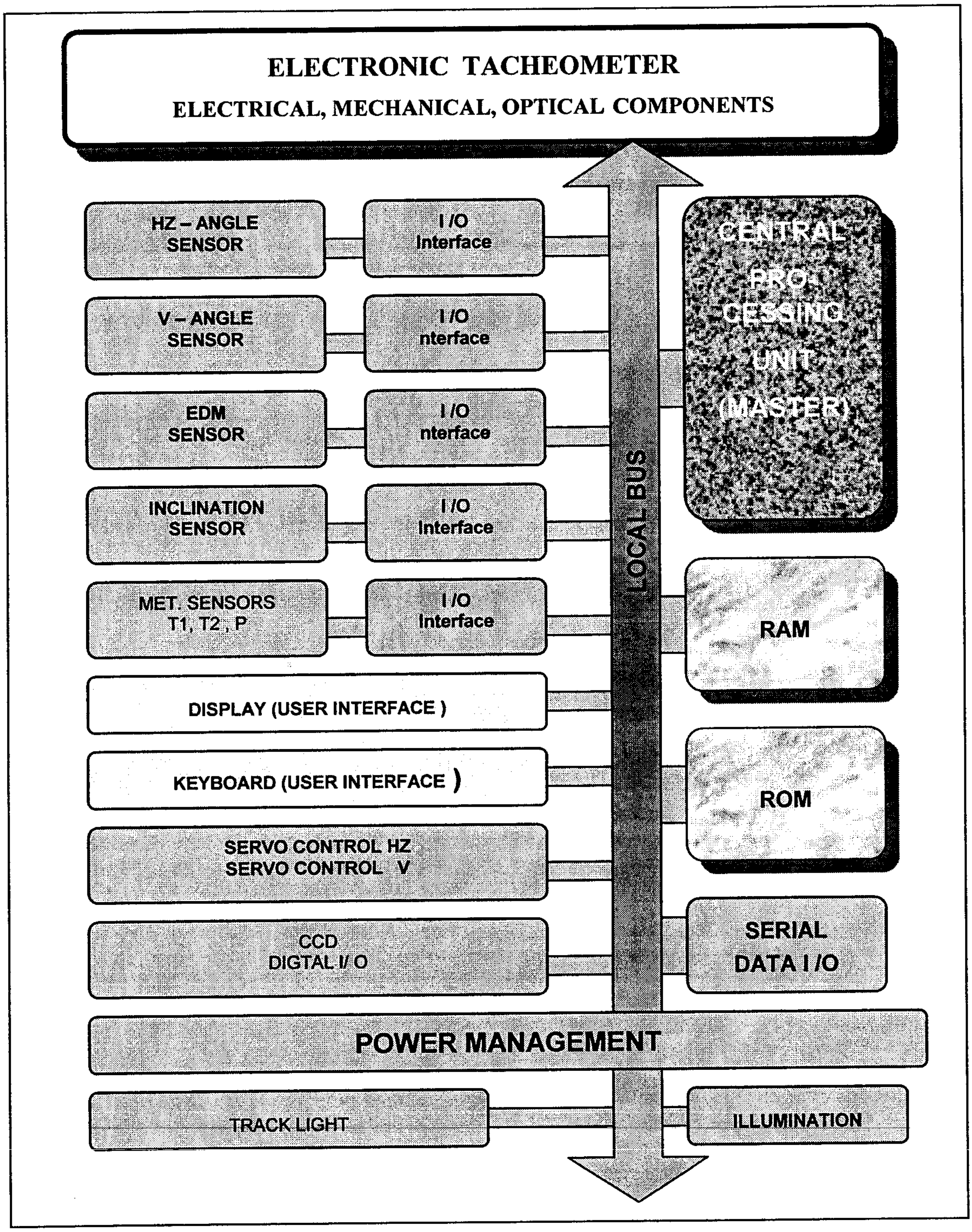


Figure 3: Opto-electronic structure of a modern tacheometer

This points out much better the opto-electronic concept and clarifies additionally interconnection of sensor units, firmware, application software, data acquisition, data transfer and user interface. Operation of these hybride systems has become as complex nowadays making it nearly impossible to survey all functions. The first initializing procedure of an electronic tacheometer can require more than 100 (!) operating steps (keystrokes) and settings. Multitude of instructions and data entry not only has the advantage of extended applications but also is implying as well for the manufactures as for the user to produce (instrumental) errors (HENNES, 1998). The complex sequence from original sensor signals to final results often makes it impossible to locate the reason for a wrong measuring result. Furthermore it is impossible to decide if this was a user's wrong operation or a failed measurement. The interaction of configuring an instrument, controlling, correcting and data processing demonstrates figure 4. That's why it becomes more and more difficult to design robust checking methods. Particularly it is advised to check preferable sensor groups or if possible the complete measuring device using a most simple but effective and representative procedure.

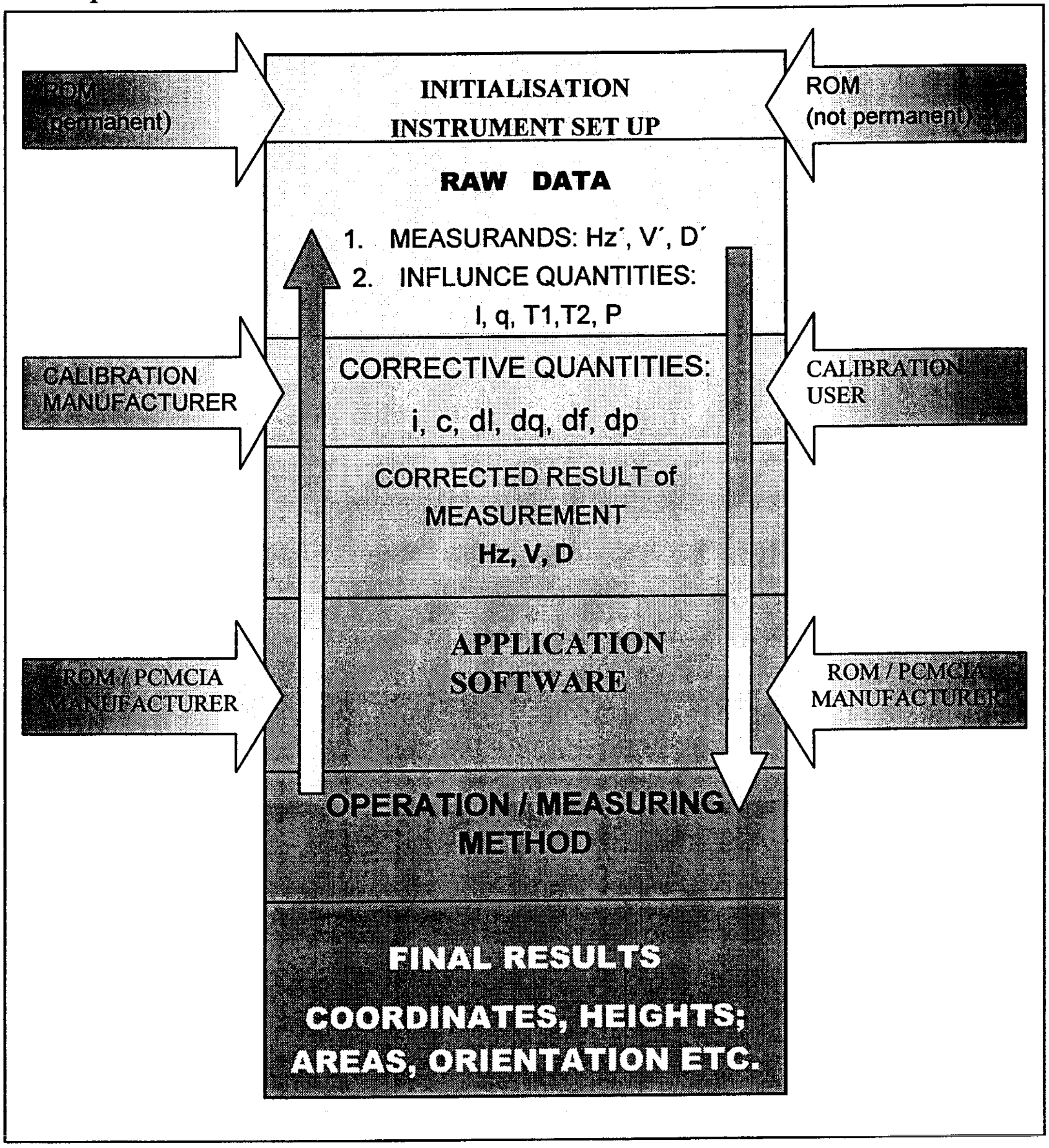


Fig. 4 : Measuring process of a microprocessor controlled surveying instrument

In practice this is not so easy, but first rudimental proposals were published (GOTTWALD, 1998, FISCHER 1998). It is a major task for manufacturers, universities or other institutions specialized on this field to prove new test methods with respect to recent developments and short innovation cycles. Moreover it is important that these procedures are economically reasonable gaining acceptance in practice is as high as possible.

GOTTWALD, 1998 and STAIGER, 1998 propose a stepwise proceeding in 4 phases. Phase 1 and 2 consist of routine checks respectively field procedures. They comprise all these actions, which may and have to be realized by the surveyor in the field or short-time before survey (s. Fig. 2, left part). Beside the FIG publication (1994), which relates to EDM, the new drafts of ISO 17123 – 1,2,3, 4 specify investigations to verify appropriate functioning and to determine accuracy in use for levels, theodolites and EDM's. All proposed procedures are field tests without the need of special additional equipment.

Phase 3 and 4 encompass calibration and extensive testing for acceptance and performance. They demand for a high grade test equipment and reference conditions, where traceability is guaranteed.

In general preferring of so called *system calibration* or *system checks* can be observed. The objective is to aspire to a global test, which confirms correct functioning *of all relevant sensors, controlling (firmware)* and the *application software*. Without knowledge of the specific behavior of a single sensor final results are compared to reference quantities. E.g. FISCHER 1998 describes a proposal and simulation results of investigating a tacheometer.

The practice in calibrating digital levels (phase 3, 4) is similar, but already better proved. Without knowledge of the code, the correlation model and the imaging process *system calibration* yields representative quantities for scale, accuracy, resolution, stability or drift (PIETSCH, 1992, HEISTER, 1994, REITHOFER ET AL., 1996).

The theme quality control and metrological confirmation becomes much more confusing with regard to GPS technology. Though the system is already well established and successfully used in surveying, published methods for checking and calibrating satellite positioning systems are only a few (BÄUMKER, FITZEN 1996, INGENSAND 1997, LANDAU, 1998, STEWART ET.AL. 1998) and no common standard.

CONCLUSION AND RECOMMENDATIONS

It is obvious that there are two major reasons for reconsidering new methods to check, test or calibrate geodetic instruments: (1) New technologies have radically influenced the design of surveying equipment that traditional methods for investigating instruments have become more less obsolete. (2) A state of the art quality management system (QMS) demands for a metrological confirmation system, which should include documented procedures for field and lab checks. The old instructions don't cover all the requirements of a QMS. For the time being there are no standards (ISO, EN etc.) closing the gap properly.

In order to attain new concepts for economical acceptable test method it is necessary that

- the chain from the uncorrected measurands to final results is documented by manufacturers in all details (reference manual),
- the instrument can be reset any time in a controlled basic configuration with clearly documented defaults,
- user friendly operation with a minimum of misoperations is provided,
- simple but effective testing methods (4 phases model) are proposed by manufacturers, universities or other qualified institutions,
- independent accredited calibration laboratories are to constitute, guaranteeing traceability and which are specialized on investigating geodetic equipment. These institutions should be able issuing calibration certificates in accordance with the WECC or any other international organizations.

These remarks may stimulate the discussion about instrument testing between practitioners and experts with the objectives to establish guidelines for calibration or performance tests, procedures for effective checking the functional units of the "black-boxes". But new guidelines have as well to be set up for routine checks and data processing procedures, to guarantee reliable results and best accuracy.

LITERATURE

BÄUMKER, M., FITZEN, H.-P. (1996): Permanente Überwachungsmessungen mit GPS. In: Brandstätter/Brunner/Schelling(Hrsg.): Ingenieurvermessung 96, Ferd. Dümmler's Verlag, Bonn, p. B7/1 – B7/12

FIG-PUBLICATION (1994): Recommended Procedures for Routine-Checks of Electro-Optical Distance Meters (EDM), FIG Technical Monograph No. 9.

FISCHER, E.-N.(1998): Prüfung elektronischer Sensorsysteme bei elektronischen Tachymetern. Allgemeine Vermessungs-Nachrichten (AVN), p. 374 – 378.

GOTTWALD, R. (1998): Prüfung und Kalibrierung von Vermessungsinstrumenten. Vermessung, Photogrammetrie, Kulturtechnik, p. 409 – 413.

HEISTER, H.(1987): Zur automatischen Kalibrierung geodätischer Längenmeßinstrumente. Schriftenreihe des Studiengangs Vermessungswesen der Universität der Bundeswehr München, Heft 27.

HEISTER, H.(1994): Zur Überprüfung von Präzisions-Nivellierlatten mit digitalem Code. In: Schriftenreihe des Studiengangs Vermessungswesen der Universität der Bundeswehr München, Heft 46, p. 95 – 101.

HEISTER, H. (1996): Qualitätsssicherung und Kalibrierung geodätischer Meßmittel. In: Brandstätter/Brunner/Schelling(Hrsg.): Ingenieurvermessung 96, Ferd. Dümmler's Verlag, Bonn, p. C2/1 – C2/9

HEISTER, H.(1997): Hybride Vermessungssysteme. In: 81. Deutscher Geodätentag 1997 in Karlsruhe, Deutscher Verein für Vermessungswesen Schriftenreihe, Vol.27, p. 271 – 275.

HENNNES, M (1998): Tachymeter – Technologien, Instrumentenfehler, Kalibrierung. Paper presented at DVW- AK 5 Workshop, Febr. 16./17., Darmstadt.

INGENSAND, H.(1997): Neue Verfahren zur Zertifizierung geodätischer Messinstrumente. In:

IX. Internationale Geodätische Woche Obergurgl, Institutsmitteilungen der Universität Innsbruck – Geodätisches Institut, Heft Nr. 17, p. 93 - 107

LANDAU, H. (1998): Zur Qualitätssicherung und Vernetzung von GPS-Referenzstationen. Vermessungswesen und Raumordnung (VR), Vol. 60/8, p. 438 – 445.

PIETSCH, U. (1992): Untersuchungen zur Systemgenauigkeitder automatisierten digitalen Nivelliere NA2000/NA3000. Diplomarbeit, Institut für Geodäsie (unveröffentlicht).

REITHOFER, A., HOCHHAUSER, B., BRUNNER, F.K. (1996): Calibration of Digital Levelling Systems. Österreichische Zeitschrift für Vermessung und Geoinformation (VGI), Vol.84, p. 284 - 289

STAIGER, R. (1998): Zur Überprüfung moderner Vermessungsgeräte. Allgemeine Vermessungs-Nachrichten (AVN), p. 365 – 372.

STEWART, M., TSAKIRI, M., MARTIN, D., FORWARD, T.(1998): Traceability and Calibration of Satellite Positioning Systems. Survey Review, Vol. 34, 269, p 437 – 446.

Present State of Standardization in Surveying Profession

Václav Slaboch

Chair of FIG WG 5.1 "Quality Assurance and Standardization"

Research Institute for Geodesy, Topography and cartography,

CZ – 250 66 Zdiby 98

vugtk@vugtk.cz

Summary

Four important aspects influence the present situation in standardization of surveying activities:

- standardization of quality assurance and certification according to the series of Standards ISO 900x
- standardization of geographic information and geomatics as represented by ISO/TC 211 and CEN/TC 287
- standardization of accuracy requirements of various engineering professions
- standardization of metrological procedures for verification of surveying instruments

Information technology and globalization changed the surveying profession and surveyors all of a sudden have to face the fact, that the profession considered until recently an "art of positioning and mapmaking" is not any more an art but in most cases a „push button technology“ which can be carried out "by any one with a good common sense and basic knowledge of IT".

Except for the metrological procedures, the initiative in the creation of new standards in the fields of quality assurance, geographic information and accuracy requirements have been taken over by other professions. This concerns not only proposal of new standards but also creation of completely new terminology.

Since the FIG PC Meeting in Berlin in 1995 the FIG realized the seriousness of the situation and created a liaison status with ISO/TC 211 Geographic Information and Geomatics. CERCO established in 1997 a WG on Quality and exactly at this moment is meeting to discuss the "Good reasons for introduction of the Quality Control according ISO 9000". CLGE are preparing a document on quality assurance and certification of companies and individuals acting in the surveying businesses. IAG participates actively in the final revision of the ISO/WD 15046-11.8 Geographic Information/Geomatics - Part 11: Spatial referencing by coordinates. FIG established a Task Force for standardization.

This presentation aims to inform on the present situation in the field of standardization and tries to define the role that surveyors should play in this field.

Standardization of quality assurance and certification according to the series of Standards ISO 900x

General trend not only in private surveying sector but recently also within the National Mapping Agencies which have to compete on the market for the "just share" in the mapping contracts.

The needs for ISO 900x application to surveying result from:

- globalization
- multidisciplinary character of surveying
- outsourcing trends (contracting)

The most important activity in this field is represented bythe newly created CERCO Working Group on Quality chaired presently by the IGN France. It dealing not only with the Quality Geographic Information but also with the quality of management and individual mapping procedures and techniques.

The commission is presently meeting at Marne-La-Vallée by Paris.

Members of the CERCO WG Quality are all NMA from the EU countries, including Norway, Switzerland and most of the CEE countries.

A revision of ISO 900x Standards Series is under preparation for year 2000.

Standardization of geographic information and geomatics as represented by ISO/TC 211 and CEN/TC 287

At the first glance these two "sister" committees deal with surveying only indirectly, but if we look deeper into the problem we begin to understand that that the series of about 20 new standards will very soon influence the whole surveying profession.

At present a very important working draft ISO/WD 15045-11.8 "Geographic Information/Geomatics – Part 11: Spatial referencing by coordinates" has been put on the table for discussion and it is up to the national surveying organization to forward there remarks to ISO/TC211 through their national standardization organization.

At least surveyors should get acquainted with the news terminology created mainly by geomaticians which is sometimes surprisingly different from the traditions represented even by the most recent books on surveying and geodesy.

Standardization of accuracy requirements of various engineering professions

This area concerns mainly the activities of TC59/SC4 – Dimensional tolerances and measurement in building industry.

This is a domain of a interdisciplinary approach where surveyors are expected to define the accuracy parameters corresponding to optimum quality assurance.

The FIG (managed by the Swedish team) started a pioneering activity already in 1972 by the joint co-operation between ISO, FIG and CIB by publishing "The measuring practice on building site".

Similar activities are now undertaken in cooperation FIG/RICS by preparing Specifications for Surveying and mapping which could serve as good aid especially in international cooperation and in developing countries. Following documents are under preparation:

- Terms and Conditions of Contract for Land Surveying Services
- Client Specifications Guidelines for Surveys of Land, Buildings and Utility Services
- Vertical Aerial Photogrphy and Digital Imagery

Standardization of metrological procedures for verification of surveying instruments

These activities are in the focus of activities of FIG WG 5.1 which should result in Guidelines for the testing of several different instruments. WG-5.1 is divided into several ad-hoc groups namely (chair person in brackets)

- Digital Levels (Ingensand)
- Total Stations (Steiger)
- Laser Planes (Gelhaus and Kaspar)
- GPS (Jaroslav Šimek, tbc)
- Procedures for Laboratory Calibration (Hansbert Heister)

The WG is chaired by Vaclav Slaboch (CZ) and vicechaired.Hansbert Heiter (D).

The new updated ISO Standards for survey equipment

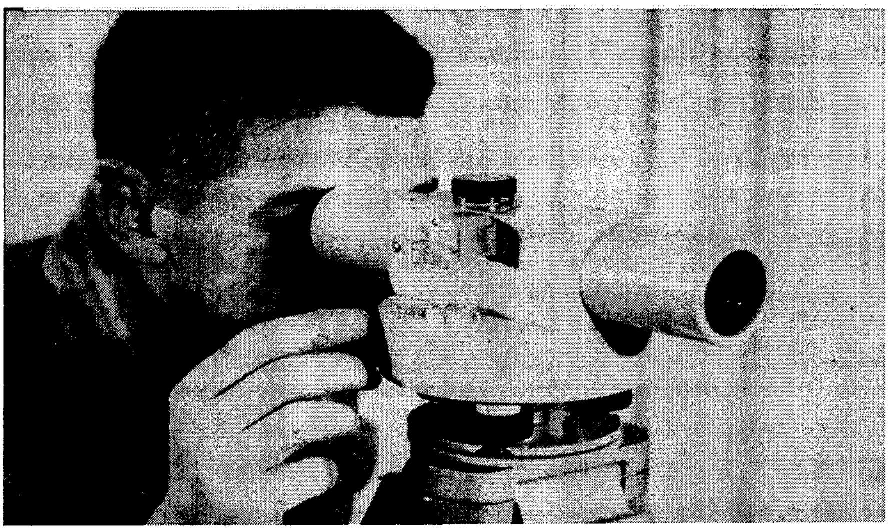
Jean-Marie Becker

1. Introduction.

The surveying profession has been subject to many important changes during the last decades. We have seen a rapid technical evolution especially concerning the surveying techniques and the equipment used for different applications. The Surveyors around the world today commonly use digital levels, laserplanes, total stations and GPS. However ISO, the International Standard Organisation, has not yet succeeded to put on the marked standards for the new instruments. ISO is still working hardly on the updating and harmonisation of earlier standards for a older generation of instruments like as example EDM and theodolites.

Inside ISO, several Technical Commissions (TC59/SC4 and TC172/SC6) have produced different standards concerning the measuring instruments used for levelling purposes. Unfortunately these standards made for the same instrument and the same purpose namely "Field procedures for determining the accuracy of surveying instruments" are often quite different because of different approaches and goals of the Technical Committees. TC59 looked from the building constructing views and TC172 from the instrument manufacturer views.

Since 1997 a Joint Working Group (JWG) common for both TC works on a harmonisation and updating of these standards. The goal is one standard for one instrument. One of the projects concerns levels and is chaired by J-M Becker. A reviewed draft proposal has been discussed in Berlin the 4 March 1999 after examination of the comments from the national standard organisations. The following paper will present the preliminary results concerning the field procedures recommended for the determination of the achievable precision with levelling instruments for different applications.



2. Objective for the new standards

The objectives for the standards are to specify field procedures who have to be followed each time the achievable precision or "accuracy" for a given surveying instrument (level as example) used together with its ancillary equipment (tripod, staffs, etc) has to be determined.

3. Requests on the standards

3.1- Some common requests for each standard:

- Only "one standard" for "each instrument".
- Who can be used anywhere and
- Whiteout any use of special ancillary equipment.
- By common field operators (technicians as well as academics)

That is to eliminate confusions, difficulties in applications and in interpretations.

3.2- Requests from the surveyors on the standards:

It is important that the Surveyors are convinced about their practical utility in other case they do not apply the standard. For that purpose the Surveyors basic requests concerning the use of their equipment have to be taken in account:

- The standard has to:
  - Be useful for different survey applications.
  - User friendly
  - Do not take to much time to be implemented (about ½ hour)
  - The results have to be easy to be interpreted
- The standard has to determine the achievable precision for the following cases:
  - a single survey team (one given instrument + ancillary equipment + personal)
  - at a specific given time

- for a given specified project/ application/purpose
- under well specified environmental conditions (meteorological, vegetation, ground, environment, etc)

Or for:

- a longer time period
- several survey teams with the same equipment
- same team with several instrument of the same type/fabricate
- through similar projects
- all type of survey conditions

- The surveyor requests can be summarised in the following question*:*

"Can I achieve required precision/("accuracy") for a given project with my equipment, yes or no?"

The goal is to investigate if the equipment is appropriated for the intended-measuring task.

4. Field procedures.

The procedures we want to describe are designed for field and not for laboratory use. As mentioned above the results are specific for *each* field test/check and representative only for the *particular* (or similar to) conditions at the *time* of determination: weather, temperature, and environment, ground surface, operator, instrument & staff numbers, etc. Test procedures for calibration or laboratory checks are not described here.

Two different field procedures have been proposed: a simplified and an accurate method.

In all cases the equipment has to be adjusted before testing in accordance with the manufacturers handbooks

4.1- The accurate field checks method.

This field method is proposed for the determination/check of the highest achievable precision/("accuracy") with specific equipment. Normally it is for the purpose of *precise levelling* (linear applications) where high accuracy is demanded and where the observations at each set-up normally are made with *equal site lengths backwards and forwards.*

The accuracy will be expressed in terms of the *standard deviation of 1km double-run levelling.*

Establishment of a test line.

In a plane area with a homogenous ground surface (gravel preferably) fry from vegetation or other disturbing factors (water plane, grass) two points A & B have to stable monumented at a distance AB of about 60m. The chosen site length will be 30m which is normally the recommended distance for high precision levelling. (See figure 1 below)

Note:

- A variation of 10% between the site lengths at each set-up can be accepted. That is a realistic tolerance compatible with normal field applications.

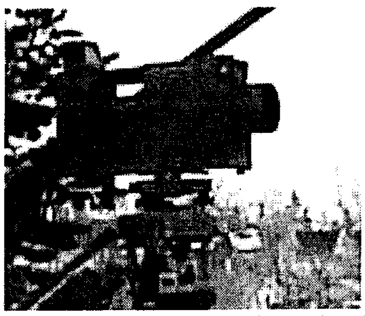
- Also greater site lengths (up to 50 – 60m) can be used for the purpose of testing the equipment's capacity and range of accuracy or to fulfil specific project recommendations/specifications.
- It is very important that all factors specific for each check: equipment, ground surface, vegetation, weather, operators, etc are registred and well documented.

The observation procedure:

The measurements are made in two sets with interchanging the positions of the staffs between A and B. Each set consists of *n-pairs of readings* (preferably 20) bachwards to staff A and forward to staff B , respectively B and A , and resulting in n-height differences. Between each pair of readings *a new instrumental set-up* has to be made.

All details about how to operate, calculate and evaluate will be described in the coming standard with in appendix and example.

Table NR 1 in appendix is an example of this accurate check method.



Notice that here n = 20 and that two different site lengths (30m and 47m) have been used.

Evaluation of the results:

The analyse of the results will be made with the help of statistical tests helping the surveyor to decide if its equipment yes or no allow him to achieve the expected "accuracy".

4.2- The simplified field method

This method is based on a limited number of measurements (minimum n = 10) for the check of the levelling equipment used especially on construction *workside where radial measurements* with unequal site lengths at each set-up are of common use. Equal site lengths are exceptions.

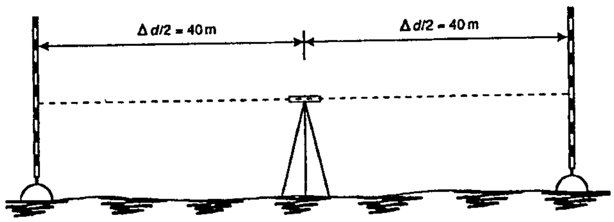
Establishment of a test line:

In a relatively plane area two points A & B have to be monumented at a distance corresponding to the maximum and minimum site length ranges that will be used inside the specific project. As example if inside a construction project the needed site lengths are between 10 and 70m, the distance for A-B will be about 80m. A and B have to be stable during the check period.

Observations procedures.

The measurements are made in two different steps.

The *first* step (1) with equal site lengths is a copy of the accurate method describe before but limited to 10 set-ups. The goal for it is to determine a *reference height difference* between A and B, value that is considered as *true value* of the height difference of the levelled points A and B. (see fig 2a)



For the *second* step (2) the instrument is placed so that the maximum eccentricity for the set-ups is used: in our example 10m and 70m (see fig.2b below). Again 10 set of observations on both staffs A and B are made. Figure 2b below:

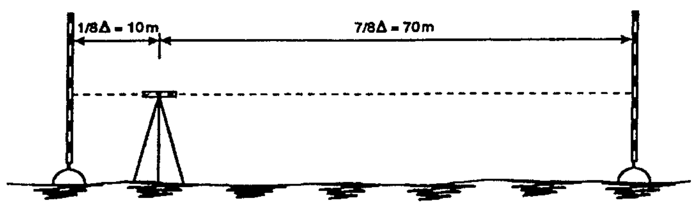


Figure 2 — Second configuration of the test line for the simplified test method

Table NR 2 in appendix is an example of this simplified check method.

Notice that n = 10 has been chosen

Evaluation of the results

Explanations for the calculation and the evaluation of the check will be provided in the annexe together with the standard.

5. Conclusions

FIG-C5 is grateful that the ISO Technical Committees TC59 and TC172 have taken in account the requests from the surveyor communauty for the uppdating and harmonisation of existing standards. We also have noticed that efforts are undertaken to prepare standards for the *new generation* of survey instruments like total stations, laserplanes and perhaps GPS. We hope that these standards will soon be reality.

FIG Commission 5 will contribute with its experts (WG 5,1) to the elaboration of this standards through collaboration with ISO and participation in the work.

Furthermore FIG-C5 will help the surveyors to implement these standards in the best way.

FULL TEST DAT

OBS: All readings are i metres, All other calculations are in mm

Instrument No: 2739 **Type:** NA3003 **Operator:** HB **Date:** 28 Nov, 1997 Location: NLS

Staff A No: 10A **Staff B No:** 10B **Weather:** Sunny, -5 C

Backward=30 Forward=30						Backward=27 Forward=33					Backward=4 Forward=50				
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Set Up	Backward	Forward	Dn=	v=		Backward	Forward	d'=	v'=		Backward	Forward	d"=	v"=	
No	rbn	ran	Rbn-ran	(x)-dn	v*v	rdn	rcn	rdn-rcn	d'-(x)	v'*v'	rfn	ren	rfn-ren	d"-(x)	v"*v"
	m	m	Mm	mm	mm2	m	m	mm	mm	mm2	m	m	mm	mm	mm2
1	1,5157	1,2978	217,9	0,060	0,004	1,6105	1,3928	217,7	-0,2600	0,0676	1,3893	1,1710	218,3	0,340	0,116
2	1,5166	1,2986	218,0	-0,040	0,002	1,6143	1,3967	217,6	-0,3600	0,1296	1,3895	1,1711	218,4	0,440	0,194
3	1,5275	1,3093	218,2	-0,240	0,058	1,6151	1,3973	217,8	-0,1600	0,0256	1,3833	1,1649	218,4	0,440	0,194
4	1,5273	1,3092	218,1	-0,140	0,020	1,6158	1,3982	217,6	-0,3600	0,1296	1,3885	1,1705	218,0	0,040	0,002
5	1,5303	1,3125	217,8	0,160	0,026	1,6144	1,3966	217,8	-0,1600	0,0256	1,3917	1,1739	217,8	-0,160	0,026
6	1,5401	1,3223	217,8	0,160	0,026	1,6150	1,3969	218,1	0,1400	0,0196	1,3943	1,1763	218,0	0,040	0,002
7	1,5431	1,3249	218,2	-0,240	0,058	1,6106	1,3928	217,8	-0,1600	0,0256	1,4029	1,1848	218,1	0,140	0,020
8	1,5476	1,3298	217,8	0,160	0,026	1,6129	1,3949	218,0	0,0400	0,0016	1,4036	1,1855	218,1	0,140	0,020
9	1,5399	1,3222	217,7	0,260	0,068	1,6089	1,3910	217,9	-0,0600	0,0036	1,4074	1,1892	218,2	0,240	0,058
10	1,5327	1,3146	218,1	-0,140	0,020	1,6119	1,3938	218,1	0,1400	0,0196	1,4085	1,1903	218,2	0,240	0,058
11	1,4957	1,2779	217,8	0,160	0,026	1,6061	1,3883	217,8	-0,1600	0,0256	1,4092	1,1911	218,1	0,140	0,020
12	1,5037	1,2857	218,0	-0,040	0,002	1,6013	1,3834	217,9	-0,0600	0,0036	1,4163	1,1983	218,0	0,040	0,002
20	1,4988	1,2809	217,9	0,060	0,004	1,6046	1,3868	217,8	-0,1600	0,0256	1,4116	1,1935	218,1	0,140	0,020
		Sum(S)=	3269,4	Sum(v*v)=	0,3960		Sum(S')=	3267,5	Sum(v'*v')=	0,5540		Sum(S")=	3272,4	Sum(v"*v")=	1,0360
		Mean(x)=	217,96	s=	0,1682				s'=	0,1922				s"=	0,2628

SIMPLIFIED TEST DATA

OBS: All readings are in metres. All calculations in mm

Instrument No: 2739 **Type: NA 3003** **Operator: HB** **Date:28 Nov, 1997**

Location: NLS

Staff A No: **Staff B No:10B** **Weather: Sunny, -5 C**

Backward=	40 m	Forward=	40 m			Backward=	10 m	Forward=	70 m	
1	2	3	4	5	6	7	8	9	10	11
Set Up No	Backward	Forward	dn=	v=		Backward	Forward	d'=	v'=	
	rbn	ran	rbn-ran	(x)-d	v*v	rdn	rcn	rdn-rcn	d'-(x)	v'*v'
	m	m	mm	mm	mm2	m	m	mm	mm	mm2
1	1,6978	1,551	146,80	-0,13	0,0169	1,4737	1,3263	147,40	0,73	0,5329
2	1,6952	1,5486	146,60	0,07	0,0049	1,4711	1,3235	147,60	0,93	0,8649
3	1,6972	1,5506	146,60	0,07	0,0049	1,4824	1,3351	147,30	0,63	0,3969
4	1,6957	1,549	146,70	-0,03	0,0009	1,4837	1,3366	147,10	0,43	0,1849
5	1,6988	1,5521	146,70	-0,03	0,0009	1,4894	1,3427	146,70	0,03	0,0009
6	1,6958	1,5492	146,60	0,07	0,0049	1,4937	1,3471	146,60	-0,07	0,0049
7	1,6998	1,5531	146,70	-0,03	0,0009	1,4982	1,3509	147,30	0,63	0,3969
8	1,6997	1,5531	146,60	0,07	0,0049	1,4954	1,3476	147,80	1,13	1,2769
9	1,7011	1,5544	146,70	-0,03	0,0009	1,4947	1,3468	147,90	1,23	1,5129
10	1,7041	1,5574	146,70	-0,03	0,0009	1,4948	1,3469	147,90	1,23	1,5129
		Sum(S)=	1466,70	Sum(v*v)=	0,0410		Sum(S')=	1473,60	Sum(v'*v')=	6,6850
		Mean(x)=	146,67	s=	0,0675				s'=	0,8176

$$s = \sqrt{\frac{\sum(v*v)}{n-1}}$$

$$s' = \sqrt{\frac{\sum(v'*v')}{n}}$$

Ocean tide loading effects on height

A H Dodson and D J Curtis[2], IESSG, University of Nottingham, UK

T F Baker - Proudman Oceanographic Laboratory, Bidston, UK

ABSTRACT

The paper outlines the theoretical principles behind Ocean Tide Loading (OTL) before concentrating on the practical implications for GPS positioning, giving examples of loading effects at both global IGS sites as well as at European Tide Gauge GPS stations. The paper describes how, with modern ocean tide models, it is possible in most areas to accurately model the loading effect, and also demonstrates how neglect of the effect can lead to aliasing of results, particularly when using short, episodic measurement strategies. This latter effect can be particularly important when assessing time series of height observations.

INTRODUCTION

There is now widespread use of GPS for a variety of applications ranging from monitoring crustal dynamics to data collection for GIS. A number of these applications require centimetric, if not millimetric, accuracy and at that level of accuracy various instrumental and environmental effects have to be taken into account. Amongst these the deformation of the Earth due to external forces, such as Ocean Tide Loading (OTL) and atmospheric pressure loading are important. These effects may be more critical in the execution of episodic surveys (eg for monitoring purposes), when the deformations caused by them may be aliased into the longer period movements being investigated.

Furthermore, whilst OTL effects are largely differenced out in short baseline measurements, the use of remote reference stations (eg from a national or international reference network) will often mean a significant effect may remain in the GPS data, unless it has been properly accounted for by appropriate error modelling.

This paper reviews the theory behind OTL effects, with reference made to the deformation caused in the UK, Europe and world-wide. The effect of signal aliasing, in episodic surveys, is also described and illustrated. The paper concludes by comparing some recent ocean tide models which demonstrate a high level of agreement. This confirms that there is the potential to remove OTL effects from GPS data to a high level of accuracy over most regions of the world, provided an up-to-date tide model and appropriate loading computation software are available.

BASIC TIDAL THEORY

The moon and the sun exert tidal forces which deform the solid Earth such that, at low latitudes, its surface moves through a range of typically 40 cm in a little over 6 hours. This deformation is called the Earth's body tide (for more detailed information on Earth tides see for example Baker, 1984 and on ocean tide loading Curtis, 1996). Using a geophysically determined Earth model, the tidal deformation of the Earth can be computed and is given in terms of dimensionless parameters (Love numbers). Then, given the lunar and solar ephemerides and the co-ordinates of a site, the vertical

[2] Now at The Hydrographic Office, Taunton, UK

and horizontal (body tide) deformations can be computed for any epoch. Software used for processing space geodetic data normally uses this procedure to correct for the Earth's body tide (see IERS standards, McCarthy, 1992). The body tide displacements can be computed to an accuracy of about 2 mm, the remaining uncertainty being due to variations in structure of the Earth and its inelasticity at tidal periods.

It is common in tidal work is to express tides as harmonic functions of time. A few of these tidal harmonics are dominant and the largest were given symbols by G H Darwin. The most significant are listed in Table 1. The frequencies, amplitudes and phases of these harmonics in the tide generating forces are extremely well known, from astronomical observations. The response of the solid Earth to these forces is relatively simple and the amplitudes of the body tide harmonics are, to a first approximation, straightforward functions of latitude.

Darwin symbol	***Name***	***Period*** *(h=solar hours; d = days)*
Semi-diurnal tides:		
M_2	Principal lunar	12.42 h
S_2	Principal solar	12.00 h
N_2	Major lunar elliptical	12.66 h
K_2	Luni-solar declinational	11.97 h
Diurnal tides:		
O_1	Principal lunar	25.82 h
K_1	Luni-solar declinational	23.93 h
P_1	Principal solar	24.07 h
Long period tides:		
M_f	Lunar fortnightly	13.66 d
M_m	Lunar monthly	27.55 d
S_{sa}	Solar semi-annual	182.62 d

Table 1: The main tidal harmonics.

The ocean tides are, however, more complicated since the oceans respond dynamically to the tidal forces, resulting in each tidal harmonic having a complicated spatial variation. Numerical hydrodynamic models are required to compute the tides in the oceans and marginal seas. The accuracy of present day models is mainly determined by the resolution of the bathymetry and the computational grid spacing used, as well as the methods employed to model the dissipation mechanisms. Data from altimeter satellites (eg TOPEX-POSEIDON) are now significantly improving our knowledge of the main tidal harmonics in the deep oceans and as such also provide useful constraints in numerical modelling of shallow (marginal) seas.

The ocean tides cause periodic variations in the mass of water in the oceans leading to periodic loading on the Earth's crust. This gives an additional tidal deformation of the solid Earth. This ocean tide loading deformation can produce in excess of 10 cm of vertical displacement in some regions of the world, and there are also associated horizontal displacements which are often, but not always, smaller than the corresponding vertical movement. The IERS standards (McCarthy, 1992) recommend that space geodetic measurements are corrected for vertical and horizontal tidal loading displacements, but GPS software packages do not often include options for these corrections and furthermore, the appropriate loading parameters are often not available for a particular site.

Even GPS measurements at distances 500 to 1000 km from the ocean require ocean tide loading corrections if sub-centimetre accuracy is required. Due to its island situation and position on the continental shelf the UK experiences some of the largest tidal loading effects anywhere in the world, which can be a significant component in GPS measurements. As well as making tidal loading corrections imperative, the UK loading signal provides a useful illustration of the effect on GPS measurements.

COMPUTING OCEAN TIDE LOADING

The computation of OTL deformation requires a model of the ocean tides, which gives the distribution of water mass around the Earth, and an Earth model which accounts for the response of the solid Earth to the load imposed by that mass. For the accuracies needed in space geodetic measurements, the Earth can be assumed to be elastic at tidal periods.

A global integration of the surface mass load, together with a load response function for the Earth, allows the computation of the deformation at any given position. The loading deformation is site dependent as it depends upon the spatial distribution of the ocean tides with respect to the observation site, and both the near and far ocean tides are important. Recently, the main uncertainty in OTL computation has come from errors in the ocean tide models rather than the Earth model. Even for the main tidal harmonics, errors of greater than 10 or 20 % of the ocean tide amplitude were common. For sites within 300 km of the coast, the most common procedure has been to use a global ocean tide model (typically 1^{o} x 1^{o} resolution), supplemented with a higher resolution numerical tide model for the adjacent shallow seas. Recent activity in ocean tide modelling, stimulated by the data from altimeter satellites, has lead to significant improvements in the accuracy of ocean tide models.

Once the OTL displacements at a site have been computed for each of the main tidal harmonics in the ocean tide, the total loading at each epoch of the GPS measurements can be computed. The deformation for each tidal harmonic is uniquely defined by an amplitude and a phase. The latter gives the time of maximum deformation with respect to the phase of the corresponding astronomical tide generating force. Since the astronomy is very well known, the loading can be readily computed from the astronomy for any past or future epoch.

THE EFFECT OF OTL IN THE UK

The spatial variation of the computed M_2 ocean tide loading for the United Kingdom is illustrated in Figure 1. A recent global ocean tide model (Le Provost et al, 1994) has been used for the computations. This model covers all the world's oceans, including the Arctic, and has a very high resolution on the continental shelves. Figure 1 shows the total M_2 loading which is a combination of loading due to the global oceans other than the North Atlantic (1 to 2 mm in amplitude), loading from the North Atlantic beyond the shelf edge (which decreases with distance from the ocean, from an amplitude of about 2.5 cm in north west France to 0.5 cm in the centre of the North Sea) and from the tides on the North West European shelf. It can be seen that the spatial distribution of the total M_2 effect in the UK is complex. There is an area to the south west of the British Isles, where the total M_2 tidal loading is essentially uniform in phase and is over 4.5 cm in amplitude. However, in the eastern English Channel, in the Irish Sea and in north east Scotland, there are points of zero M_2 amplitude i.e. nodal points (called loading amphidromes). The M_2 ocean tide takes just over 6 hours to propagate from the south west of Britain to each of these three areas and therefore the marine tides are

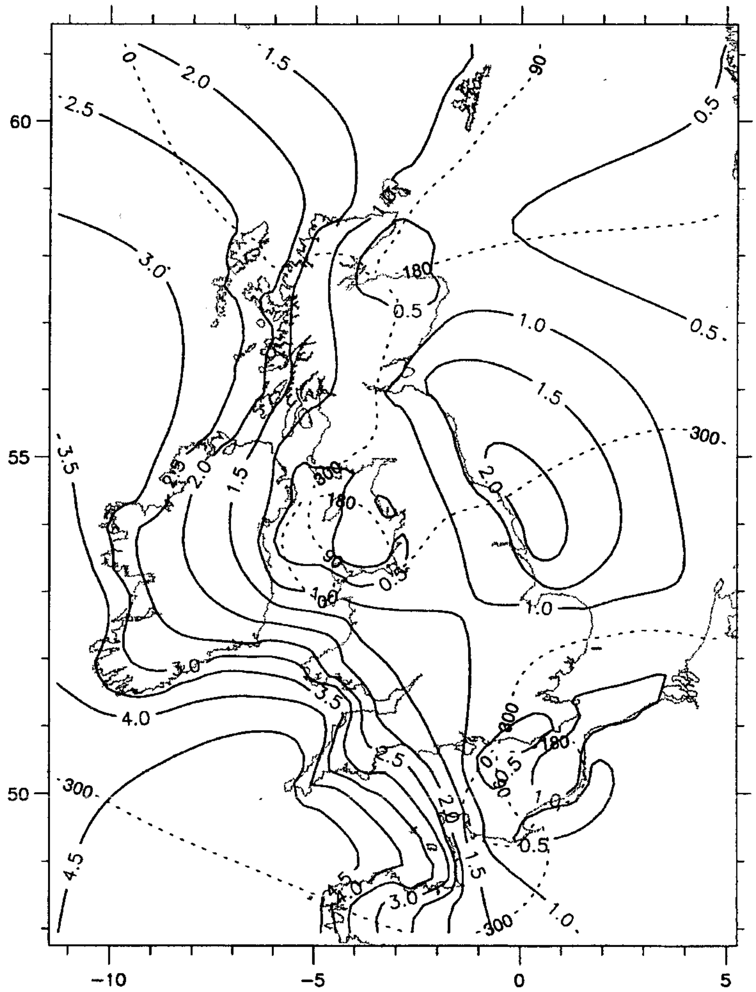


Figure 1: M_2 OTL vertical displacements in cm (solid lines) with phases in degrees (dotted lines) [Baker et al, 1995]

locally in antiphase with the Atlantic tides. Thus, when it is high tide in the ocean to the south west of Britain, all parts of the British Isles are displaced downwards in phase with the ocean tide.

However, in these three areas the local seas are at low tide and hence there is a mass deficit giving local upward displacements. At the nodal points, the loading displacements from the two sources cancel. This spatial variation of the M_2 tidal loading has been verified by observations at eight sites using tidal gravimeters (Baker, 1980). It is clear from Figure 1 that tidal loading is not simply a function of distance from the coast and the amplitude of the marine tide, but is significantly affected by the spatial variations of the phase of the marine tide.

The other tidal loading harmonics have smaller amplitudes than M_2. For example, the S_2 loading has a similar spatial pattern to Figure 1, but is only one third of the magnitude, whilst K_1 varies from 5 mm to 2 mm and O_1 varies from 3 mm to 1.5 mm in amplitude across the UK.

The extreme south west of England, has a very large OTL signal. The four main tidal loading harmonics are shown in Figure 2 for the vertical displacements at Newlyn over the period from 15 to 19 November 1993. It can be seen that M_2 and S_2 are in phase on the 15 November. This is called spring tides and follows the new moon on 13 November. The loading is 12 cm in range and over the next 7 days M_2 and S_2 gradually get out of phase giving a minimum range of tidal loading of 6 cm on 22 November (neap tides, when the moon is at first quarter).

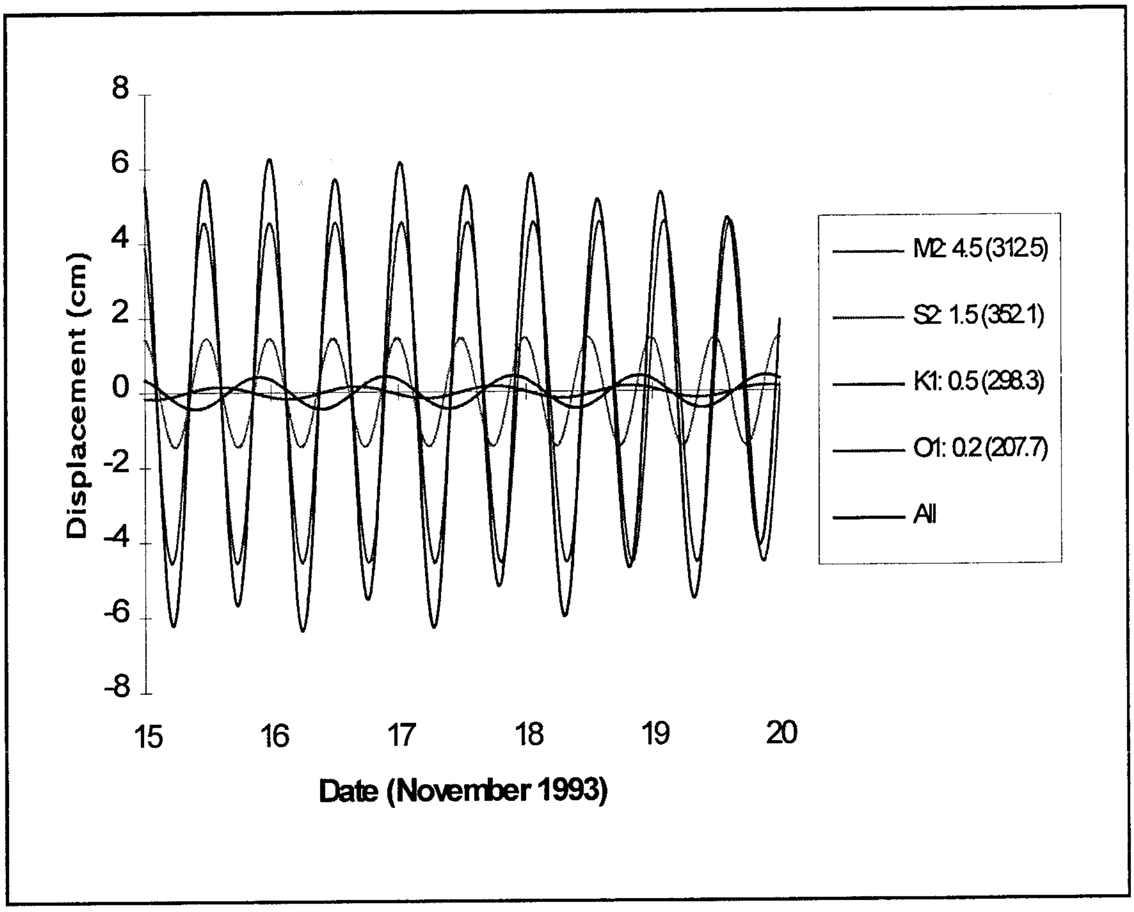


Figure 2: Time series OTL vertical displacements at Newlyn. The amplitudes and phases are given in the legend (amplitudes in cm; phases in brackets in degrees) [Baker et al,1995]

The total loading signal is more easily seen in Figure 3(a), where the spring-neap cycle is clearly evident from the full moon on 30 October to the full moon on 29 November. Although the diurnal tidal loading is very small, it can also be seen in this figure, since it gives the so called diurnal inequality between successive high load tides.

In Figures 3(b) and 3(c), the horizontal displacements due to tidal loading in the north and east directions are given for the same time period. Although smaller than the vertical displacements, they are about 1 cm in amplitude and are therefore significant for many GPS applications.

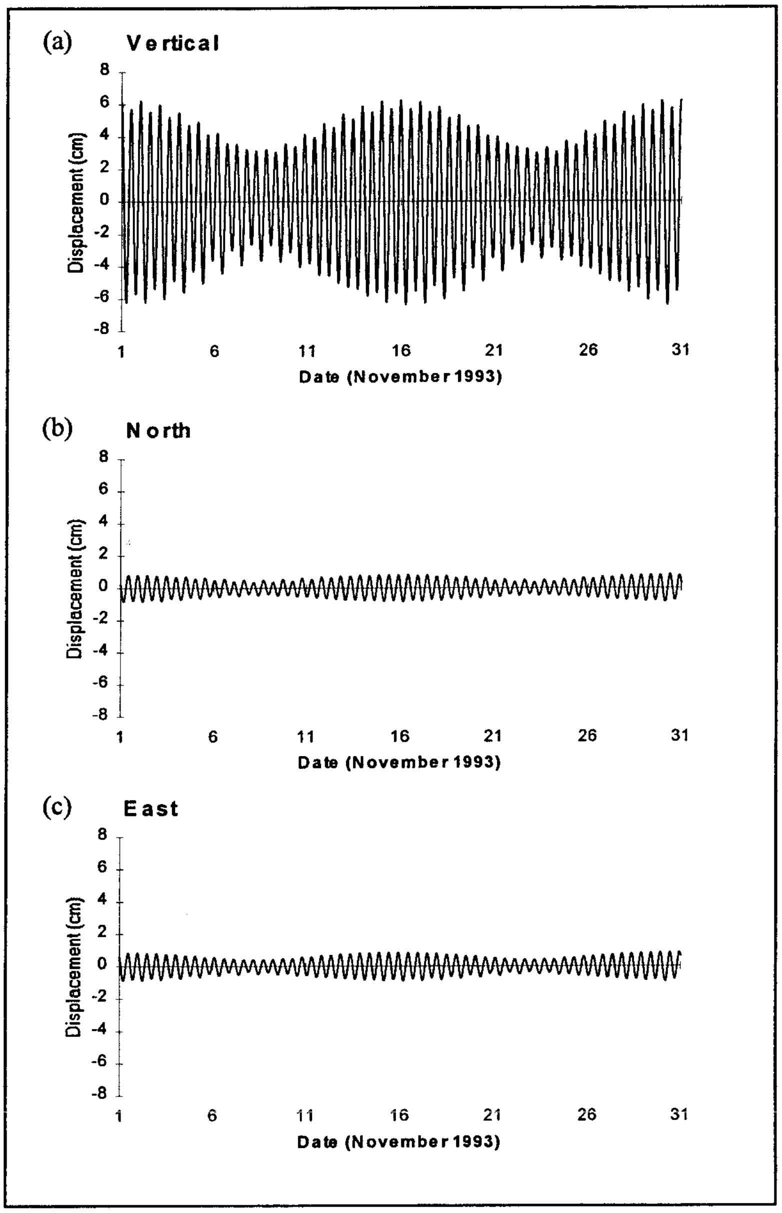


Figure 3: Time series of loading displacements at Newlyn, UK (November 1993). (a) Vertical loading; (b) horizontal loading in the north direction; (c) horizontal loading in the east direction. [Baker et al, 1995]

OTL EFFECTS ON GPS MEASUREMENTS

Portsmouth - Newlyn Baseline

As part of a series of GPS measurements made for monitoring the heights of Tide Gauges in Great Britain and Europe the baseline between Portsmouth and Newlyn, in the South and South West of England respectively, was observed with GPS over a period of several days. From Figure 1 it can be seen that this region of Britain has some of the largest loading gradients, with maximum amplitudes in the extreme South West and virtually no loading effect in the South East. Figure 4 shows the results of the GPS positioning (height) without any OTL corrections applied. The computed OTL differences for the stations are superimposed on the figure, and it can clearly be seen that the very prominent semi-diurnal variations in the GPS results are due to tidal loading.

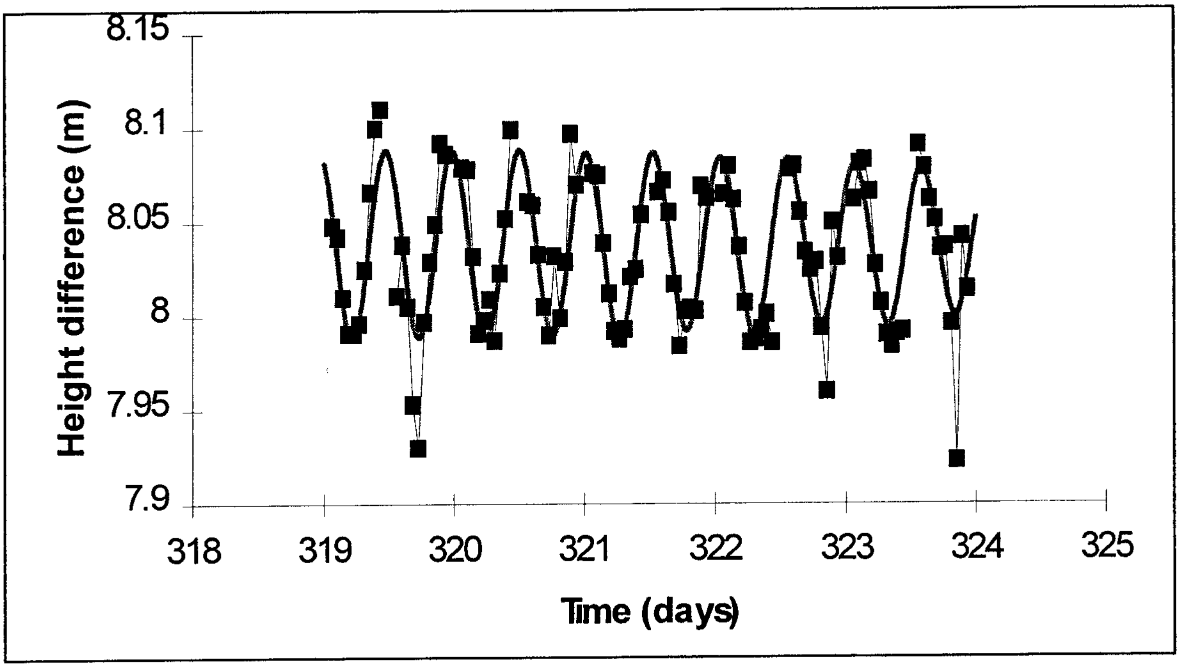


Figure 4: Height of Newlyn relative to Portsmouth (15-19 November 1993) - observed height difference (squares) with the predicted OTL deformation curve superimposed [Curtis, 1996]

After correcting the GPS data for M_2, S_2, K_1, and O_1 tidal loading at the two sites the mean height difference between the stations changed by almost one centimetre (from 8.031m to 8.040m), and the standard deviation of the GPS results was significantly reduced, by almost 50%, from 3.9cm to 2.1cm. The change in the mean height difference between the stations shows that tidal effects are not completely averaged out even when averaging the 3-hourly solutions (from continuous 24hour data) over 5 days.

This effect is compounded when episodic, rather than continuous, data is analysed since the OTL signal is aliased into the displacement signal. This aliasing can be clearly seen in Figure 5, where the same data has been processed using consecutive 6 hour data sets. The squares in the figure represent the 6-hourly solutions which show an aliased periodic variation. The solid

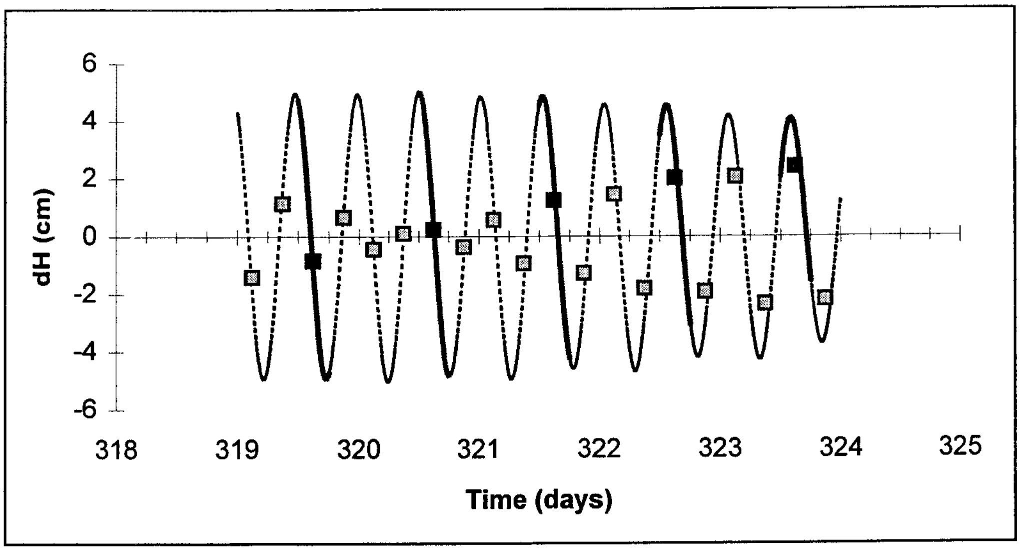


Figure 5: OTL model displacements at Newlyn relative to Portsmouth [Curtis, 1996].

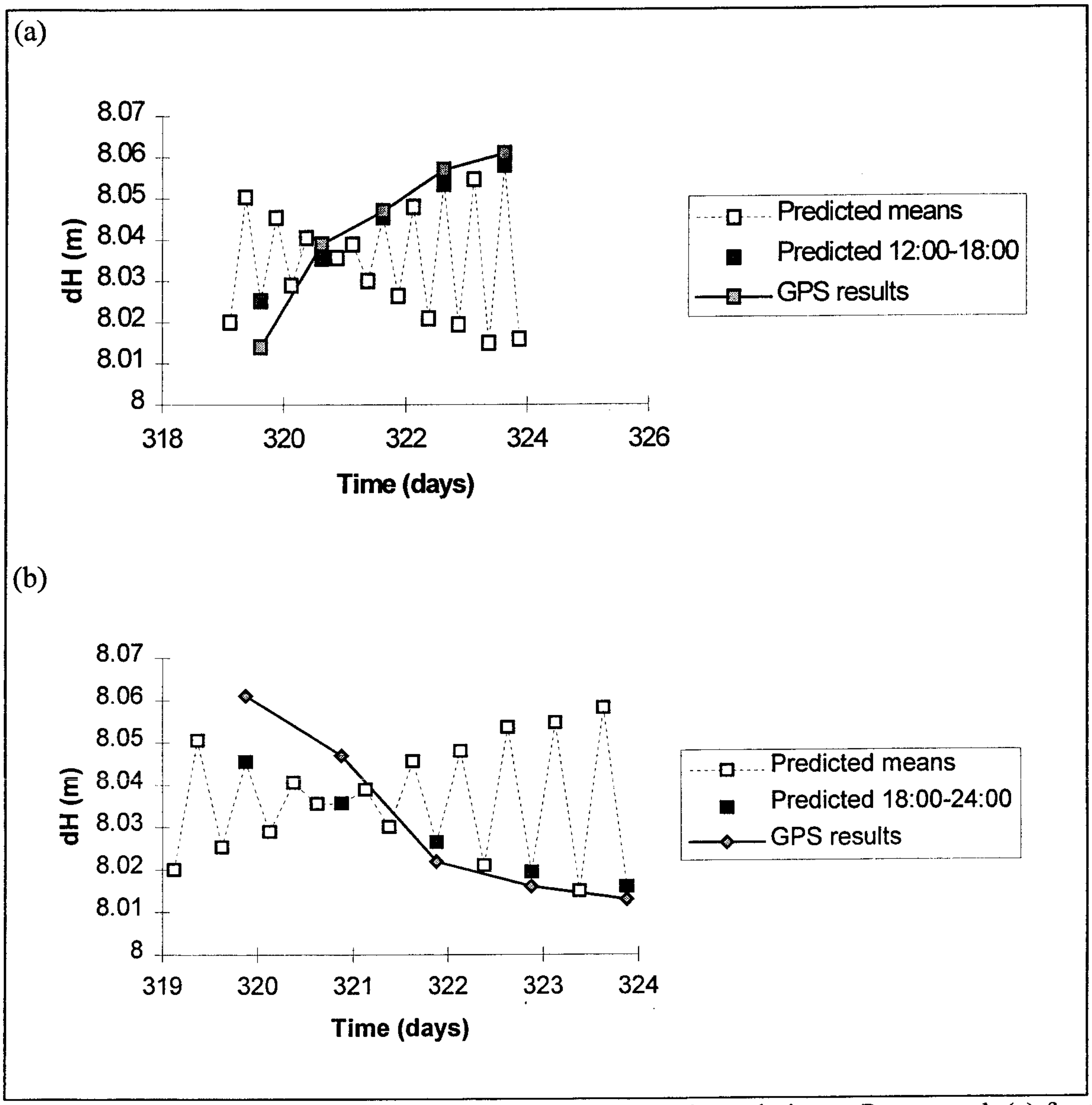


Figure 6: OTL model displacements and GPS results at Newlyn relative to Portsmouth (a) for the period 12:00 - 18:00 and (b) 18:00 - 24:00 on consecutive days [Curtis, 1996].

lines show the displacement actually occurring over the same 6 hour period (12:00 - 18:00) on each consecutive day. The solid squares show the results of using only that 6 hour period each day for the daily solutions, and clearly demonstrate the effect of the aliasing in that an apparent , non-existent, long wavelength displacement is produced from the 6-hourly GPS solutions.

The aliasing can be predicted from the OTL model, and in Figure 6 these predictions are compared against real GPS data solutions for two different 6 hour data periods, namely

12:00 - 18:00 and 18:00 - 24:00. It is apparent that the daily GPS solutions (for the 6 hour data period in question) follow the result predicted from the OTL computations. Furthermore it can be clearly seen that, dependent on which data period is used, an apparent rise or fall is 'observed' by the GPS measurements, both being of the order of 5cm over 5 days. This would obviously be an erroneous result which might be mistaken for a deformation if the OTL effects were not taken into account.

European Tide Gauge Sites

A project (EUROGAUGE), sponsored by the EC, was carried out between 1992 and 1995 to monitor land movements at tide gauge stations along the west coastline of Europe, as a demonstration of the potential for using GPS in activities related to sea-level and climate studies. The tide gauge stations used are shown in Figure 7.

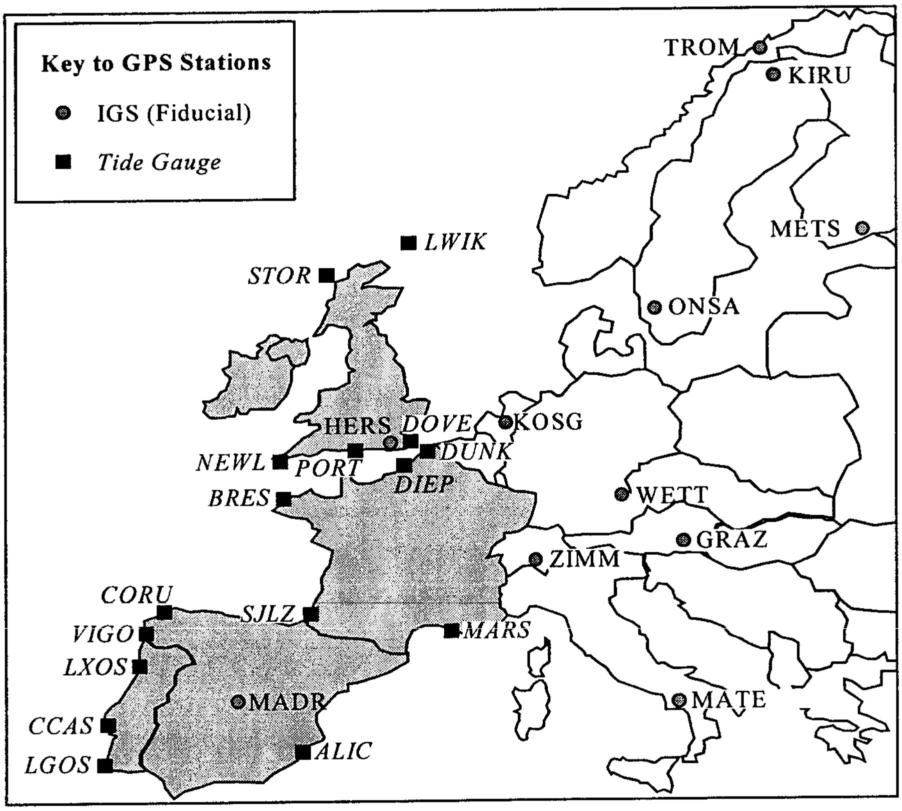


Figure 7: EUROGAUGE tide gauge and reference GPS stations

The analysis of the GPS data from this project included an assessment of the effect of applying, or ignoring, OTL. Figure 8 shows the modelled OTL effects (M_2 only) at all the tide gauge stations. It can be seen that the OTL amounts to about 4cm at Newlyn and Brest, and is in excess of 2cm at 9 of the stations. The data was analysed in 6 hour subsets, and the effect of modelling OTL was assessed. An example of the improvement in the rms of day to day repeatability is also given in Figure 8, where it is shown the improvement was approximately 80% at several stations. There is one anomaly (at Cascais) where the repeatability is worsened significantly. The reason for this is as yet unexplained, but may be due to poor modelling of local tidal effects.

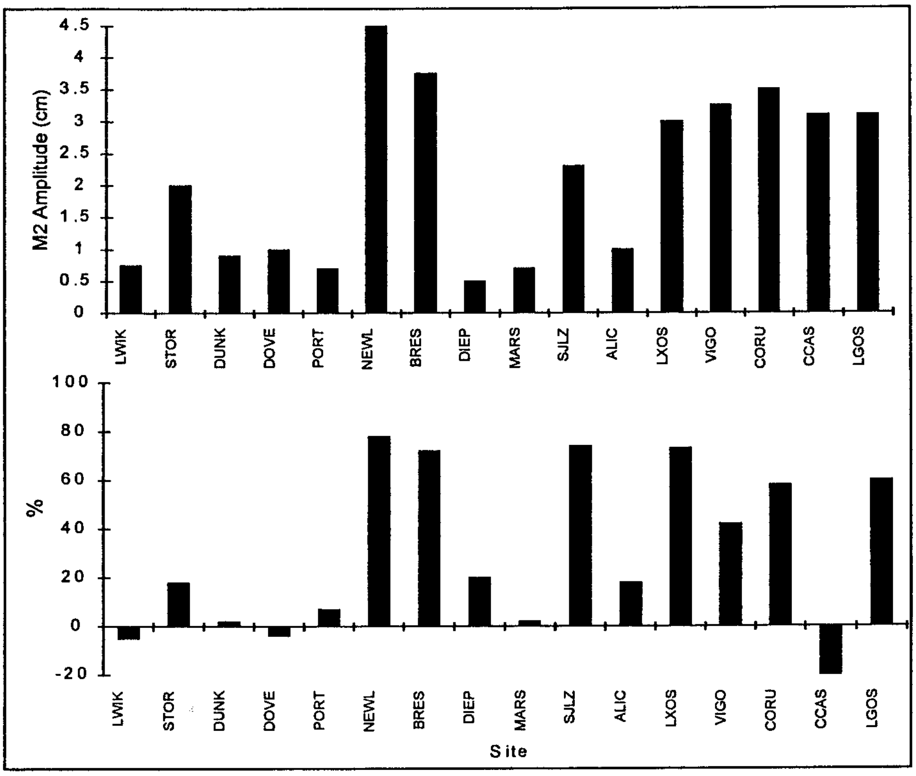


Figure 8: OTL M_2 amplitudes at tide gauge sites and % change in day-to-day repeatability when including OTL corrections [based on Curtis, 1996]

COMPARISON OF OCEAN TIDE MODELS

The fundamental basis of OTL calculation is the ocean tide model used to determine the load. Global models provide the basic information, but need to be supplemented by local (high resolution) models of any shelf seas in the vicinity of the computation area. There have been numerous models derived over the years, but recent global models have benefited particularly from satellite altimetry data, and from much increased computational power. Before the advent of this new generation of models (1995 onwards) there were significant discrepancies between models, which led to uncertainty in OTL corrections. A comparison of several global ocean tide models was undertaken by Curtis (1996), and though early models (eg Schwiderski (1980) and Cartwright and Ray (1991)) showed inaccuracies when compared with ground truth measurements, more recent models eg the purely hydrodynamic model FES 94.1 (Le Provost et al, 1994) and combined (hydrodynamic + altimetry) models such as KMS 95.1 and CSR 3.0 (Anderson et al., 1995) gave much better consistency and accuracy in comparison with tide gauge observations.

As an example, when compared to data from a set of 107 world-wide tide gauges CSR 3.0 showed 87% agreement at 4cm or less (for M_2), with an rms of 1.8cm, compared with 40% for the earlier Schwideski model, which also gave a much larger rms of 3.9cm. When the different models were compared in terms of the (M_2) OTL effect, at 44 global IGS sites, the mean difference in the vertical amplitude was 0.4mm between KMS 95.1 and CSR 3.0 with a maximum difference of 1.6mm. These values confirm that the modern generation of models has the potential to allow OTL computation to millimetric accuracy over most of the world.

CONCLUSIONS

Ocean Tide Loading can produce significant error in GPS measurements if it is not correctly modelled. Although both horizontal and vertical positions are affected, the major error is potentially in height determination where daily variations of approximately 10cm can be observed. The use of episodic measurement campaigns can lead to signal aliasing, resulting in erroneous deformations being observed. The effects of OTL can be mitigated by temporal averaging of data, but even 24 hour data sets will not average out some of the longer wavelength loading harmonics. OTL effects are differenced out from short baseline measurements, but in some parts of the world (including for example the southern UK) the spatial variation in loading effects is rapid leading to significant differential effects over baselines as short as a few tens of kms.

Modern ocean tide models, the fundamental basis of OTL calculations, have been shown to be of a sufficiently high accuracy to allow OTL effects to be calculated to millimetric precision over most of the world. However older tide models can be significantly in error and should be used with caution. All global tide models need to be augmented with local (high resolution) models of shelf sea areas for sufficiently accurate results to be obtained.

ACKNOWLEDGEMENTS

We acknowledge the support of the Natural Environment, and the Engineering and Physical Sciences, Research Councils, the European Union, and the Ministry of Agriculture, Fisheries and Food, for their funding of the work described. We also thank staff at the IESSG for help with the GPS work, and acknowledge the Eurogauge Consortium for use of their data. This paper is based on that originally presented at the FIG XXI International Congress 1998 in Brighton, England (Dodson et al. 1998).

REFERENCES

Anderson, OB, Woodworth, PL and Flather, RA, 1995, Intercomparison of recent ocean tide models, *J Geophys Res, 100,* (C12), p 25 261-25 282.

Baker, TF, 1980, Tidal gravity in Britain: tidal loading and the spatial variation of the marine tide, *Geophys J R astr Soc*, 62, p 249-267.

Baker, TF, 1984, Tidal deformations of the Earth. *Science Progress*, 69, 197-233.

Baker, TF, Curtis, DJ and Dodson, AH, 1995, Ocean tide loading and GPS, *GPS World*, Vol 6, No 3, p 54-59.

Cartwright, DE and Ray, RD, 1991, Energetics of global ocean tides from Geosat altimetry, *J Geophys Res, 96 (C9)*, p 16 897-16 912.

Curtis, DJ, 1996, Ocean tide loading for geodetic applications, *PhD Thesis*, The University of Nottingham.

Dodson, AH, Curtis, DJ and Baker, TF, 1998, Ocean tide loading effects on GPS observations, FIG '98, Commission 5, p 439-451.

Le Provost, C, Genco, ML, Lyard, F, Vincent, P, Canceil, P, 1994, Spectroscopy of the world ocean tides from a finite-element hydrodynamic model, *J. Geophys. Res.*, 99, (C12), p24 777-24 797.

McCarthy, DD, 1992, IERS Standards, *IERS Technical Note 13*, Observatoire de Paris.

Schwiderski, EW, 1980, Ocean tides: I Global tidal equations; IIA Hydrodynamic interpolation model, *Mar Geod*, 3, p 161-217 & 219-255.

Approximation of local geoid surface by artificial neural network

Tomaž Ambrožič, Miran Kuhar, Bojan Stopar, Goran Turk*

Abstract

The geoid surface determination is based on the difference between geoid point positions and reference ellipsoid point positions. The local geoid surface could be modelled by the approximation function, usually the first order, the second order surface or spline function. The least squares collocation may be applied in the above mentioned approximations. The choice of the approximating function depends on the shape of the actual geoid and the number of points where the geoid heights are known. An alternative method of approximation is the use of artificial neural networks (ANN). Unlike in other approximation methods, the neural networks need to be trained to simulate the desired functional relationship. The ANN approximation has been tested on existing astrogeodetic geoid of the chosen area. It has been concluded that ANN may efficiently approximate the real geoid heights on the chosen area. It has been found that the local geoid approximation by the ANN is consistently better than the first order approximation functions and comparable to the second order approximation functions.

1 INTRODUCTION

The Global Positioning System (GPS) presents great potential for variety of applications in the fields of surveying, engineering and geographic information. Many geodetic applications with high accuracy demands were shown. Although the GPS positioning yields three dimensional coordinates, height information is seldom used. The three dimensional capability of GPS has created considerable opportunity for efficient, high accuracy height determination. The height component is less accurate than horizontal positions due to inherent geometrical weakness, and observational error sources which primarily affect the height component. Nevertheless, if appropriate computational strategies are adopted, very accurate heights can be obtained.

It is well known that ellipsoidal height cannot be used like conventional heights derived from levelling. The reason for this is the fact that GPS heights refer to an ellipsoid and not to the geoid, as conventional orthometric heights do. However, in some cases the ellipsoidal height are of practical use. At the moment the main purpose of the use of GPS technology in Slovenia is the establishment of control networks for cadastral and engineering survey tasks where conventional orthometric heights related to mean sea level are essential. The ellipsoidal heights can be used for the determination of orthometric heights if the geoid heights are known with sufficient accuracy.

* University of Ljubljana, Faculty of Civil and Geodetic Engineering, Jamova 2, 1000 Ljubljana, Slovenia
e-mail: tomaz.ambrozic@guest.arnes.si, mkuhar@fagg.uni-lj.si, bstopar@fagg.uni-lj.si, gturk@fagg.uni-lj.si

2 ORTHOMETRIC, ELLIPSOID AND GEOID HEIGHTS

The *orthometric heights H* may be determined by GPS levelling if *ellipsoidal heights h* and *geoid heights N* are available. There are many theories, methods and papers dealing with this topic. But still, the determination of orthometric heights with GPS remains a big challenge of modern geodesy.

The well known equation relates these three quantities (Fig. 1)

$$H = h - N. \qquad (1)$$

Equation can also be applied for height differences

$$\Delta H = \Delta h - \Delta N. \qquad (2)$$

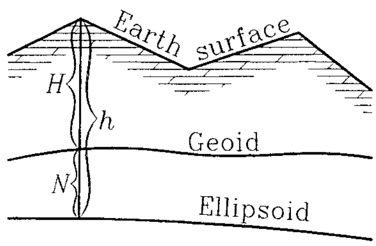


Fig. 1 Earth surface, geoid and ellipsoid

With the presence of geoid model, the solution is straightforward. It follows that if GPS derived orthometric height accuracy is to become comparable with the levelling accuracy, the geoid heights accuracy must be improved accordingly. This could be done in several ways. Common to all methods is the improvement of geoid heights from the existing model. For example, the least squares adjustment, where geoid heights are taken as observations, could be performed (Fiedler, 1992, Potterfield, 1994), or a new geoid type, the so called GPS–gravimetric geoid, could be introduced (Kenyeres, 1995). Of course, there are some other ways of improving heights from the geoid model.

But what shall we do if there is no geoid model, or the existing geoid model is not appropriate? For the time being in Slovenia exists only one geoid model with the limited applicability. This is purely astrogeodetic geoid and covers the territory of Slovenia and part of Croatia (Čolić et al, 1993). It is a relative solution because of local orientation of Slovenian trigonometric network and its accuracy is at the decimetre level. It can be used for projects spanning over larger area in the country but it is unsuitable for the local GPS networks covering limited area with the diameter of up to 20 km. In that case the model of the local geoid has to be derived. In local networks, GPS levelling can only be employed if a local geoid can be provided which meets the accuracy specification of both orthometric height *H* and ellipsoidal height *h*. Considering the capabilities of levelling and GPS, an accuracy level of cm or even better could be aimed at.

In order to determine local geoid we have tried two approaches: one is geometric modelling of the local geoid surface and the second is the use of artificial neural networks. Depending of the data available and the accuracy of existing control networks, both procedures give quite satisfactory results.

3 LOCAL GEOID SURFACE FITTING MODEL

This method involves the development of a local geoid surface model using a surface fitting procedure. If a number of height control stations exist in a GPS network which have both ellipsoidal and orthometric height, the geoid height can be found directly from the equation (1). Three or more non-colinear control stations will define a surface of geoid from which values for N can be determined for other stations.

The geoid surface could contain datum errors due to the adoptions of incorrect ellipsoidal coordinates for the origin station used in the GPS relative positioning. In order to avoid this datum problems it is well recommended to connect the GPS network to the nearest station with good geocentric coordinates.

Local geoid surface fitting model is accomplished by taking N as a function of the position of each height control stations in the network. The surface model could take the following forms:

$$N = N(y,x) = A\,y + B\,x + C \tag{3}$$

$$N = N(y,x) = A\,y + B\,x + C + E\,y^2 + D\,yx + F\,x^2 \tag{4}$$

$$N = N(y,x) = A\,y + B\,x + C + E\,y^2 + D\,yx + F\,x^2 + G\,y^3 + H\,y^2x + I\,x^2y + J\,x^3 + \ldots \tag{5}$$

where variables y and x represent easting and northing in a local plane coordinate system. The surface model could be also written in the form where geoid height differences are used instead of geoid heights itself.

If the coordinates in equations (3), (4) and (5) refer to the centre of gravity of the area covered by GPS network, coefficients have its geometrical interpretation (Illner et al, 1995). C represents the parallel shift between ellipsoid and local geoid surface. The linear terms with coefficients A and B represent the difference in the inclination of the tangent plane to the ellipsoid and local geoid surface. Second degree coefficient represent the difference in the curvature of these two surfaces. If more than three points exist, the plane (3) is overdetermined and the coefficients A, B and C are evaluated by least squares method. The size of the residuals at each control point and the standard deviation give an indication of how well points fit the plane. Computed local geoid surface model can be treated as a trend surface and serve as the first approximation. The shape of the geoid surface can be refined by the least squares collocation.

Degree of the polynomial used for trend surface depends on the number of available height control points. The more points we have the higher degree polynomial we could use. But if there are more control points available and low degree polynomial is chosen, then overdetermined solution could be solved. Depending on the number of control points, it is possible to generate more complex surfaces such as trigonometric functions or bicubic spline functions. However, these methods could become unstable when the network points are irregularly spaced (Holloway, 1988).

The number of points as well as their location and distribution over the area covered by the network is very important. It is recommended that height control points are well distributed geometrically throughout the project area, e.g. corners of project area. The number of height control points dictates the choice of the model.

As an alternative to existing geometric models the *artificial neural network* (ANN) may be used. In the following section the basics of ANN are described.

4 ARTIFICIAL NEURAL NETWORK

The basic ideas and the motivation for the early developments of ANN has been the study of the structure and processes in human brain. Although the complexity of the structure of ANN will probably never reach the human's brain, there are similarities between human's brain and ANN. They both have units called *neurons* which are mutually connected. Similarly to human's brain the ANN has to be thought or trained. There are two types of learning procedures: supervised in which the questions and answers are known and the ANN has to learn the correct answers, and the unsupervised learning where the answers are not known. The former is used in our research.

One of the common definitions of ANN is: ANN is a network of simple units which operate locally. The units are connected by connections which may reduce or amplify the signal from one unit to another. Each unit receives signals from other units, processes these signals and transmits the signals to other units.

There are several types of ANN geometry. A review of different ANN is given in several papers, books and internet sites (e.g. Lippmann, 1987, Müller, 1995, Sarle, 1998. The *multi-layer feed-forward network* is usually chosen if functional approximation is sought. Since it is our aim to approximate the geoid heights, the multi-layer feed-forward network was chosen in our research.

4.1 Multi-layer feed-forward network

The geometry of a multi-layer feed-forward neural network is shown in Fig. 2. Input units are connected to the first layer of hidden units which are further connected to the units of the second hidden layer. The units of the last hidden layer are connected to the output units. The multi-layer feed-forward networks are usually employed as the approximators of the unknown functional relation. In fact, it has been shown by Hornik et al, 1989 and Funahashi, 1989 that any continuous functions may be accurately approximated by the multi-layer feed-forward neural network.

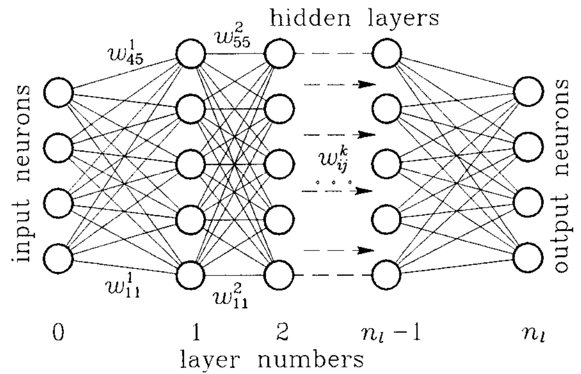


Fig. 2 Multi-layer feed-forward neural network

Each connection between the units is represented by its weight w_{ij}^k, where index i corresponds to the unit number of $(k-1)^{\text{th}}$ layer, while index j corresponds to the unit number of k^{th} layer. The number of input layer is 0, whereas the number of output layer is equal n_l. The signals travel in only one direction, i.e. from the input layer toward the output layer. The value of a unit is multiplied by the corresponding weight and added to the value of signal in the unit of the next layer

$$y_i^k = \mathrm{f}\left(y_i'^k\right) = \mathrm{f}\left(\sum_{j=1}^{n_{k-1}} w_{ij}^k y_j^{k-1}\right). \tag{6}$$

The activation function f(.) is needed in order to enable modelling of an arbitrary non-linear relation between input and output units. Although many different functions could be successful activation function, usually a differentiable and bounded function is used. One of the usual choices of activation function is a sigmoid function $1/(1+e^{-y})$, $\tanh(y)$, or Gaussian. The results of the neural network depend on the values of the weights w_{ij}^k which have to be determined by the learning (training) procedure.

As mentioned above the supervised learning has been employed in our research. Therefore, the values of input units and corresponding output units are known. A set of known input and output values is termed as *input-output pair.*

All input-output pairs are usually divided into three sets. The first is termed as *learning* or *training set* which is used to determine the connection weights w_{ij}^k. The second, named *validation set* is used to choose the optimal parameters of neural network, i.e. number of hidden layers and number of units in each hidden layer. Finally a chosen and tought neural network is tested, using a *testing set* of data.

When the learning procedure is completed, meaning that the neural network performs adequately for all input-output pairs in the learning set, the neural network is assessed on the validation set of input-output pairs and the optimal neural network is chosen.

For numerical reasons the values of input and output units have to be normalized. The normalization of the values of output units depend on the range of activation function. Usually, the linear transformation works well, although sometimes a non-linear transformation may help if the data is clustered.

The supervised learning is in fact a general optimization problem in which the minimum of the error E_p defined by the equation

$$E_p = \frac{1}{2}\sum_{i=1}^{n_o}\left(t_{pi} - y_{pi}^{n_l}\right)^2 , \tag{7}$$

is sought. We have to find weights w_{ij}^k which give the minimum error E_p. However the problem is not easy to solve since function E_p of many variables w_{ij}^k is nonlinear and may have a large number of local minima. In equation (7) t_{pi} is the actual output whereas $y_{pi}^{n_l}$ is the output evaluated by the neural network; n_o is the number of output units.

There are two essentially different approaches: *error back-propagation algorithms* which is basically a gradient method and *genetic algorithms* which is in fact a stochastic search (Goldberg, 1989, Yip et al, 1993). There are many variations and combinations of the above mentioned method (e.g. Burshtein, 1993). If the number of weights is relatively small, the gradient method is a good choice. The error back-propagation (or "generalized delta rule" as it was termed by its authors Rummelhart and McClelland, 1986) is a gradient method in which the weights are changed for a chosen step size in the direction of maximum descent for each input-output pair. However, there is always a possibility of finding only a local minimum which may not give satisfactory set of weights. One solution of this problem is simply to run the error back-propagation procedure for different starting points and then choose the best result. If the number of weights is larger, the genetic algorithms are better. Genetic algorithms are often referred to as *simulated annealing* methods.

The procedure is repeated for each input-output pair until the error is smaller than prescribed for all input-output pairs. If the prescribed error is too small, *overfitting* may occur. Overfitting means that the neural network may reproduce input-output pairs used in learning procedure, but fails to generalize them and may produce erroneous results, if some values of the input units are changed.

There are two major difficulties when using error back propagation: it is almost impossible to choose the optimal *step size* ΔW, and quite often the procedure converge to a local minimum. If the step size is too large, we may overshoot the minimum. On the other hand, if the step size is too small, the convergence is very slow. Both difficulties may be overcome by different procedures with adaptive step size (e.g. Janakiraman et al, 1993), or with the introduction of inertial term (e.g. Lippmann, 1987, Rumelhart et al, 1986).

Genetic algorithm or evolutionary simulating annealing is presented in various forms and variations (e.g. Baba, 1989, Goldberg, 1989). Many hybrid algorithms have been developed in recent years utilizing simulated annealing and back-propagation algorithms (e.g. Treadgold et al, 1998).

The parameters of the optimal neural network is problem dependent. One of the methods to choose the right network is by using the validation set to determine which one performs best. However, some general guidelines can be given. If the number of units is very large the learning procedure may be very slow, since each forward calculation takes a substantial computational effort. Although larger networks are usually able to learn the sought relationship, this may sometimes be a drawback. A large network may easily reproduce the training set of input-output pairs but fails to generalize yielding to a poor testing performance. Networks with insufficient units may have problems to learn properly during the learning procedure.

5 NUMERICAL EXAMPLES

As an alternative to linear or second order approximation the artificial neural network is used to approximate the geoid heights as a function of y and x coordinate. Therefore, the input-output pair consists of horizontal coordinates y and x as input variables, and geoid height N as single output variable.

Two cases are considered. In the first the ANN is trained and validated by the geoid heights of existing astrogeodetic geoid. The relatively large rectangular area (40 by 50 km) in western Slovenia is chosen (Fig. 3). This area is characterised by relatively large non-linear changes in geoid heights from –0.35 m to 1.70 m. In the second case a smaller region in the vicinity of Sežana is studied (Fig. 3). Geoid heights at 37 points were established from levelling and GPS results.

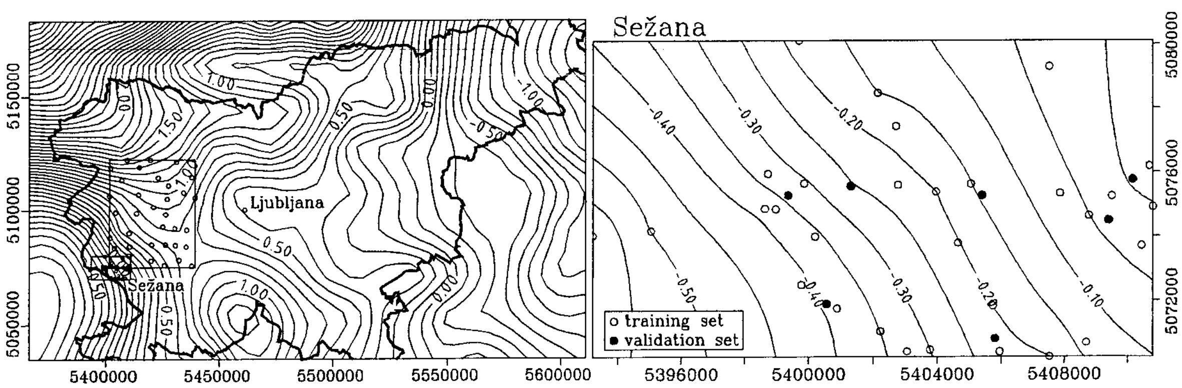


Fig. 3 Approximate geoid heights in Slovenia

The ANN is trained by the error back propagation procedure. The iterations are repeated until the relative difference between ANN prediction and target value of geoid height is lower than 5% for all input-output pairs of training set.

5.1 Western Slovenia

Since the reference geoid heights of existing astrogeodetic geoid can be determined at any number of points, several cases with different number and distribution of points are studied. Firstly, the points were distributed in a simulated levelling networks consisting of 36, 13 and 6 points which represent input-output pairs of ANN training. The set of 36, 13, and 6 points correspond to one point on 56 km^2, 154 km^2 and 333 km^2, respectively. In addition a 36 input-output pairs which are regularly distributed in a rectangular equidistant mesh (12 by 10 km), and the 36 input-output pairs which are completely randomly distributed over the region, were studied. Overall, five different training sets were considered.

Several one, two and three hidden layers neural networks were trained. The smallest ANN consisted of only five neurons in a single hidden layer. Its geometry can be represented 2 – 5 – 1, where 2 stands for the number of input neurons, 5 for the number of units in hidden layer and 1 for the number of output neurons. In this case there are $2 \cdot 5 + 5 \cdot 1 = 15$ weights to be determined. The geometry of the largest ANN considered here is 2 – 50 – 50 – 50 – 1, meaning that there are three 50 units hidden layers. The number of weights in this case is $2 \cdot 50 + 50 \cdot 50 + 50 \cdot 50 + 50 \cdot 1 = 5150$. In this case, the error function E_p is a function of 5150 variables, and the minimum of this function is to be found.

The validation of different neural networks is performed on unusually large validation set which consists of 99 points regularly distributed in the equidistant mesh. The distances between the points of validation set is 5 km in both E–W and N–S direction. The validation is based on several statistics: the range of the differences between ANN prediction and the target values, the mean of the differences, the standard deviation, and the mean of absolute values of the differences. It was found that the neural network 2 – 50 – 40 – 1 performed better than the others and was therefore chosen for this particular case. The results of the approximation by linear and second order equation as well as ANN results are summarised in Table 1.

It can be seen that the distribution of 36 input-output pairs is very important. The results obtained by completely randomised training set performed worse than the other two. It can also be concluded that second order approximation and ANN prediction give very similar results, whereas the linear approximation is clearly the worse choice.

Approximation type		Min. (m)	Max. (m)	Mean (m)	St. dev. (m)	Abs. Mean (m)
Simulated	ANN	–0.216	0.108	0.000	0.055	0.039
Levelling	Linear	–0.501	0.538	0.020	0.200	0.152
Network	2nd order	–0.176	0.115	0.002	0.056	0.043
Random	ANN	–0.310	0.215	–0.015	0.080	0.052
	Linear	–0.512	0.567	–0.004	0.205	0.155
	2nd order	–0.250	0.087	–0.017	0.063	0.046
Regular	ANN	–0.161	0.141	–0.003	0.063	0.053
Mesh	Linear	–0.468	0.452	–0.009	0.191	0.147
	2nd order	–0.136	0.121	–0.002	0.055	0.044

Table 1:Comparison between ANN, linear and second order approximations in three different 36 points training sets

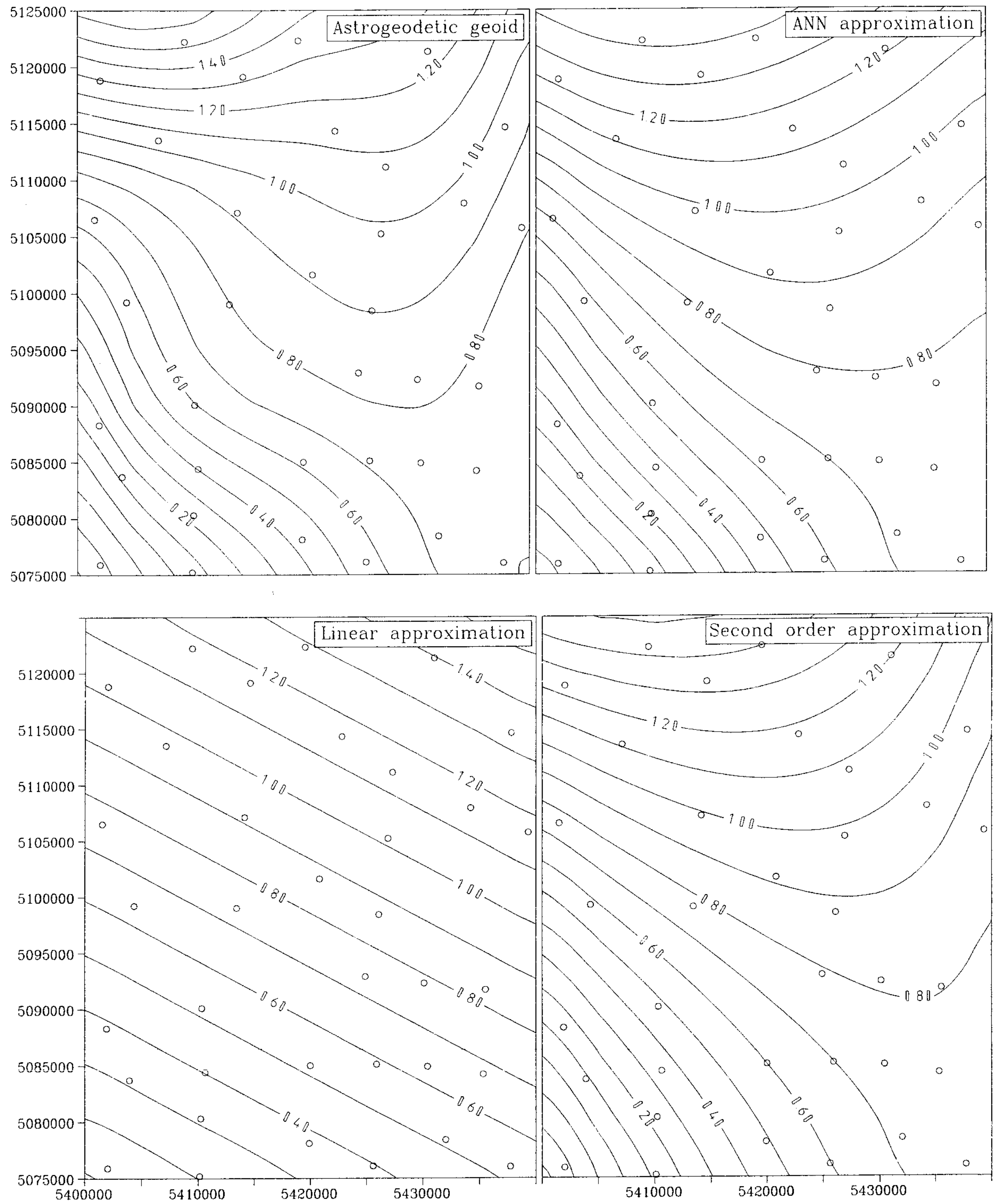


Fig. 4 Astrogeodetic geoid and ANN, linear and second order approximations

The astrogeodetic geoid as well as its ANN, linear and second order approximations are shown in Fig. 4. The circles represent the training set of ANN training and the position of points used in linear and second order approximation by least squares.

The optimal geometry of the ANN is 2 – 50 – 40 – 1 also in the cases where only 13 and 6 input-output pairs were used. The results obtained by ANN, linear and second order model are shown in Table 2.

Approximation type		Min. (m)	Max. (m)	Mean (m)	St. dev. (m)	Abs. mean (m)
13 points	ANN	–0.293	0.364	0.022	0.097	0.074
	Linear	–0.408	0.608	0.045	0.202	0.157
	2nd order	–0.108	0.180	0.015	0.064	0.053
6 points	ANN	–0.290	0.378	0.025	0.145	0.117
	Linear	–0.508	0.670	0.052	0.216	0.162
	2nd order	–0.288	0.393	–0.002	0.151	0.119

Table 2: Comparison between ANN, linear and second order approximations in 13 and 6 points training set

From tables 1 and 2 may be seen that larger training sets give more accurate results. The accuracy of results obtained by ANN and second order approximation is approximately the same, whereas the accuracy of linear model is lower.

5.2 Actual geodetic network in Sežana region

In the region nearby the town of Sežana in SW Slovenia, the geodetic network was established. Geometric levelling and GPS measurements were performed. Therefore, the actual geoid heights are known in 37 points. Among 37 points 7 were randomly selected to form a validation set. The remaining 30 were employed in training procedure (Fig. 3).

The successfulness of the ANN approximation was compared to the results obtained by linear and second order models. The results are shown in Table 3. It can be seen that the ANN performed similarly to the linear model and worse than the second order model. These results were expected since the area of investigation is relatively small and the shape of the geoid heights surface is relatively simple and easily approximated by second order model.

Approximation type		Min. (m)	Max. (m)	Mean (m)	St. dev. (m)	Abs. mean (m)
30 points	ANN	–0.016	0.025	0.003	0.011	0.008
	Linear	–0.020	0.013	–0.005	0.011	0.009
	2nd order	–0.020	0.001	–0.008	0.006	0.005

Table 3: Comparison between ANN, linear and second order approximations

6 CONCLUSIONS

The geoid heights may be approximated by different functions using least squares method. An alternative method which employs ANN is presented here. Both methods are satisfactory if the accuracy requirements are not too severe. The accuracy of few cetimeters is easily obtainable by both methods.

Presently, no perfect solution for the accurate determination of geoid heights exists. Number of arguments may be offered to support or reject these approaches of orthometric heights determination in local GPS networks. This paper presents one such tool in an attempt to benefit from rapid technological progress and suggests a need for further investigations.

REFERENCES

1. Baba, N., 1989, A New Approach for Finding the Global Minimum of Error Function of Neural Networks, Neural Networks, Vol. 2, pp. 267–273.
2. Burshtein, D., 1993, Neural Network Training using the Simulated Annealing Method, World Congress on Neural Networks, WCNN '93, Portland (Or), pp. III-397–III-400.
3. Čolić, K., Bašić, T., Petrović, T., Pribićević, B., Ratkajec, M., 1993, Improved Geoid Solution for Slovenia and Part of Croatia, Proceedings of the 7th International Symposium "Geodesy and Physics of the Earth", Potsdam '93, pp. 141–144.
4. Fiedler, J., 1992, Orthometric Heights from Global Positioning System, Journal of Surveying Engineering, Vol. 118, pp. 70–79.
5. Funahashi, K., 1989, On the Approximate Realization of Continuous Mappings by Neural Networks, Neural Networks, Vol. 2, pp. 183–192.
6. Goldberg, D. E., 1989, Genetic algorithms in search, optimization and machine learning, Addison–Wesley, Reading, MA.
7. Holloway, R. D., 1988, The Integration of GPS Heights into the Australian Height Datum, Unisurv reports S–33, School of Surveying Engineering, Kensington.
8. Hornik, K., Stinchcombe, M., White, H., 1989, Multi-layer Feed-forward Networks are Universal Approximators, Neural Networks, Vol. 2, pp. 359–366.
9. Illner, M., Jäger, R., 1995, Integration von GPS–Höhen ins Landesnetz – Konzept und Realisierung in Programmsystem HEIDI, AVN, Vol. 102, pp. 1–18.
10. Janakiraman, J., Honavar, V., 1993, Adaptive Learning Rate for Increasing Learning Speed in Backpropagation Networks, World Congress on Neural Networks, WCNN '93, Portland (Or), pp. IV-378–IV-381.
11. Kenyeres, A., 1995, GPS-levelling – Practical Approach, Proceedings of the 3rd International Seminar on "GPS in Central Europe", Penc '95, pp. 413–422.
12. Lippmann, R. P., 1987, An Introduction to Computing with Neural Nets, IEEE ASSP Magazine, Vol. 4, No. 2, pp. 4–22.
13. Müller, B., Reinhardt, J., Strickland, M. T., 1995, Neural Networks, An Introduction, Springer–Verlag, Berlin.
14. Potterfield, M., 1994, Accurate Orthometric Heights from GPS: Combined Network Adjustment Using GPS, Differential Leveling and Correlated Geoid Models, International Geoid Service, Bulletin N.3, pp. 19–33.

15. Rumelhart, D. E., McClelland, J. L., 1986, Parallel Distributed Processing, Volume 1: Foundations, The MIT Press, Cambridge, MA.

16. Sarle, W. S., ed., 1998, Neural Network FAQ, Periodic posting to the Usnet newsgroup comp.ai.neural-nets, URL: ftp://ftp.sas.com/pub/neural/FAQ.html.

17. Treadgold, N. K., Gedeon, T. D., 1998, Simulated Annealing and Weight Decay in Adaptive Learning: The SARPROP Algorithm, IEEE Transactions on Neural Networks, Vol. 9, No. 4, pp. 662–668.

18. Yip, P. P. C., Pao, Y.-H., 1993, A fast universal training algorithm for neural networks, World Congress on Neural Networks, WCNN '93, Portland (Or), pp. III-614–III-621.

Computation of postglacial land uplift from the three precise levellings in Finland

Jaakko Mäkinen and Veikko Saaranen
Finnish Geodetic Institute
P.O.Box 15, FIN-02431 Masala, Finland
tel +358 9 255550
fax +358 9 29555200
Jaakko.Makinen@fgi.fi,Veikko.Saaranen@fgi.fi

SUMMARY OF RESULTS

We have calculated vertical velocities and their change in time using the First, Second and Third Levellings of Finland. Their central epochs in the common part of the network are 1901, 1945 and 1985. Models with and without simultaneous determination of heights give the same results for velocities, but different error estimates. Observed velocity change in time is statistically significant, but the significance could just as well be explained by assuming that the loop misclosures underestimate levelling error. It also can be shown to depend on the interpretation of the empirical covariances between the levellings. We therefore consider the evidence inconclusive and further research is required. A new land uplift map, drawn from all three levellings assuming constant velocities in time, reproduces the main features of earlier maps from First and Second Levellings only, but does not support some spatial irregularities in them.

COMPLEMENTARY REMARKS

A full account of our methods and the detailed results can be found in two earlier papers (Mäkinen and Saaranen 1998, 1999). The first paper also contains a complete list of references. Here we present an update of the uplift map with a short description of the data. The update is due to new observation material from the Third Levelling, which now has progressed farther to the north.

The First Levelling of Finland was performed by the National Board of Public Roads and Waterways in 1892–1910 (Blomqvist and Renqvist, 1910). It covered Finland up to the latitude 65°N. The Second Levelling of Finland was performed by the Finnish Geodetic Institute. It was started in 1935, and by 1955 it covered the area of the First Levelling (Kääriäinen, 1966). In Lapland the network was measured 1953–1972. Parts of it were relevelled in 1973–1975 to get more information on land uplift (Takalo and Mäkinen, 1983).

The Third Levelling of Finland started in 1978. By 1993 it covered the area of the First Levelling. Plans call for completion in 2002.

We work with two different data sets. The first one consists of those lines and bench marks which belong to all three levellings. In this *common network* we can solve, not only for velocities, but for velocity change with time as well. The *maximum network* consists of the common network plus those bench marks which belong to any two of the three levellings. In it we can solve for velocities.

It is steadily expanding as the Third Levelling. progresses. Figure 1 and Table 1 give an overview of the network status:

Repeated levellings only give velocity differences between points. We join them to the tide gauge result by Vermeer et al (1988) at Hanko (cf. Figure 1), where they obtained the land uplift rate 2.73 mm/yr relative to mean sea level. For comparisons at other tide gauges see Mäkinen and Saaranen (1998). Figure 2 shows the updated land uplift map.

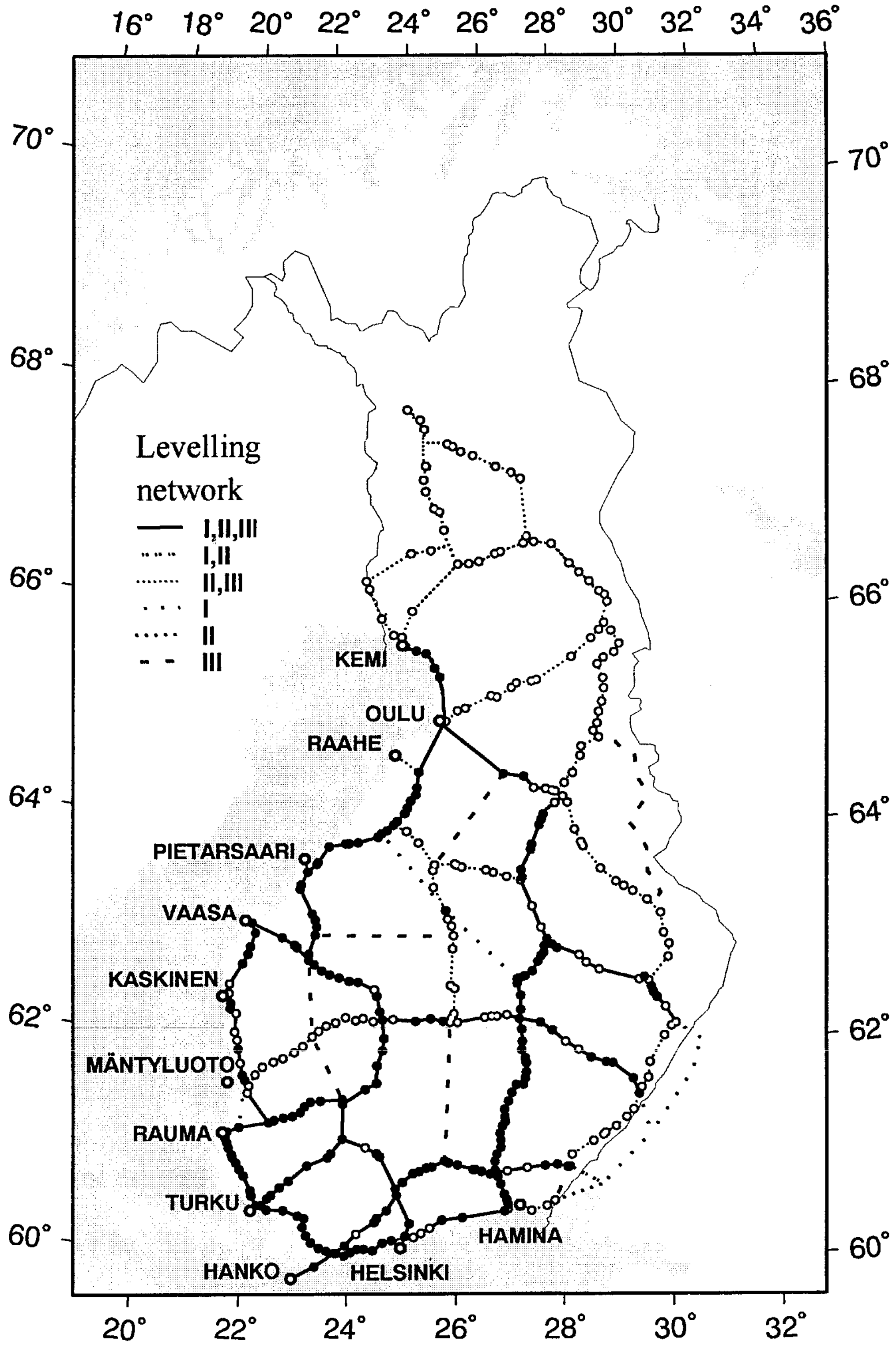


Figure 1. The levelling networks used in this paper. Third Levelling data includes field work up to 1998. Bench marks which belong to all three levellings are represented by solid circles, bench

marks which belong to two levellings are represented by open circles. The maximum network consists of all bench marks and lines. The common network consists of the bench marks and levelling lines represented by solid circles and solid lines, plus two lines in the southeastern part of the network, which run along different routes but can be used between their end points. Tide gauges are represented by large open circles and named.

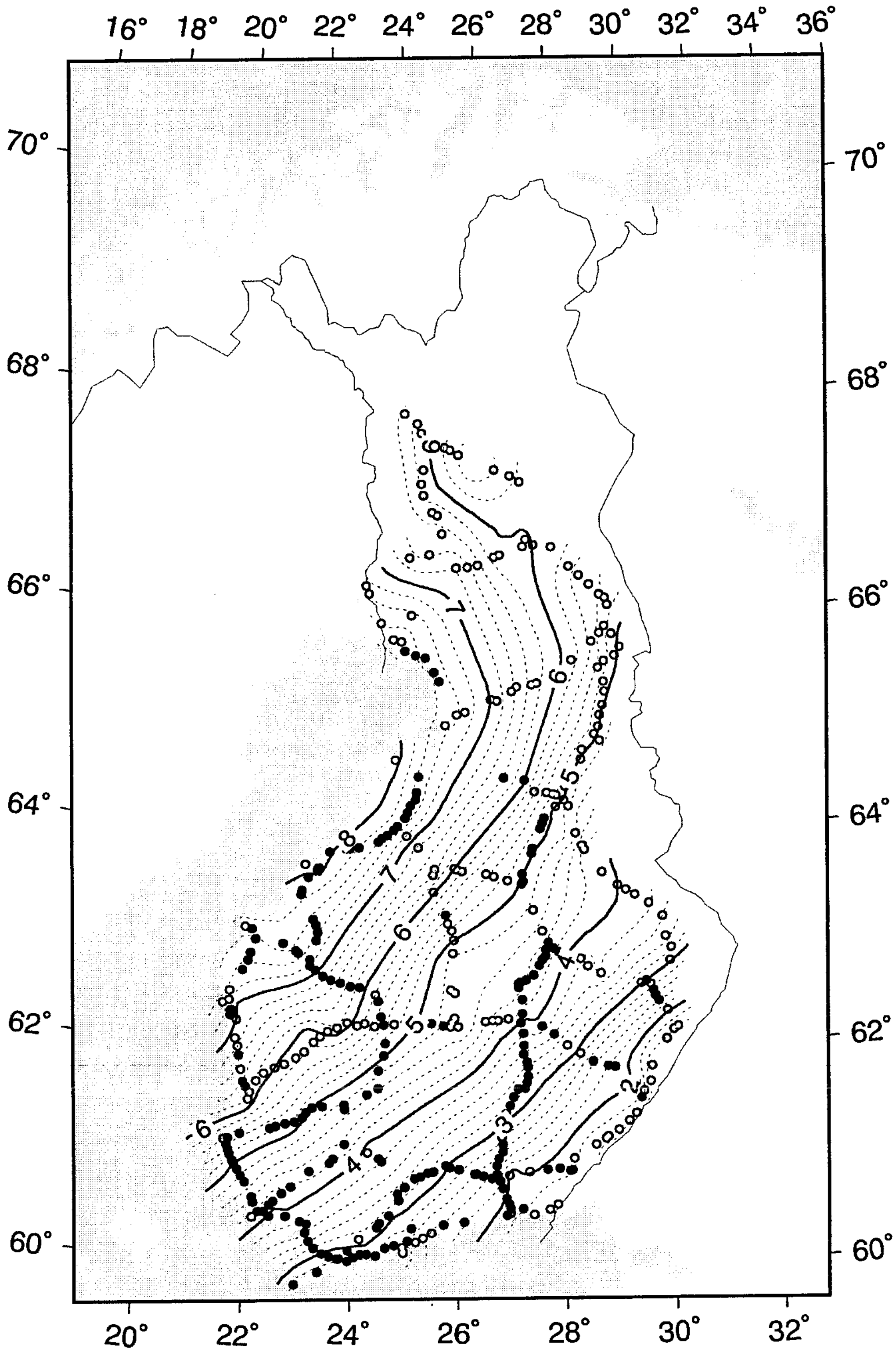


Figure 2. Land uplift relative to mean sea level in mm/yr. Velocity differences between bench marks were computed in the maximum network of Figure 1 using all three levellings. They were

joined to the tide gauge result by Vermeer et al (1988) at Hanko (Figure 1). Standard errors of velocity differences in the network are less than 0.6 mm/yr (one-sigma).

Table 1. Network statistics. The precision figures are the apriori estimates from loop misclosures. Both they and the mean epochs refer to the common network. The corresponding table of Mäkinen and Saaranen (1999) quoted erroneously epochs referring to the maximum network. We have 199 bench marks in the common network and 386 bench marks in the maximum network.

Levelling	Performed in years	Mean epoch	Precision mgpu/$\sqrt{km}$	Number of loops in common net	Number of loops in maximum net
First	1892–1910	1901.0	1.254	9	11
Second	1935–1955	1945.4	0.460	9	18
Third	1978–	1984.8	0.854	9	24

REFERENCES

Blomqvist E, Renqvist H (1910): Das Präcisionsnivellement Finlands 1892–1910. Fennia 31:2, 1–265.

Kääriäinen E. (1966): The Second Levelling of Finland in 1935–1955. Publ. Finn. Geod. Inst. 61.

Mäkinen J. and V. Saaranen (1998): Determination of postglacial land uplift from the three precise levellings in Finland. J. Geodesy 72: 516–529.

Mäkinen J. and V. Saaranen (1999): Computation of land uplift from the three precise levellings in Finland. Proceedings of the 13th General Meeting of the Nordic Geodetic Commission, Gävle, May 25–29, 1998. National Land Survey of Sweden, in press.

Takalo, M. and J. Mäkinen (1983): The Second Levelling of Finland for Lapland. Publ. Finn. Geod. Inst. 99.

Vermeer M., J. Kakkuri, P. Mälkki, H. Boman, K. K. Kahma and M. Leppäranta (1988): Land uplift and sea level variability spectrum using fully measured monthly means of tide gauge readings. Finnish Marine Research 256: 3–75.

Improving a horizontal datum without changing the coordinates

B-G Reit

The National Land Survey of Sweden

E-mail: bo-gunnar.reit@lm.se

ABSTRACT

The 7-parameter similarity transformation between a global and a local reference system presupposes knowledge of the separation between the geoid and the local ellipsoid. The local geoidal heights might be impaired by errors of several metres. To prevent these errors from having a bad influence on the horizontal fit, the transformation parameters should be determined with appropriate height constraints. Parameters determined in this way can be used to compute improved geoidal heights above the local ellipsoid, which in turn will put the local geodetic coordinates reduced to the ground in better agreement with the actual point positions at the earth's surface.

INTRODUCTION

Little attention has been paid to the proper weighting of coordinates when computing the seven parameters of the similarity transformation between two 3D-reference systems. One common situation is transformation between a global reference frame like WGS 84 or one of the ITRF systems and some national or local horizontal datum.

Generally, old national/local geodetic datums were determined by a conventional terrestrial triangulation, measuring distances and angles, the local datum point being fixed on basis of astronomical observations. The measurements were reduced to the ellipsoid, at best taking into account the separation between the geoid and the ellipsoid of the local datum. Often the network has evolved over a time span of several decades. For these and other reasons the geometrical quality of the system might be impaired by considerable distortions, some distortions being quite local, others having a more systematic character. An example of the latter is a bias in the scale.

Today geodetic reference frames of high accuracy can be established by using GPS techniques. An internal accuracy (1σ) of better than 1 cm in the horizontal component and 1-3 cm in the vertical component is quite feasible.

Co-location of points of the GPS network and the local network makes it possible to investigate possible distortions of the local network. However, changing the coordinates of a network is not a very popular action, as it causes a lot of work and additional costs to the users. One way of bringing the coordinates of the local and the global systems in better harmony is to tamper with the definition of the local datum. For instance, a scale bias can be taken away without changing the horizontal coordinates just by moving the local ellipsoid a suitable amount in the vertical direction relative to the geoid, 6 metres corresponding to a change in the scale of approximately 1 ppm. The implication of such an operation is the concern of a few geodesists, while the great majority of the users will probably not be affected by the change. A further improvement may be achieved by giving the ellipsoid a suitable tilt.

THE PROBLEM

Given a set of points with coordinates known in a local and a global system, find the location of the local ellipsoid relative to the geoid (or, equivalently, relative to the ground) which brings the local horizontal coordinates in "best" agreement with the relative positions of the ground stations.

The Solution

It is assumed that the coordinates of the global system have an accuracy superior to the local ones.

The local horizontal datum is 2-dimensional. However, if it had been possible to compute this datum in a completely consistent way, that is if the separation between the ground and the surface of the ellipsoid had been known to a high degree of accuracy, the 3-dimensional reference frame obtained by combining these heights with the horizontal coordinates would have resulted in a just as well-founded model of the earth's surface as does the geocentric datum obtained by GPS. In this case a 3D similarity (Helmert) transformation could be used for transformation between the two datums according to the well-known formula (the reader is supposed to be familiar with the notation used below)

$$\mathbf{X}_L = \Delta\mathbf{X} + (1+\delta)\mathbf{R}\mathbf{X}_G \tag{1}$$

where

$$\mathbf{X}_L = \begin{bmatrix} X_L \\ Y_L \\ Z_L \end{bmatrix} = \begin{bmatrix} (\nu_L + H_L + N_L)\cos\varphi_L \cos\lambda_L \\ (\nu_L + H_L + N_L)\cos\varphi_L \sin\lambda_L \\ (\nu_L(1-e_L^2) + H_L + N_L)\sin\varphi_L \end{bmatrix}$$

$$\nu_L = a_L / \sqrt{1 - e_L^2 \sin^2\varphi_L}$$

$$H_L = orthometric\ height$$

$$N_L = geoidal\ height$$

$$\mathbf{X}_G = \begin{bmatrix} X_G \\ Y_G \\ Z_G \end{bmatrix}$$

In practice, the accuracy of the local vertical component is low. Not taking this fact into account might be hazardous when using eq. (1) to compute the parameters of the similarity transformation. Therefore the approach is to estimate the parameters of the 3D similarity transformation without letting the vertical positions of the local datum influence the 7-parameter fitting. This can be achieved in two alternative ways.

1. Let the seven transformation parameters as well as the geoidal heights (N_L) above the local ellipsoid be unknowns of the least squares fitting.

2. Assign appropriate weights to the coordinates of all co-located points used in the least squares fitting.

A serious drawback of alternative 1 is that in case of a low number of co-located points or if these points are covering a small area the solution will be sensitive to local distortions in the horizontal coordinates. Alternative 1 drops out as a special case of alternative 2 when the vertical components are assigned weights close to zero. The weighting approach is also favourable in another way as it makes it possible to take into consideration the fact that triangulation points often have heights of poor accuracy while benchmarks might have horizontal coordinates from digitising with an accuracy not better than 5-10 meters. For these reasons the second alternative is to be preferred.

Alternative 2 necessitates a rewriting of equation (1) in a way that makes it possible to assign proper weights to the coordinates of the local datum. How this can be done is described in **[1]**.

Keeping the scale as unknown and at the same time giving the geoidal heights weights close to zero or treating them as unknowns will result in a singular or ill-conditioned equation system. Putting the scale correction equal to zero is preferable as the local 3D coordinates then will produce correctly scaled chord distances. The drawback is that the ellipsoid might move several metres in the vertical direction. If such changes in the local geoidal heights are unacceptable the scale must be determined by some other means and that value put into the equations.

A NUMERICAL EXAMPLE

The Swedish national grid system, RT 90, is lacking a rigorous datum definition. The system is based on the Bessel 1841 ellipsoid although the adjustment of the triangulation network was originally performed on the Hayford ellipsoid using the same geoid as was used in the computation of European Datum 1987. In a second step the coordinates were transformed to the old map grid in use on the Bessel ellipsoid using a 2D Helmert transformation. The resulting coordinates were designated RT 90. The applied procedure later gave rise to an erroneous definition of the geoid model to be used in conjunction with the RT 90 system.

During 1992-1993 a network of 20 permanent stations for GPS (SWEPOS) was established. In August 1993 a GPS-campaign was carried out resulting in a 3D geocentric reference frame, SWEREF 93, with an internal accuracy at the centimeter level, cf. **[2]**.

The parameters of a 3D similarity transformation have been computed by a conventional 7-parameter fitting, using as the vertical component of the local system the sum of the heights above the geoid and the geoidal heights from the erroneous model mentioned above. As can be seen from fig. 1, the residuals indicate a systematic banana shaped horizontal distortion, with a maximum of 0.35 meters, cf. table 1.

The same stations have been used to make an optimal horizontal fitting according to alternative 2 of the previous section. In this computation the weights used were based on an a priori standard deviation of 0.04 meters for each of the two horizontal components and 15 meters for the vertical component. The scale difference between the systems was kept to zero. For the vertical component of the local coordinates height above the geoid was used instead of height above the ellipsoid.

Table 1: Conventional 7-parameter fitting

Station	North[m]	East[m]	Up[m]
ARJE	-.090	-.066	-.142
BURE	.033	.069	.061
SKEL	.043	.046	.031
HASS	.073	-.182	-.048
JONK	-.047	-.048	-.036
KARL	.045	.028	.256
KIRU	.004	-.353	-.132
KLIN	-.038	-.195	.000
LEKS	.019	.130	.176
LOVO	-.019	.110	-.034
MART	-.020	.102	.027
NORR	-.092	.003	-.104
ONSA	.042	.015	-.083
OSKA	-.072	-.118	-.065
OSTE	-.061	.095	-.078
OVER	.182	-.060	-.001
SUND	-.002	.125	.128
SVEG	-.004	.112	.017
UMEA	.015	.115	.201
VANE	.063	.016	-.101
VILH	-.057	.063	-.074
r.m.s.	.063	.124	.108
max	.182	-.353	.256

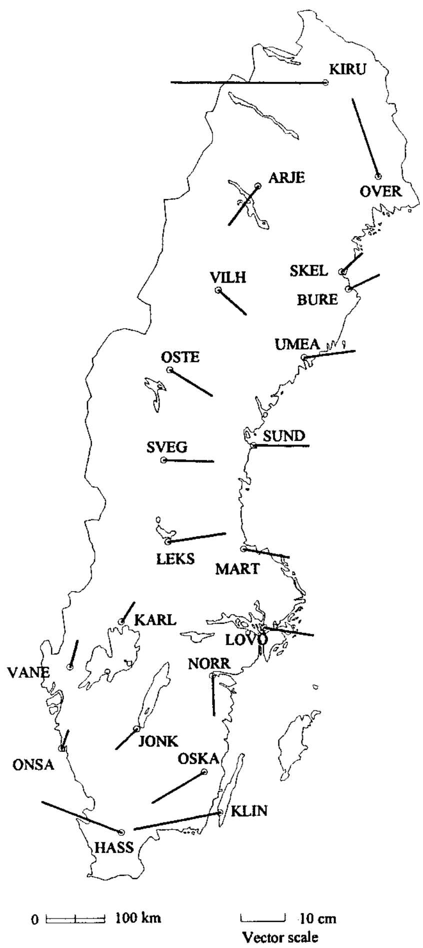


Fig. 1. Conventional fit, horizontal residuals

As can be seen from fig. 2 and table 2 a considerable reduction of the horizontal residuals have taken place compared to the conventional 7-parameter approach.

Using heights above the geoid instead of heights above the local ellipsoid, means that the vertical component of the residual vector represents the height of the geoid above the local ellipsoid. Keeping the scale difference to zero, means that the backward reduction of the horizontal distances from the surface of the local ellipsoid to the ground will be in good agreement with the GPS baselines. This fact can be used for investigating possible scale biases of the electro-optical and the microwave instruments used for the distance measurements, provided that the GPS baselines do not have a bias in the scale.

A redefinition of the local datum according to the fitting principles outlined above may be undertaken in the following way. Let the horizontal geodetic coordinates of the old and the new datum be coincident and let the geoidal heights of the redefined datum be given by the vertical residuals from the fit. The new datum defined in this way will be relieved of any deformation that can be modelled by the 7-parameter transformation. That is scale error and tilting of the ellipsoid. The implication of this is that subsequently measured EDM distances using instruments being consistent with the meter definition have to be reduced to the ellipsoid using the geoidal heights of the redefined datum. Doing so, these measurements will be in better agreement with horizontal geodetic coordinates than when reduced according to the old datum definition.

Table 2: Optimal 7-parameter fitting

Station	North[m]	East[m]	Up[m]
ARJE	-.037	.021	-0.351
BURE	-.022	.032	-9.315
SKEL	-.010	.030	-8.426
HASS	.038	.018	-1.038
JONK	-.071	-.008	-4.586
KARL	.013	-.043	-6.965
KIRU	-.045	-.083	2.142
KLIN	.070	-.057	-4.377
LEKS	.031	.015	-7.138
LOVO	.053	.031	-12.037
MART	.018	-.007	-11.270
NORR	-.045	-.024	-8.492
ONSA	-.083	.080	-2.040
OSKA	-.002	-.034	-5.531
OSTE	.000	-.017	-4.678
OVER	.050	.035	-4.893
SUND	.024	.011	-10.666
SVEG	.025	-.019	-4.362
UMEA	.012	.032	-10.157
VANE	-.030	-.025	-4.097
VILH	.008	.021	-4.330
r.m.s.	.040	.037	6.898
max	-.083	-.083	-12.037

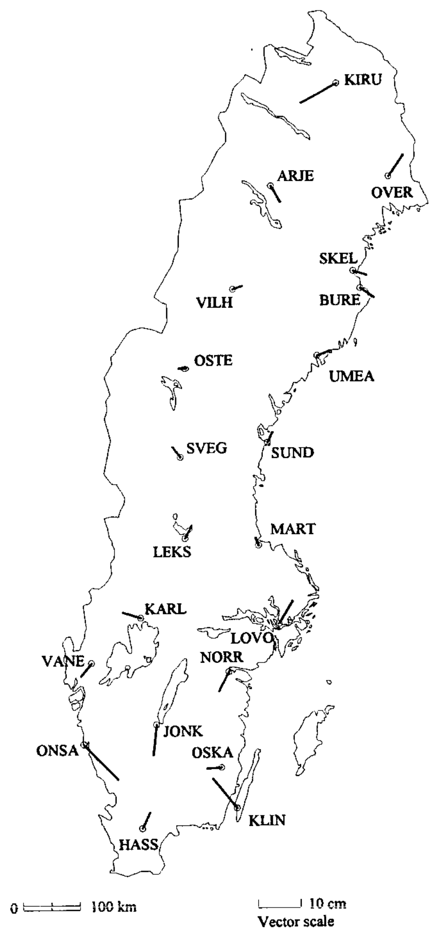


Fig. 2. Optimal fit, horizontal residuals

DISCUSSION

Taking a long view, positioning by satellites will make all local geodetic datums obsolete. However, the change to truly geocentric reference frames will take place gradually. Meanwhile there will be a need to convert data back and forth between global and local datums. A mapping function must then be established between the horizontal part of the global system and the local horizontal datum.

With no distortions present a 3D-similarity transformation would produce a mapping free of error. In reality accidental errors and systematic distortions will give rise to differences between the transformed and the original coordinates. If the aim of the mapping is the best possible modeling of the distortions there certainly exist better mapping functions than the similarity transformation. However, at the initial stage of the analysis the similarity transformation is an excellent tool for studying the distortions, provided that the residuals are expressed as local north, east and up components. Another advantage of the similarity transformation over more exotic mapping functions is that it is implemented in a great deal of software packages. A reasonable approach to the mapping task would be to offer the users a two step transformation over more exotic procedure, one step being the similarity transformation. For the average user this step will probably produce satisfying results. For the advanced user having high accuracy demands, a second interpolation step can be offered.

When computing the seven parameters of the similarity transformation it is important to only use points with known accuracy, that is to be sure that no gross errors are present in the coordinates. The number of control points used in the fitting must be large enough to ensure that local distortions stay local and do not have a bad influence on the estimated transformation parameters. These precautions are of vital importance when dealing with cases like the one mentioned in the numerical example where the 7-parameter fitting was used for a redefinition of the local datum. For a local system covering a small area, say 100 by 100 km, like the grid system of a city and environs, the method described in [3] might be a useful alternative to the 7-parameter transformation.

Note that the approach outlined in this article is quite different from the solution suggested in [4].

CONCLUSION

The Swedish experiences show that it might be worthwhile to include the modeling of the heights of the geoid above the local ellipsoid as a part of the fitting process in order to obtain an optimal horizontal fitting between the global and the local geodetic datum.

ACKNOWLEDGEMENTS

The author wishes to thank Dr. M. Ekman at the National Land Survey for valuable comments.

REFERENCES

1. Reit, B.-G., 1998. The 7-parameter transformation to a horizontal geodetic datum. *Survey Review*, 34(268): 400-404.

2. Reit, B.-G., 1995. SWEREF 93 - a Swedish reference system for GPS. *Reports of the Finnish Geodetic Institute*, No. 95:4 (Editor M. Vermeer): 139-144.

3. Reit, B.-G., 1997. A simple way of introducing a global reference frame in surveying and mapping. *Survey Review*, 34(264): 87-90.

4. Vanicek, P. and Steeves, R. R., 1996. Transformation of coordinates between two horizontal geodetic datums. *Journal of Geodesy*, Vol. 70: 740-745.

Status of the European height systems UELN and EUVN

Johannes Ihde

Bundesamt für Kartographie und Geodäsie, Außenstelle Leipzig

Karl-Rothe-Straße 10-14, 04105 Leipzig, Germany, e-mail: ihde@leipzig.ifag.de

After a break of ten years, the work on the United European Levelling Network (UELN) resumed in 1994 under the name UELN-95. The objectives of the UELN-95 project being to establish a unified vertical datum for Europe at the one decimeter level with the simultaneous enlargement of UELN as far as possible to include Eastern European countries. More than 3000 nodal points were adjusted, constraint-free, in geopotential numbers linked to the reference point of UELN-73 (gauge Amsterdam). The new heights in the system UELN-95/98 are available for more than 20 participating countries.

The European Vertical GPS Reference Network (EUVN) is designed to contribute to the UELN project along with the connection of European tide gauge benchmarks as contribution to monitoring absolute sea level variations, the establishment of fiducial points for the European geoid determination, and the stepwise development of a European kinematic height reference system. The EUVN includes 195 points all over Europe. At every EUVN point, three-dimensional coordinates in ETRS89 and levelling heights primary in the system of the UELN-95 have to be derived. The GPS computations are finalised, though some levelling connections still have to be realized. At the tide gauge stations of EUVN additional sea level observations have to be included.

Heights and Vertical Control in the Czech Republic - Evolution and Present State in Context of Current European Projects

Jaroslav Šimek

Research Institute of Geodesy, Topography and Cartography

CZ-250 66 Zdiby 98

Abstract

After a brief review of the evolution of the vertical control in the Czech Republic from 1918 to the beginning of nineties, an outline is given of the present Czech activities towards incorporation of the national vertical control into the European structures currently established within the international projects like UELN and EUVN. An attempt to compile a map of annual velocities of vertical movements of the earth surface for a part of Central Europe is also mentioned. Finally, a feasibility of GPS heighting is discussed in the context of modelling high resolution quasigeoid for the Czech Republic and some recent results are presented.

Use of GPS in unification of vertical datums and detection of levelling network errors

Markku Poutanen

Finnish Geodetic Institute

Geodeetinrinne 2, FIN-02430 Masala, Finland

Markku.Poutanen@fgi.fi

ABSTRACT

We discuss the interconnection of vertical networks using GPS and gravimetric geoid models, and give some examples how this can be done in practice. We also discuss several error sources of GPS-based methods which degrade the accuracy of height determination. GPS can be used for interpolation of heights, and some medium/low precision spirit levelling may be replaced by GPS. It can also be utilised in maintaining country-wide levelling networks or connecting national levellings. However, many ordinary spirit levelling errors cannot be controlled by GPS and it will not replace precise levelling over relatively short distances.

INTRODUCTION

During recent years, use of GPS in height determination has increased rapidly. This brings forward the question whether slow and expensive levelling can be replaced by GPS, or, at least, the levelling errors can be controlled with it. There are two different things which we have to take into account: the accuracy of the GPS itself and the accuracy of the geoid model we need to transform heights above the ellipsoid to orthometric or normal heights. In the following we discuss some aspects of the GPS in height determination.

The observing geometry and the way the errors affect GPS measurements are most unfavourable to the vertical component. The unmodelled (and unknown) troposphere errors are one of the most important accuracy limiting factors today. Even in permanent GPS networks, the residual errors can amount to up to 0.01 ppm scale error (*M. Ollikainen*, private communication, 1999) and the height component can vary in the cm-range. We will discuss this later in this paper.

The land uplift, caused by the post-glacial rebound, is a typical phenomenon in the Fennoscandian area. The maximum uplift value, about 10 mm/yr., is at the end of the Gulf of Bothnia. We must correct the height values for the uplift from the standard epoch of the height datum to the epoch of the GPS observations. For this, we can use e.g. the uplift values given in the map of Kakkuri (1991).

The time-independent part of the luni-solar tide should be treated in a consistent way in spirit levelling, geoid model and GPS computations if orthometric (or normal) heights are to be obtained from GPS observations. We may distinguish three different cases of tide corrections: *Non-tidal geoid* and *non-tidal crust* when the tide is fully removed, *mean geoid* and *mean crust* when the tide is retained, and *zero geoid* when the attraction of the Sun and the Moon is removed but the permanent tidal deformation is retained. For the crust, there is no analogy to the zero geoid; the ”zero crust” would be the same as the mean crust. For an extensive discussion on the permanent tide in GPS observations, see Poutanen *et al.* (1996).

The current GPS programs give coordinates reduced to the non-tidal crust. The levelling, however, refers to various geoids, depending on the country. In Table I we reproduce the summary table of Ekman (1995). The differences between various geoids/crusts amount to up to 10 cm in the area of Fennoscandia but we can easily convert all quantities e.g. to their mean values using formulae of Ekman (1989):

$$\Delta H_m - \Delta H_n = 0.296\,\gamma(\sin^2\varphi_N - \sin^2\varphi_S) \quad [m] \tag{1a}$$

$$N_m - N_n = (1 + k)(0.099 - 0.296\sin^2\varphi) \quad [m] \tag{1b}$$

$$\Delta h_m - \Delta h_n = -0.296\,\boldsymbol{h}\,(\sin^2\varphi_N - \sin^2\varphi_S) \quad [m]. \tag{1c}$$

The first formula is used to convert height differences of the non-tidal crust above the non-tidal geoid to height differences of the mean crust above the mean geoid and is appropriate for treating levelling. The second formula converts the non-tidal geoid heights above the ellipsoid to the mean geoid heights and the third formula converts height differences of the non-tidal crust above the ellipsoid to the height differences of the mean crust, and is used for GPS ellipsoidal heights. Above, γ, k and $\boldsymbol{h}$ are the Love numbers, and φ_N and φ_S refers to the latitude of northernmost and southernmost station, respectively.

In each case, the *Love numbers used in the original non-tidal calculations must be applied*, regardless how close to or far from reality they are. E.g. in the Swedish levelling $\gamma = 0.8$ has been used, for the OSU89B spherical harmonic model $k = 0.3$, and $\boldsymbol{h} = 0.609$ in the Bernese GPS software; already these three are incompatible among themselves.

To obtain orthometric or normal height from GPS observations one needs a geoid model. There are several good models available in the Fennoscandian area, like the Nordic Standard Geoid NKG-96 (R. Forsberg, private communication, 1997), BSL95A and FIN95 (Vermeer, 1995). Additionally, there are also several global geoid models, like OSU91A and EGM96 but these are less accurate locally. The NKG-96 is a cm-geoid without any major distortions due to a considerably better gravity coverage of the surrounding area than its predecessor NKG-89. Its level was adjusted using results of several GPS campaigns.

Tidal correction of these geoid models is sometimes a bit complicated question because it depends on the underlying global potential spherical harmonic expansion. However, most of these seem to be of a non-tidal type.

Country Height system	geoid	geoid type	epoch
Finland N60, orthometric	classical	mean	1960
Sweden RH70, normal	quasi	non-tidal	1970
Norway	classical	mean	none

NN 1954, orthometric			
Denmark DNN GI, orthometric	classical	non- tidal	1950
W.Europé UELN 73, orthometric	classical	mixture	(1960)
E.Europé Kronstadt, normal	quasi	?	?

Table I. *European height systems (Ekman 1995).*

All the above-mentioned geoids are quasi-geoids and thus heights referring to these are normal heights. At seashore both the orthometric height H_{ort} and the normal height H_{norm} are equal because geoid and quasi-geoid coincide. However, the difference increases with terrain height by (Heiskanen and Moritz, 1967)

$$H_{norm} - H_{ort} = \frac{C}{\bar{\gamma}} - \frac{C}{\bar{g}} \approx \frac{\Delta g_B}{98\,2000[\text{mgal}]} H_{ort} \tag{2}$$

where C is the geopotential number, $\bar{g}$ is the mean gravity, $\bar{\gamma}$ is the mean normal gravity, and Δg_B is the Bouguer anomaly. One can compute exact difference if geopotential numbers are available, otherwise the difference can be estimated with the Bouguer anomaly.

Global geoid models are not fitted to a particular height system. Therefore, one needs a local adjustment (height and tilt) of a geoid to use it in height determination with GPS. Geoid models may have long periodic (several hundreds of km) errors which are not removed in such simple adjustment. There are some pre-fitted models, like FIN95 which is adjusted to the Finnish N60 system. However, also these are time-dependent. E.g. FIN95 is approximately in epoch 1993 because it is the mean epoch of the GPS observations used in adjustment. In precise work, one has to either reduce the observed ellipsoidal heights to the epoch of the geoid model by using the known uplift values or make a new local adjustment of the geoid.

UNIFICATION OF VERTICAL DATUMS

Small local levelling networks, like those of cities or municipalities can be connected to national levelling networks in a straightforward way. This is not true anymore in connecting national networks. It is an oversimplification to compare height systems of two countries just by giving one number for the height difference. We have to more or less arbitrarily choose what values are intercompared. As an example, when speaking about Swedish and Finnish heights, we first convert Finnish orthometric heights to normal heights which are consistent with the gravimetric (quasi-)geoids available. At the seashore both heights are equal and we need not to distinguish there between them.

Next we convert Swedish heights to refer to the mean geoid instead of the non-tidal geoid and finally, the land uplift correction brings the heights to the same epoch (from year 1960 in Finland, 1970 in Sweden). After this procedure there is not so much left of the original systems and one may ask, with a good reason, if there is any sense to make this.

A more reasonable way to do the connection is to establish a new, well defined height system for the area and compute transformation parameters from the old national systems to the new one. In this we follow the guidelines shown in Ekman and Mäkinen (1991 and 1995; hereafter E&M) where they introduce the *Nordic Height System 1960* (NH60). However, for practical reasons, especially from the viewpoint of GPS, we refine their proposal slightly. For brevity, we call it here NEH2000 (*The North European Height System 2000*):

1. The zero point is the *NAP* (Normaal Amsterdams Peil).
2. The heights are *normal heights.*
3. The permanent tide (time average of the tidal deformation) is retained so the crust refers to the *mean crust*. The geoid is the *mean geoid.*
4. The normal heights are reduced from the national system epoch to the epoch 2000.0 using the *land uplift relative to the geoid* (i.e. the sum of the apparent uplift and the eustatic rise of the sea level). The absolute uplift which can be obtained with GPS, can be converted to this "levelled" uplift by subtracting the rise of the geoid.

In the following we will shortly discuss the advantages and disadvantages of these choices.

1. The selection of the NAP as the zero level agrees with the definition in E&M and is consistent with the UELN 73 zero point. In the European-wide EUVN campaign (Ihde *et al.*, 1998) it is also a natural choice and may also give a good connection to the proposed World Vertical Datum (Rapp, 1995). There are some theoretical arguments to use the potential W_0 to define the zero level (e.g. Grafarend and Ardalan, 1997) but in practice there is always a strong tendency to retain the old definitions.
2. In their paper, E&M used geopotential numbers. Their aim was mainly to study the sea surface topography and with the small heights there is no need to distinguish between the normal and orthometric height. While geopotential numbers are required for the NEH2000, the question of their conversion to metric units remains. The gravimetric geoids we use nowadays are often quasi geoids, i.e. basically height anomaly maps, and one should use normal heights with them. The difference between the normal height and the orthometric height is less than 10 cm in most places in the Fennoscandian and Baltic states area because the heights are so small. Especially, with GPS, when the user wants "heights above the sea level", one uses an available geoid model and then obtains normal heights directly. The common intuition about "the height above the sea level" is more relevant with the orthometric height, and the normal height concept conflicts this intuition. We admit that physically orthometric height is the natural height system and there is no principled reason not to use it *if* a good geoid becomes available. There are strong opinions in favour and against of both systems and the topic requires a lot more discussion, especially among the groups who compute global or regional geoids. In this paper we look at the topic in the viewpoint of current practice and data availability.
3. There is an extensive discussion on GPS and tide in Poutanen *et al.* (1996) where the use of mean geoid and mean crust is proposed. In some cases the zero geoid could be theoretically better but the mean geoid has the advantage that it describes the temporal mean of the actual, instantaneous equipotential surface corresponding to the mean sea level. The definition thus implies that GPS observations should be reduced from the non-tidal crust to the mean crust. One should abandon the physically irrelevant non-tidal concept in geodetic measurements. One should also note that the reference surface of the levelling is defined in a different way in different countries as shown in Table I, and the heights H should be converted to refer to the mean geoid.

4. The heights in the Fennoscandian and the Baltic area are changing due to the postglacial rebound of the crust. In order to maintain the correct connection to the global networks, the absolute land uplift should be applied because the height above the ellipsoid, h, is changed by the absolute uplift. However, with normal heights one has to use the levelled uplift value, i.e. the geoid rise is to be subtracted. The magnitude of the geoid rise, however, is relatively small and can be neglected in all but the most accurate nation-wide measurements. If one uses GPS to obtain the absolute uplift value, the geoid rise can be been taken e.g. from Ekman (1993). The magnitude of the rise is 5 – 10 % of the uplift value. The epoch 2000 is quite close to the ending of precise levellings in the Nordic countries. It is now proper time to discuss updating the national height systems.

REALISATION OF A COMMON VERTICAL DATUM

A more dense network is needed to firmly connect a vertical datum, like NEH2000 to the national height systems like N60 or RH70. We show only some guidelines here, mostly based on the discussion above and references therein. The selection of the geoid has the key role here. We have a good situation in Fennoscandian area because even two excellent geoids are available, NKG96 and BSL95A. Both are fixed to a common datum, using GPS data as described above. In the following, we choose the NKG96 for the basis of the NEH2000 realisation because it is slightly better in the Southern Baltic than the BSL95A (Martin Vermeer, private communication, 1997).

The GPS data set we use here contains a total of 95 points in Finland, Sweden and Estonia. The network can be seen in Fig. 1 where we show the post-fit residuals. According to the guidelines above, we made the following corrections to the data values:

H: The heights were reduced from the national datum epoch (FIN 1960, S 1970) to epoch 2000 for land uplift using the uplift values from Kakkuri (1991) and for the eustatic rise of the sea level, value 1mm/yr.

h: The values obtained from GPS were first corrected from non-tidal crust to mean crust, using (1c). After that the uplift of about 5 years (from the mean epoch of all measurements to 2000) was added. Additionally, geoidal height N was converted from non-tidal to mean height. After this, we computed

$$r = h_{GPS}^{2000} - N_{mean} - H_{lev}^{2000} \tag{3}$$

and fitted a polynomial surface

$$S = \sum_{i=0}^{m} \sum_{j=0}^{n} a_{ij} (\lambda - \lambda_0)^i (\varphi - \varphi_0)^j \tag{4}$$

separately for each country. The post-fit residuals are shown in Fig. 1, and the coefficients of the fit in Table II. In the adjustments we used the value $\lambda_0 = 60°$ and $\varphi_0 = 20°$ for all three countries.

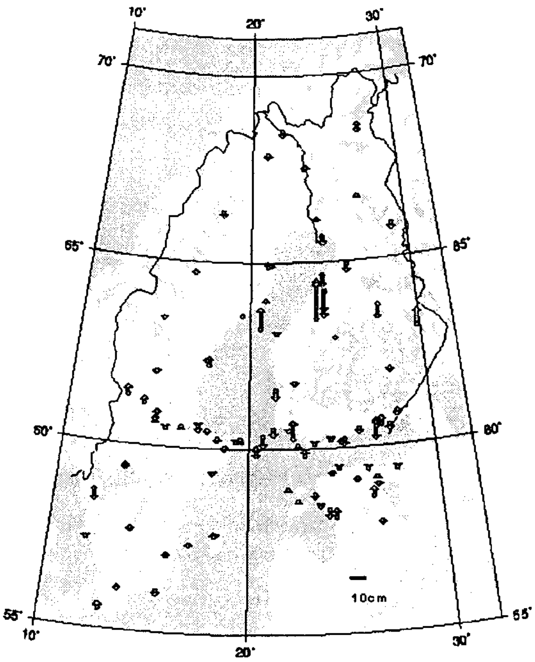


Figure 1 *Post-fit residuals. A second degree polynomial surface was fitted separately to the height difference $H_{GPS} - H_{lev}$ in each country and the arrows shows the residuals of this fit. Except some outliers, all residuals are well below 10 cm.*

Finland

$S = .17012968E{+}00 \pm .34890958E{-}01$

$+ X^{*} .47965032E{-}01 \pm .18969820E{-}01$

$+ Y^{*} .37165633E{-}01 \pm .17007044E{-}01$

$+ X^{2}{*} -.77663047E{-}02 \pm .22111091E{-}02$

$+ X^{*}Y^{*}\ .56553564E{-}02 \pm .23894568E{-}02$

$+ Y^{2}{*} -.53395461E{-}02 \pm .20179607E{-}02$

Sweden

$S = .13171358E{+}00 \pm .22111548E{-}01$

$+ X^{*} .39942825E{-}01 \pm .79216262E{-}02$

$+ Y^{*}\ -.47656854E{-}01 \pm .13020401E{-}01$

$+ X^{2}{*}\ -.60229776E{-}02 \pm .94603853E{-}03$

$+ X^{*}Y^{*} .75005216E{-}02 \pm .19034318E{-}02$

$+ Y^{2}{*}\ -.48163915E{-}02 \pm .16280249E{-}02$

Estonia

$S = .39838716E{-}01 \pm .19115648E{+}00$

$+ X^{*} -.37574914E{+}00 \pm .17291180E{+}00$

$+ Y^{*}\ -.41951423E{-}02 \pm .50219724E{-}01$

$+ X^{2}{*} .61359686E{-}01 \pm .66604315E{-}01$

$+ X^{*}Y^{*}\ .32666889E{-}01 \pm .18979704E{-}01$

$+ Y^{2}{*}\ .37674007E{-}02 \pm .41947902E{-}02$

Table II *Coefficients of the fit of Eq. (4). Here we have marked $X = \varphi - \varphi_0$, $Y = \lambda - \lambda_0$. $\varphi_0 = 60°$, $\lambda_0 = 20°$ in all cases.*

We did not use the geoid rise because of its smallness; also the land uplift in Estonia was neglected because the datum epoch is not well defined and because the country is relatively small in area. Also no correction was applied to convert Swedish normal heights to refer to the mean geoid. Finnish heights were not converted to normal heights. These corrections are either small or will be absorbed into the fit. The idea was to minimise the steps for practical purposes, still not unnecessarily lose accuracy.

In conversion of national heights to a unified datum (NEH2000, EUVN, ...), one should first make the uplift correction to get the heights in the epoch of the datum. After this, using (4) and coefficients from Table II, a datum shift can be done. These steps can be done automatically if one uses a land uplift model, either a grid from which a value can be interpolated, or a (high order) polynomial surface for less accurate applications.

ERRORS AND ERROR CONTROL

Formal errors of a GPS solution do not agree with the actual errors of the measurement but are normally far too optimistic. In the following we discuss some of the errors in GPS height determination and how they affect the accuracy of height determination with GPS.

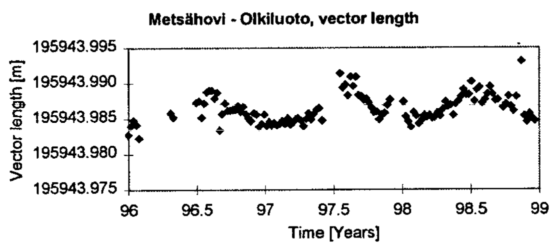
Troposphere related errors are more difficult to eliminate than the ionosphere errors because troposphere affects in the same way both L1 and L2 frequencies. Measurement of the water vapour content via the signal path is not possible in routine measurement. Use of a standard troposphere model may result in an error which affects both scale and height. The scale error $\Delta l / l$ is

$$\frac{\Delta l}{l} = \frac{\Delta \rho}{R_{\oplus} \cos z_{\max}} \tag{5}$$

where $\Delta\rho$ is the troposphere model error, $R_{\oplus}$ is the Earth's radius and z_{max} is the maximum observed zenith angle. The height error Δh amounts to

$$\Delta h = \delta\Delta\rho_{ab} / \cos z_{max} \tag{6}$$

where $\delta\Delta\rho_{ab}$ is the relative troposphere error between points a and b. In Fig. 2. we show a scale error obtained in the Finnish permanent GPS network FinnRef (Ollikainen and Koivula, private communication 1999) which most likely comes from the troposphere model error. According to Eq. (5), only a ±2 cm error in troposphere refraction estimation is sufficient to produce the detected scale variation of about ±0.01 ppm. We have here no methods to see directly the total height error of Eq. (6), but there are good reasons to assume that it exists.



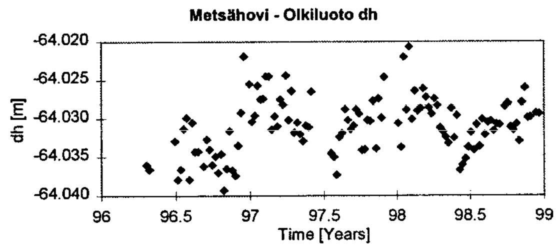


Fig. 2. *Change of vector length and height difference between the Finnish permanent GPS network stations Metsähovi – Olkiluoto. The change in vector length is visible in other vectors, too, and one can estimate the scale error to be of the order of 0.01 ppm. The change in height difference is somewhat bigger but we do not know if there is any constant bias. Both effects are most likely coming from minor errors in troposphere delay model and they have a clear yearly cycle. (M. Ollikainen and H. Koivula, 1999, private communication)*

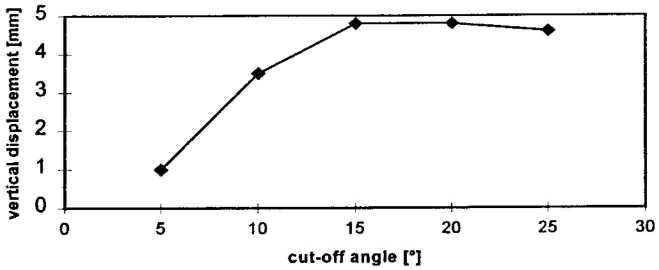


Fig. 3. *Effect of cut-off angle in computed height. The vector length was 36 m. Both antennae were Dorne Margolin type choke ring antennae but the other antenna had a radome, the other was without. The zero of the vertical scale is arbitrary. (Kylkilahti, 1999)*

Troposphere error grows rapidly at small elevation angles. Therefore, in most applications, the cut-off limit in GPS processing is set to 15°. There is also another elevation dependent error source, namely the antenna phase centre variation. The position of the antenna electrical phase centre depends e.g. on the direction and frequency of the incoming signal. If identical antennae are used in the whole network, the phase centre error cancels out almost totally but if there are different antennae (or even different antenna mountings, radomes, snow layer on top of an antenna, multipath...), the error remains. The phase centre variation causes a systematic error in height which cannot necessarily be seen in normal processing, although it can be even centimetres. It can be eliminated with a field calibration but this implies a lot of extra work. Antenna patterns have been determined e.g. for the FinnRef stations (Kylkilahti, 1999).

Also the change of cut-off angle affects the height component; we demonstrate the effect in Fig. 3. The reason for this is that the relative phase centre variation of two antennae is a function of elevation angle. In this example we get relatively small change because antennae were identical and only radome and mounting were different. If possible, cut-off angle should be kept in data processing unaltered from campaign to campaign.

Use of precise ephemeris is also crucial in height determination. In Fig. 4. we show the error in height when broadcast ephemeris are used instead (Ollikainen 1997). The behaviour shown in Fig. 4. is applicable to this particular campaign only. Change in SA pattern may cause different distance dependency in other campaigns.

This far we have been speaking on errors only. After this we may ask, what kind of accuracy can be achieved in GPS levelling. In Fig. 5. we give an example of this. The figure is taken from thorough study of Ollikainen (1997) where he tested GPS levelling on two areas in South Finland. He used different geoid models with and without local adjustment. When compared to the spirit levelled heights, the mean accuracy of the height differences with the best geoid models was ±15 mm. This includes also errors of the spirit levelling. When spirit levelling errors were removed, the mean accuracy of the GPS levelling was ±12 mm.

This kind of accuracy can be achieved even in a moderately wide area. On the other hand, accuracy may not improve remarkably in smaller networks because errors coming e.g. from antenna phase centre variation or multipath do not depend on distance.

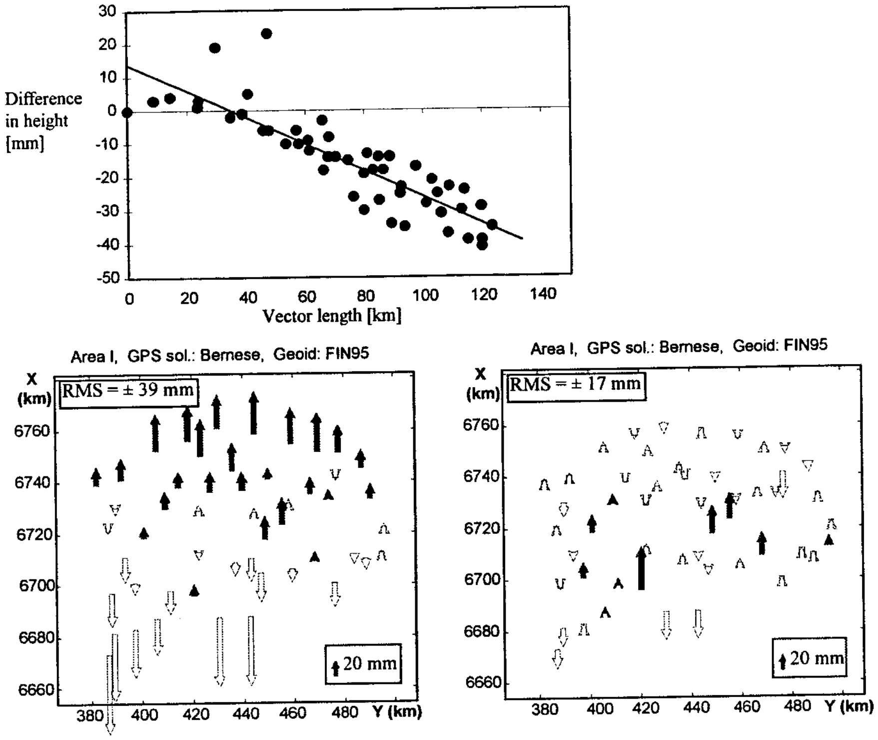
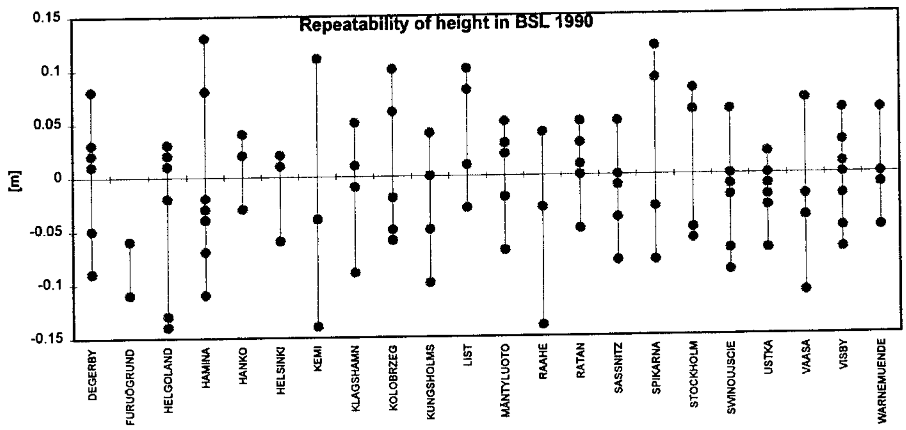
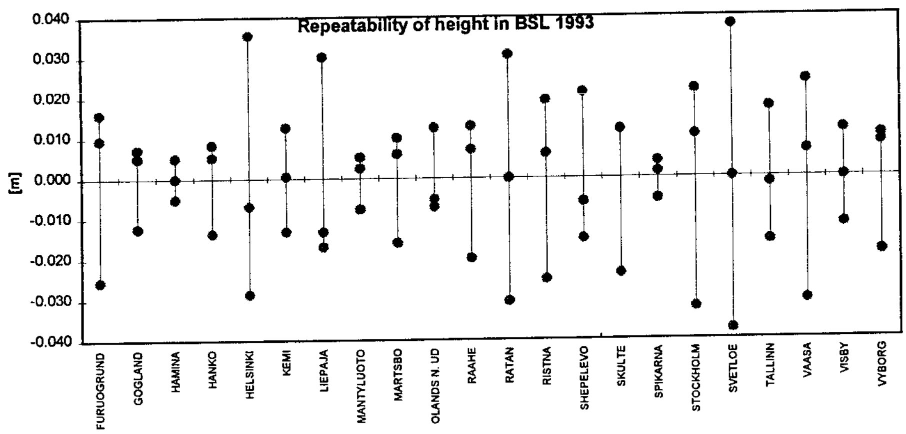


Fig. 5. *Height difference between GPS levelling and spirit levelling (left): using FIN95 geoid model; (right): after an additional plane fit (level and tilt). (Ollikainen 1997)*





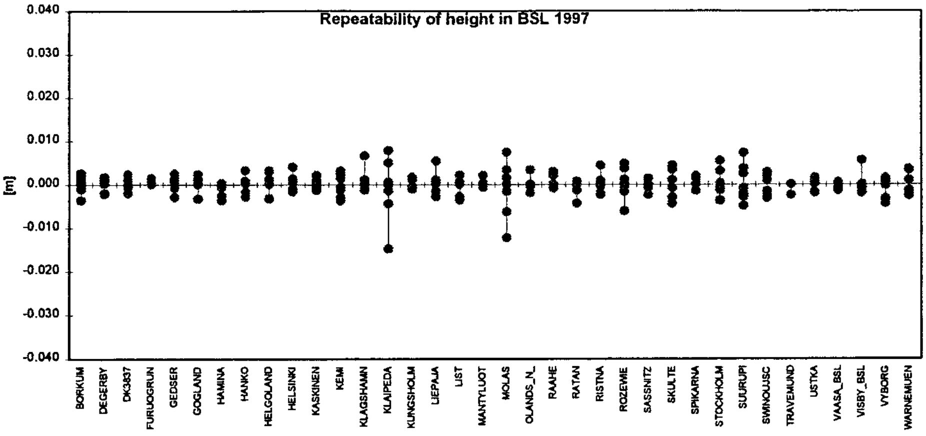


Fig. 6. *Repeatability of BSL 1990, 1993 and 1997 GPS campaigns in height component. Note the different scale in 1990 campaign plot.*

The repeatability of the 1997 campaign is about one tenth of that of the 1990 campaign, namely 1 mm in horizontal and 2.6 mm in the vertical component.

CONCLUSION

During the nineties, there has been a vast development on receiver technology, precision of satellite ephemeris and data processing. As one might expect, the increase in accuracy has been as dramatic as the change in technology. We demonstrate this in Fig. 6. where repeatability in height of three successive Baltic Sea Level GPS Campaign is shown. However, the systematic errors can be larger than might be expected from the repeatability. In (Poutanen, 1995) we show an example from the 1993 campaign, where the solutions of different computing groups deviate more from each other than the repeatability of the results of one group, indicating unmodelled systematic errors.

In the current state-of-the-art the GPS determination gives a sub-centimetre accuracy in the height above the ellipsoid. The geoid has the key role in determining orthometric or normal heights with GPS. The error of the geoid could be even an order of magnitude bigger than the error in GPS measurement. Combination of other methods, tide gauge observations and satellite altimetry, gives us a way to improve the accuracy, and a method to fix the position and level of a geoid. Without a local adjustment, long wavelength errors may still remain.

The need for a common vertical datum is quite obvious in the future when GPS observations are extended over the border of a country. There are several ongoing projects aiming to this goal, like EUVN (European Vertical GPS Reference Network, Ihde *et al.*, 1998). We know that the proposed new systems could remain in scientific use only and their utilisation in common use is a long process requiring European-wide agreements and acceptance. The final decision of the new system will be political and economical, not a scientific one. If the level and geoid model of EUVN will be selected according to the general guidelines shown above, the NEH2000 heights will be directly in the datum of EUVN.

What could one say about the future of GPS in levelling? If we consider the whole error budget, including observational and computational errors, it is obvious that over relatively short distances precise levelling cannot be replaced by GPS. It is possible to replace low-precision levelling where accuracy requirements are in the cm range (see e.g. Ollikainen 1997, and Fig. 5.).

The same holds true in detecting levelling errors. If we speak about loop closure errors of mostly a few cm, it is doubtful if GPS can bring any new information, or the amount of work required could be almost as big as that of a partial relevelling. Many levelling benchmarks are in such places that a direct GPS measurement is not possible but an auxiliary marker has to be established. Connection of the benchmark and the temporary marker requires spirit levelling. If there are tens of new points, the amount of the additional work will be considerable.

GPS could be used in detecting gross errors in levelling network. On the other hand, one should be able to eliminate gross errors already during the levelling in all but very special cases (like water crossings or spike measurements) where GPS can be used as an extra check.

The situation changes when we speak about regional or country wide levelling networks. Levelling over distances of several hundred kilometres may take years (or decades) and even with an excellent formal error of 0.5–0.8 mm $km^{-1/2}$ the uncertainty at the "other end" will be comparable to that which can be achieved with GPS in a few days. All this depends on how good geoid models will be available. On this basis, also unification of vertical datums can be done with the aid of GPS. GPS is also suitable for point densification inside a pre-existing network and invaluable in roadless areas where spirit levelling is impossible.

I can see the future of levelling as a mixture of traditional levelling and GPS. Local precise work is still done with traditional methods but the less accurate part will be replaced by GPS. National levelling networks could be maintained with a sparse permanent GPS network. The permanent stations are used as control points for local updates or densification either by traditional methods or by GPS. When the large Nordic precise levelling works end in 3–4 years from now, one should seriously discuss the role of permanent GPS networks, how the levelling networks can be maintained and how much of traditional levelling can be abandoned in the future. But certainly we will still need the levelling instruments.

ACKNOWLEDGEMENTS

The author would like to thank *Hannu Koivula, Jaakko Mäkinen, Matti Ollikainen,* and *Martin Vermeer* for valuable discussions and comments on this manuscript.

References

Ekman M (1989): Impacts of geodynamic phenomena on systems for height and gravity. *Bulletin Géodésique* **63**: 281–296.

Ekman M (1993): Postglacial rebound and sea level phenomena, with special reference to Fennoscandia and the Baltic Sea. In *Geodesy and Geophysics. Lecture notes, NKG Autumn school 1992 organized by Nordiska Kommissionen för Geodesi, Korpilampi, Finland 7–13 September, 1992* (Ed. J. Kakkuri). *Publ. Finnish Geodetic Institute* **115**, 7–70.

Ekman M (1995): What is the geoid? In *Coordinate systems, GPS and the geoid.* (Ed. M. Vermeer). *Reports of the Finnish Geodetic Institute* **95:4**, 49–51.

Ekman M and Mäkinen J (1991): The deviation of mean sea level from the mean geoid in the Baltic Sea. *Bulletin Géodésique* **65**: 83–91.

Ekman M and Mäkinen J (1995): Mean sea surface topography in a unified height system for the Baltic Sea area. In *Coordinate systems, GPS and the geoid.* (Ed. M. Vermeer). *Reports of the Finnish Geodetic Institute* **95:4**, 53–62.

Grafarend E and Ardalan A (1997): W_0: an estimate in the Finnish Height Datum N60, epoch 1993.4, from twenty-five GPS points of the Baltic Sea Level Project. *Journal of Geodesy*, **71**, 673–679.

Heiskanen WA and Moritz H (1967): *Physical Geodesy*. Freeman.

Ihde J, Adam J, Gurtner W, Harsson BG, Schlüter W and Wöppelmann G (1998): The concept of the European Vertical GPS reference network (EUVN). *Report on the results of the European vertical reference network GPS campaign 97 (EUVN'97)*. EUREF symposium, June 10–12, 1998, Bad Neuenahr-Ahrweiler, Germany. p. 5–18.

Kakkuri J (1991): *Geodetic operations in Finland 1987–1991*. The Finnish Geodetic Institute.

Kylkilahti A (1999): The antenna calibration of the Finnish permanent GPS network (FinnRef). Master of Technology thesis, Helsinki University of Technology, in preparation.

Ollikainen M (1997): Determination of orthometric heights using GPS levelling. *Publ. Finnish Geodetic Institute* **123**.

Poutanen M (1995): A combined solution of the Second Baltic Sea Level GPS Campaign. In *Final results of the Baltic Sea Level 1993 GPS Campaign* (Ed. J. Kakkuri). *Reports of the Finnish Geodetic Institute* **95:2**,115–123.

Poutanen M, Vermeer M and Mäkinen J (1996): The Permanent Tide in GPS Positioning. *Journal of Geodesy* **70**, 499–504.

Rapp RH (1995): A World Vertical Datum proposal. *Allgemeine Vermessungs-Nachrichten* **8–9/95**, 297–304.

Vermeer M (1995): Two new geoids determined at the FGI. *Reports of the Finnish Geodetic Institute*, **95:5**.

The Future of Vertical Geodetic Control

G. Steinberg
Survey of Israel

H. Papo
Technion, Israel Institute of Technology

Abstract

The phenomenal progress made by motorized leveling in the past two decades has not changed the status of vertical geodetic control (VGC) worldwide. Only a few countries have been able to establish and to maintain (re-observe at reasonably short time intervals) their national VGC networks at first order accuracies. The establishment of a national VGC network by precise leveling has been indeed a technical possibility for many years and for some countries it has been even economically affordable. We claim that today those national VGC networks are not indispensable. Due to their many drawbacks such as high cost, slow pace, obstacle-crossing impotence and monumentation problems, we believe that they are going to be replaced in the near future by a more attractive alternative. The alternative we envision is a national VGC network based on GPS measurements which forms an integral part of a 3-D spatial network. The reference surface is the WGS84 (or any future equivalent) ellipsoid and thus the system of heights is ellipsoidal and not orthometric. We believe that the switch from orthometric (OVGC) to ellipsoidal (EVGC) height systems at the national (or at the continental or even at the worldwide) level is imminent and inevitable. OVGC networks may continue to be used in the future but that only in the form of floating islands with a limited geographical extent. Viable candidates for such orthometric islands are regions and projects where the use of orthometric heights remains mandatory. There are cases where the island can be ellipsoidal in character or in other words form a densification layer beneath the national EVGC network. Typical examples of such orthometric or ellipsoidal islands are urban areas, low-shore areas, sensitive water system projects etc. Each island maintains its own independent datum based on those points through which it is attached to the national EVGC network. In Israel we are already experimenting with the above flexible scheme of vertical control. The national EGVC network is composed of two layers: a primary layer of ten permanent GPS stations at distances of 30-90 kilometers and a secondary layer of 150 extremely well monumented points (the G1 geodynamic network) at distances of 10-20 km. Four orthometric islands based on the above secondary layer are at various stages of construction:

The Mediterranean island: a narrow strip of land along the coast covering an area of 2000 km^2;

The Mount Carmel island: with 67 points over an area of 600 km^2;

The City of Eilat island: by the coast of The Red Sea with 66 points over an area of 20 km^2;

The City of Modyin island: and its environs (still in the early stages of planning).

1. Introduction

Over the years geodesy has been, and still is, very proud of its precise leveling technique that can produce elevation differences between distant points on the earth surface with an unsurpassed accuracy of fractions of a millimeter. The problem with leveling has never been its accuracy. The main problem associated with leveling is the technique itself which is notoriously time consuming, too expensive in terms of manpower and extremely cumbersome in having to cover literally step by step the distance between the end points. Motorized leveling has solved the slow-rate problem but the price remains prohibitively high, and still lengthy meandering is needed in order to cross mountains, wide rives or lakes. On top of the above problems related to the leveling technique, vertical crustal motions literally “slap us in the face” by rendering obsolete the heights of many points only a short time after their measurement and adjustment (see [3] as an example for difficulties in interpreting results of precise leveling). Vis-a-vis leveling, GPS as a geodetic tool has been advancing in the past 10 or 15 years by leaps and bounds. In terms of accuracy its progress has been impressive although in the vertical direction it is still at the mercy of chronic poor-geometry, insufficiently well resolved tropospheric delays, multipath and antenna focus variations. For the time being GPS can not compete with precise leveling as far as accuracy is concerned. But in all other respects it is a sure winner: it is fast, cheap, can easily bypass almost any obstacle. It only remains to see GPS make a significant improvement of accuracy in the “up” direction too. There is one additional problem which concerns not so much us - geodesists and surveyors - but rather the so called “consumers of heights”. Even if the ellipsoidal (GPS borne) heights were ten times more accurate, there are going to be users who would prefer the "good, old..." and familiar orthometric heights. "Sensitivity to transition" analyses of applicability of the ellipsoidal heights in various scientific and engineering enterprises have been made and published (see [1]). The bottom line has being that although the transition is not going to be easy, in the majority of cases it would be possible and even advantageous to use ellipsoidal (geometric) heights instead of ordinary orthometric (physical) heights. In this paper we are dealing not so much with the above "non-geodetic" problem but rather with the establishment of a novel and unorthodox system of vertical geodetic control. Miracles that were apparently commonplace in biblical times don’t happen any more. The inevitable transition from orthometric to ellipsoidal vertical control systems is just one such miracle which requires lots of work and patience from all those involved - geodesists as well as their "customers" the consumers of heights (see [7]). In the next section we present schematically our vision of that future national ellipsoidal vertical geodetic control that will be based on GPS measurements alone.

2. The ellipsoidal alternative

As an alternative to the existing national orthometric vertical control (NOVC) networks we propose to declare the ellipsoid (the WGS84 ellipsoid or any suitable equivalent) as the only reference surface needed to define the position of a control point in all 3 dimensions, including heights (above the ellipsoidal surface). The realization of a comprehensive and accurate ellipsoidal vertical geodetic control (EVGC) network based on GPS measurements does constitute an integral part of a 3-D spatial control network of any given country. As we all know, the above is not a vision anymore. GPS measurements backed up by the IGS permanent stations array have reached a level that makes it possible for any country to establish a nationwide GPS-borne 3-D geodetic control network. The time required for surveying and subsequent analysis is short and the cost is unbelievably low. As with the conventional horizontal and vertical control networks there is an advantage in constructing the EVGC net in a hierarchical order. Monumentation subtleties, optimal distances between points, types of GPS receivers and antennae, software sophistication, duration of measurement sessions - all have to be designed and coordinated in view of the well known principle of going “from the whole to the part”. A typical national EVGC network should consist of three to four control layers as follows:

(a) The fundamental primary (datum) layer consists of points at distances of 100-250 km. Most or even all of those points should be permanent GPS stations. The datum of the fundamental layer should be based on fixed (nominal) ellipsoidal heights of those points. If kinematic or dynamic models of analysis are adopted, datum definition may become more complicated but still it is an easily manageable problem. Accuracy of the adjusted heights should be of the order of 6-8 mm.

(b) The secondary layer consists of points 25-40 km apart, where relative accuracies in height are better than 1 cm. Location and monumentation of its points should be based on geometrical and geo-technical considerations.

The first two layers are not intended for the occasional user but are there to provide control for lower level (densification) layers.

(c) The third layer should consist of points at 6-10 km apart with relative accuracies still at the level of 1 cm. Due to the significantly shorter GPS vectors, a vertical accuracy of 1 cm can be obtained by relatively short GPS measurement sessions. Another characteristic that is particularly important for the third layer of control is easy accessibility. An option that should be considered is to collocate between third layer control points and existing conventional vertical control points (particularly well monumented benchmarks).

Whenever considered necessary, a lower (forth) layer of vertical control can be constructed to serve the specific needs for vertical control of a municipal area or that of a large industrial complex. Our proposal provides also a solution for height users who insist on having a forth layer of vertical control where heights are, however orthometric and not ellipsoidal. Such a mix-up is possible provided the boundaries of the "orthometric island" (OI) are clearly outlined and measurements and their processing are performed in a clear and consistent manner as proposed in the next section of this paper. Mixing physical (orthometric heights) and geometric (ellipsoidal heights) entities is not a new idea in Geodesy. In pre-1957 times (the first “sputnik” was put in orbit in 1957) national horizontal geodetic control networks were attached to the physical body of the earth by means of astronomical (physical) observations. In our vertical OI analogy each island will have its own independent datum - just as was the case in the past when each country had its own geodetic datum.

3. Orthometric islands

Orthometric islands should be regarded as a temporary measure. They are intended to soften the shock of transition from the current orthometric heights system to the future ellipsoidal system. One can regard the OI as a compromise, a continuation of the present state of affairs with only one significant difference namely: each OI maintains an independent orthometric height datum. Thus there will be no need to maintain a consistent and up-to-date nationwide orthometric vertical control system. If a certain geographic area is marked for extensive crustal motions it may be declared an independent OI; it will have to be re-leveled frequently and be analyzed as a kinematic entity in order to determine time tagged heights. Orthometric islands may be the solution for cases where ellipsoidal heights are inapplicable. Typical examples of engineering enterprises where ellipsoidal heights are inconvenient or even impossible to use are large water system constructions, tunnel excavations with two-ends monitoring, construction in highly developed shore areas, open-ditch irrigation projects etc. Another reason for preferring or needing an OI may be due to local difficulties in collecting GPS measurements as for example in heavily wooded areas, in large cities, etc. In such cases only an OI can provide control for vertical surveys which are performed by levels, theodolites or total stations. No matter why we do establish a local OI system it should be always regarded as a secondary control which “draws” its datum from the national ellipsoidal control

(NEC) GPS-borne network (see [6]). Several variations and a number of problems can be envisioned and should be tackled as they develop:

* There may be an overlap between two adjacent islands so that a number of points may belong to both islands. A hierarchical order may have to be set to take care of such dualities.

* Certain points belonging to an island may have only orthometric heights i.e. (have not been visited by GPS). Their ellipsoidal heights will depend on the quality of the local geoid model.

A typical orthometric island is composed of m points, which are distributed within a given region and are monumented so as to create a continuous local vertical control network. The OI is surveyed by conventional (classical or motorized) precise or second order leveling techniques. Part of the m OI control points (a subset of size k) belong also to the 3-D NEC network. Through those k points the OI is "attached" to the higher order NEC network. Those k attachment points are instrumental for defining the datum of the OI. In the remaining part of this section we describe the methodology of OI datum definition:

(a) From the output of the NEC 3-D network adjustment (GPS measurements) we extract a vector h_0 (of size k) and a corresponding covariance matrix Q_{h0}.

(b) The best available geoid model (in the OI area) is employed to compute geoid undulations for each one of the k points. The result is a vector u_0 and an estimate of its covariance matrix Q_{u0}.

The two vectors are combined to create a vector of nominal orthometric heights (of size k):

$$\overline{H}_0 = h_0 - u_0$$

with a respective covariance matrix

$$Q_{\overline{H}0} = Q_{h0} + Q_{u0}$$

(c) The n (n>m) leveling measurements of the OI are adjusted by the method of parameters where the vector of m unknown orthometric heights H is partitioned into:

H_0 - heights of the k "attachment" points.

H_1 - heights of the remaining (m-k) points.

The normal equations are formed as usual, keeping in mind the above partitioning:

$$\begin{pmatrix} N_{11} & N_{10} \\ N_{01} & N_{00} \end{pmatrix} \begin{pmatrix} H_1 \\ H_0 \end{pmatrix} = \begin{pmatrix} U_1 \\ U_0 \end{pmatrix} \quad .$$

The H_1 unknowns are eliminated from the normal equations system by substitution:

$$H_1 = N_{11}^{-1} \left(U_1 - N_{10} H_0 \right) \quad .$$

As a result we obtain the reduced system of normal equations:

$$\overline{N}_{00} H_0 = \overline{U}_0$$

where

$$\overline{N}_{00} = N_{00} - N_{01}\, N_{11}^{-1}\, N_{10}$$
$$\overline{U}_0 = U_0 - N_{01}\, N_{11}^{-1}\, U_1 \quad .$$

Datum of the orthometric heights within the OI is still to be determined. Due to lack of datum the rank of the above $\overline{N}_{00}$ k by k matrix is only (k-1). The rank deficiency of $\overline{N}_{00}$ [d = k-(k-1) = 1] means that there is a whole family of different H_0 vectors all of which satisfy the reduced system of normal equations. Any two such vectors are related through Helmert's matrix E (k by 1):

$$H_{0j} = H_{0i} + E \cdot \Delta_{ij}$$

where

$$E^T = [1 \;\; 1 \;\; 1 \ldots \;\; 1]$$

and Δ_{ij} is the vertical datum shift between the $i^{\underline{th}}$ and the $j^{\underline{th}}$ solutions.

We define datum of the OI by the well known weighted free net (WFN) principle where out of the multitude of H_0 solutions we select the one (and only) solution which minimizes the following quadratic form:

$$D^T P_D D = \min$$

where

$$D = H_0 - \overline{H}_0$$

and

$$P_D = Q_{\overline{H0}}^{-1} \quad .$$

It can be shown that the unique vector H_0 , that satisfies the above minimum condition, satisfies also the following linear equations:

$$E^T P_D D = 0 = \overline{E}^T D = \overline{E}^T H_0 - \overline{E}^T \overline{H}_0 = \overline{E}^T H_0 - \overline{\overline{H}}_0$$

and finally

$$\overline{E}^T H_0 = \overline{\overline{H}}_0$$

which are known as the weighted free net constraints of H_0. In this case as the datum "defect" is of size one (d=1) the WFN constraints are in fact a single linear equation.

The reduced normal equations bordered by the above WFN constraints form a system of full rank and result in a unique solution for H_0:

$$\begin{pmatrix} H_0 \\ \lambda \end{pmatrix} = \begin{pmatrix} \overline{N}_{00} & \overline{E} \\ \overline{E}^T & 0 \end{pmatrix}^{-1} \begin{pmatrix} \overline{U}_0 \\ \overline{\overline{H}}_0 \end{pmatrix} = \begin{pmatrix} Q_{H_0} & \overline{E}\left(\overline{E}^T \overline{E}\right)^{-1} \\ \left(\overline{E}^T \overline{E}\right)^{-1} \overline{E}^T & 0 \end{pmatrix} \begin{pmatrix} \overline{U}_0 \\ \overline{\overline{H}}_0 \end{pmatrix}$$

where λ is the Lagrange multiplier of the WFN constraint.

The solution for H_0 is thus:

$$H_0 = Q_{H_0}\overline{U}_0 + \overline{E}\left(\overline{E}^T\overline{E}\right)^{-1}\overline{\overline{H}}_0 \qquad .$$

Matrix Q_{H0} , the reflexive symmetrical generalized inverse of $\overline{N}_{00}$,that happens to be also the covariance matrix of H_0, is computed by the following expression as shown in [2]:

$$Q_{H_0} = \left(\overline{N}_{00} + \overline{E}\overline{E}^T\right)^{-1} - \overline{E}\left(\overline{E}^T\overline{E}\overline{E}^T\overline{E}\right)^{-1}\overline{E}^T$$

4. Unorthodox vertical control in Israel

Geodetic activities in Israel are led by the Survey of Israel (SOI). Active participants in those geodetic activities (mainly in research and development) are also the Technion in Haifa, the Tel Aviv University, the Geological Survey in Jerusalem and the Geophysical Institute in Holon. Ideas presented in this paper, that were published earlier in [5], are not yet fully accepted in Israel. Understandably, they have not been adopted as a basis for the official geodetic policy of the state. What we intend to do in this section is to describe a few existing components of our still nonexistent ellipsoidal vertical geodetic control system. Pending an official adoption of our proposals those components could be easily put together with little additional work so as to form a living example of our "future" vertical geodetic control system.

The vertical geodetic control network in Israel is conventional i.e. it is based on precise leveling. The primary net has been re-leveled and readjusted recently in order to update the rapidly changing heights of its points. Due to budget cuts it seems that the SOI will not be able to embark in the foreseeable future in additional massive re-leveling of the primary net. The inevitable effect of the above situation is that in just a few years the adjusted heights will become obsolete and the nationwide vertical control network will become a fiction. A possible remedy at the moment seems to be the marking of a number of orthometric islands in regions and municipal areas where budgets for re-leveling could be obtained.

Our NEC exists in 3-D in the form of a dense network of some 150 extremely well monumented control points - the so-called G1 geodynamic network. The GPS measurements campaign of the G1 lasted a period of little more than half a year (1996-97) where each point was occupied by a dual frequency P-code GPS receiver for at least two 24-hours sessions. The accuracy of the G1 network is estimated at the level (one sigma) of 4-5 mm in horizontal position and about 10 mm in ellipsoidal heights. Three (out of the seven presently operating) permanent GPS stations were active during all of the G1 measurement campaign and provided primary geodetic control for the adjustment of the G1 net. By its accuracy and also by the distribution of its points (see figure 1) the G1 can very effectively serve as a NEC network for the State of Israel. In the remaining part of this section we describe three existing orthometric islands (OI) which are in the process of being connected to the NEC by GPS measurements. A forth island (still in the design stages) will serve as a laboratory for experimenting with "pure" ellipsoidal height control.

(a) The Mediterranean-Sea island

Part of the existing primary conventional (orthometric) vertical control network along the Mediterranean Sea shores of the State of Israel forms a narrow strip roughly 10 km wide. (see fig. 2). The Med-Sea island will be re-leveled, readjusted and attached to the NEC. In fact, due to the recent (1997-98) readjustment mentioned above, all that has to be done at the moment is to attach a number of its points (benchmarks) to the NEC. The geoid model which is an indispensable part of the attachment process (see section 3) is still of insufficient quality and a special effort will have to be made for its eventual improvement before it could be used to attach the island to the NEC.

(b) **The Mount Carmel Island**

As part of a long range research project for improving the geoid model of the State of Israel (see [4]), an area of some 600 km^2 has been surveyed intensively by GPS and by short-leg precise trigonometric leveling. The 67 control points form a contiguous island, which overlaps with the Med-Sea island. In this case as the geoid model is of a superior quality with an accuracy of 4 cm (one sigma) all that remains to be done is to attach the Mount Carmel island to the G1 network (NEC) by GPS measurements.

(c) **The Eilat (Red Sea) Island**

This tiny (20 Km^2) island has been set as a pilot project to explore possibilities for the creation of small "municipal" OI. A dense network of control points set at ½ km distance has been surveyed by second order leveling as well as by GPS. The island is already attached to the NEC (G1 net) and its local geoid model, due to the extremely high density of surveyed points, is accurate to better than 1 cm (one sigma).

(d) **The city and environs of Modyin island**

This island is still on the "drawing table". The new city of Modyin and its environs are located far from the Mediterranean Sea. As such they seem to be ideal for conducting an experiment with "ellipsoidal" islands (EI). In such a case, the island is nothing else but a densification of vertical ellipsoidal control within the islands boundaries. The local geoid model will have to be improved dramatically on the basis of our experience with (b) and (c). So far from our limited experience with the compilation of unorthodox vertical control in Israel we have learned that it can be extremely flexible and adaptive.

5. Summary

In trying to sum up the ideas that were presented in this paper we would like to make a number of observations taking the position of a "devil's advocate" of our proposal. We list items which are instrumental for the realization of an ellipsoidal vertical control system and which, at least at the moment, seem to be missing, inadequate or prohibitively expensive and hard to get:

(a) GPS is still unable to provide reliable and accurate to 3-4 mm ellipsoidal height differences.

Only by using chock-ring antennae and after long measurement sessions (12 hours and up) can we hope to get closer to that accuracy goal.

(b) High accuracy (1 cm sigma) local geoid models (within the range of an orthometric island) are still a hard to get commodity. Development of models at that level of accuracy requires extensive fieldwork in addition to the integration of gravity and topographic data. Airborne gravimetry may hold a solution of the precision geoid problem by providing the necessary data in remote or inaccessible areas.

(c) The community of height users is still reluctant to part with the orthometric heights to which it has been accustomed so far. The proper way of "making friends" is to let them have a fair chance to work with ellipsoidal heights and learn from their own experience. We, the ellipsoidal-height providers, will have to follow them up and help in solving the many smaller problems on the way, as they arise.

In spite of the above listed difficulties we think that the transition from orthometric to ellipsoidal vertical control is inevitable and in fact it is already happening. The final format may be somewhat different from the model that was described in sections 2 and 3 above. We believe that it is our duty to explore honestly the new ellipsoidal alternative for vertical control. To do that, first of all, we have to suppress our inherent professional prejudices and suspicions. We, the small community of thinking-ahead-of-our-times individuals, have to prepare for the inevitable transition from orthometric to ellipsoidal vertical control by intensifying our research and by gaining experience with the new media in small scale experiments as well as in more ambitious pilot projects. Extensive research during a quarter of a century has brought up excellent results in improving conventional leveling. Let us hope that a similar wave of enthusiasm coupled with plenty of talent will be invested in studying the new technique and will bring us soon to

the vertical geodetic control of the future.

References

[1] Kaula W.M., (1987), "The Need for Vertical Control", Surveying and Mapping, Vol. 47, No. 1, pp. 57-64.

[2] Koch K.R., (1988), "Parameter Estimation and Hypothesis Testing in Linear Models", Springer Verlag, Berlin, Heidelberg.

[3] Makinen J. and Saaranen V., (1998), "Determination of Post-Glacial Land Uplift from the Three Precise Levellings in Finland", Journal of Geodesy, 72, pp. 516-529.

[4] Sharni D., Papo H. and Forrai J., (1998), "The Geoid in Israel: Haifa Pilot", in "Second Continental Workshop on the Geoid in Europe", Budapest, Hungary.

[5] Steinberg G. and Papo H., (1998), "Ellipsoidal Heights: The Future of Vertical Geodetic Control", GPS World, Vol.9 , No.2, pp. 41-43.

[6] Wells D., Kleusberg A. and Vanicek P., (1996), "A Seamless Vertical Reference Surface for Acquisition Management, and Display of Hydrographic Data", Report submitted to The Canadian Hydrographic Service.

[7] Zilkoski D.B., (1993), "A Strategy for an Orderly Transition from Leveling-Derived Orthometric Heights to GPS-Derived Orthometric Heights", ACSM/ASPRS Annual Convention, New Orleans, Louisiana, USA.

Requirements on the height benchmarks in the third precise levelling of Sweden.

Per-Ola Eriksson

ABSTRACT

The development in the field of surveying is very intensive today. The instrumentation as total stations, levelling instruments and GPS-equipment is constantly improved in order to be easier to use and to achieve better accuracy of the measurements. Calculation programmes are also improved for the same reasons.

However, most measurements are related to some kind of reference points in the terrain that should carry the coordinates or the heights calculated from the measurements, so what happens if the benchmarks does not meet the standard of the measurements is that the good measurements performed are loosing a bit of its value. The efforts made to achieve high-class measurements are more or less wasted. Or can we really afford to make those accurate measurements just for the pleasure to see that nice low RMS in the result files from the calculation?

Not only the stability and accessibility of the benchmark is important, but also to be sure that the point used in the field is the right one. To guarantee that, it is necessary to have a reliable identification and documentation of all the points.

The quality of a measurement can easily be judged by calculation, and we can accept the measurement or remeasure, but how can the quality of a benchmark be judged? Unfortunately this can usually not be done until the point is used the next time, perhaps after several years, and then it is too late.

This paper mainly describes the requirements on the benchmarks in the third precise levelling of Sweden and what is done in order to meet those requirements. The actions taken here can however be applied to most kinds of networks.

INTRODUCTION

The third precise levelling of Sweden has been going on since 1979. The network will consist of about 50 000 km double run levelling and about 50 000 benchmarks to represent the measurements in the terrain.

Before the project started the requirement of high quality benchmarks was realized. Earlier levellings in Sweden had suffered from unreliable benchmarks in different ways, and that had essentially decreased the value of those works. Therefore we could not perform a project like this with such an enormous amount of high quality measurements without trying our very best to secure the measurements in the terrain for the users.

This paper describes mainly the actions taken in order to establish reliable and permanent benchmarks in the third precise levelling, but the problems are mostly the same for all kinds of network. Besides, at least in Sweden, vertical movements are the most critical for a benchmark because of the ground frost, so we have to pay special attention to those problems when we deal with a vertical network.

The paper will give an account for the whole process of benchmarking, from planning the new points to updating and maintenance.

A good benchmark means the right kind of marker applied in the right way in the most stabile kind of foundation at the right place for the users. The point is permanent to destruction, it is distinctly identified and the information connected to the point is updated and relevant.

With the development of the GPS-technique, the need of benchmarks is beginning to be called in question, especially for horizontal networks, but also for vertical networks. It should not be necessary to connect the measurements to benchmarks since we can determine the measuring points on each occasion. We should only need a few reference points in order to connect the measurements. For the vertical component ellipsoidal heights could be used.

For the local users however this arrangement is not a realistic alternative within the foreseeable future, since many local users do not yet have access to the GPS-technique. On the local level many measurements of different kinds are done for many different purposes, and in many cases the GPS-technique is not available or the most suitable. Ellipsoidal heights are for practical reasons impossible to handle for most local users. So classical techniques will still have an important role in this field for many years, and that requires as good benchmarks as possible. Besides, it has always been necessary to connect measurements from different epochs, and then the benchmarks are essential as a carrier of earlier measurements.

REQUIREMENTS ON THE BENCHMARKS

A study group at the NLS worked with the specifications for the new levelling project for several years, often in collaboration with our Nordic colleagues. The importance of high quality benchmarks was stressed in the report presented from the study group in 1976. This is shown in the following extract from the long list of specifications regarding the benchmarks:

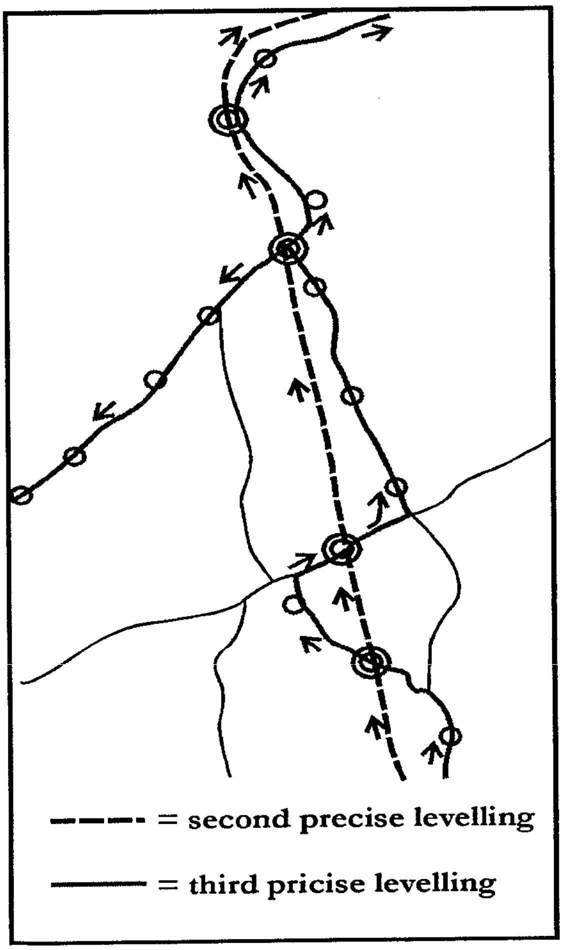
- The net should be shaped so that all polygons have equal circumference all over the country (between 80 and 120 km).
- The net should cover the whole country, which is not done in previous precise levellings.
- When the location of the lines is planned, the local needs and demands should be taken in consideration as far as possible.
- The distance between the benchmarks should be 1 km.
- The choice of location of the points should be done with the greatest care and every possible effort (since those are the fundament of the network and should guarantee the permanency for many years). The prime foundation should be bedrock.
- The marker should be designed with a well-defined highest point so that a rod with a flat bottom can be set up vertically.
- The point should be located so that the risk of destruction is minimised.
- The marker should be equipped with an identification, and a description should be drawn up in such way that no mix up with other points can be done.
- If the distance between points in bedrock is too far, special measures will have to be taken. (Type of benchmark described below).

- The point number system should be based upon the map sheet system in order to generate unique point numbers.
- All information about the points should be stored in a database.
- Maintenance and updating of the new network should be carried out on a regular basis.

Considering that those recommendations were drawn up in 1976, one must say that they were very well thought-out, and they are mostly still up to date. Since those guidelines have been at hand from the very beginning of the work, it is all done in the same shape, which is a big advantage. Let us now see what have been done in order to fulfil the recommendations.

PLANNING THE NETWORK

When we are planning for new measurements we must take into account that different users need the established points for future measurements. Therefore it is important that the points are located so that they are easy accessible in order to minimise future measurements for connections, since those measurements means extra costs every time they are carried out. The distance between the points is about 1 km, and must not exceed 1,5 km. In urban areas the distance is shorter. The points should be located where they are needed.



Fig; 1 Location of new lines along old precise levelling lines

In order to assure that, a map with the planned lines is sent to all the local users, community authorities, road and railroad authorities and others who can possibly be users of the points. They are invited to give their opinion of the plan, and when their view is collected, we can make a final plan, where we try to combine all the different demands. This is not always so easy. Sometimes all the local demands cannot be fulfilled. Different local users can have totally different opinions of the location of a line. We also have certain demands of our own upon the network configuration in order to obtain a strong and homogeneous network all over the country *(see fig.1)*. In those cases we have to find the best common solution together.

This process starts one year before the fieldwork (*see table.1*), but when it is done we know that the established points will be as useful as possible to as many users as possible.

Before the fieldwork starts, we then visit each one of those users to discuss local matters. That could be the location of the lines on a detail level. Benchmark maps and descriptions over the local networks are collected. Perhaps the local user has points of his own that can be used, and in that way have the local network connected to the national network without any extra measurement. In addition to that we can avoid setting out another point at the same place, which could cause mistakes by using the wrong point. That can happen if we do not know the location of the local points.

Local points can be used if they meet all the demands valid for benchmarks in the precise levelling network. Many local officials proudly shows maps and excellent calculations from their measurements at the office, but when the benchmarks are inspected in the field, it turns out far too often that the local points are not possible to use. Since we cannot risk the quality of the precise levelling network by using those points, the local user will have to connect his measurements himself, which means extra costs. In addition to that he has in fact an unreliable network, in spite of the good measurements, and that will cost even more in the long run.

Sometimes a local user wants more points or a denser network than the specifications for the national network. In that case the user will have to pay for that extra densification. Such additional points or lines can be prepared either by the local authorities or the NLS, and can be levelled at the same time as the measurements in the national network. The result from the benchmarking in the national network can be sent to the clients immediately after the fieldwork. Since the benchmarking is performed one year before the levelling, the clients will have time enough to decide on those matters.

There are other reasons to establish the points one year in advance. One is that the points will have time to stabilise and rest over the winter between benchmarking and the levelling. This is important especially for the underground type of benchmark described below.

Since we are setting out about 2 000 points/year we will need some time to prepare the levelling. Each levelling team must be equipped with all the descriptions and maps they will need during the field season before they go out into the field.

	Jan	Feb	March	April	May	June	July	Aug	Sept	Oct	Nov	Dec
Year 1									1	1	1	1
Year 2	1	1	1	1 / 2	2	2	2	2	2	2 / 3	3	3
Year 3	3	3	3	3 / 4	4	4	4	4	4	4 / 5	5	5
Year 4	5	5	5	5								

Table 1: Time schedule for the process

1: Planning of the network. Planning for the location of the lines in collaboration with the local users

2: Field work. Setting out the benchmarks. Discussions on detail level with the local users.

3: Storing benchmark descriptions into databases. Preparations for levelling.

4: Field work. Levelling.

5: Calculation. Storing levelling data and results into databases. Delivery of data to the users.

FIELD WORK

Location of the benchmarks

When a network is established, disregarding what kind of network, it is desirable that the "lifetime" of the points is as long as possible. A point like this turns out to be quite valuable when we sum up all the costs for benchmarking, levelling, and calculation. All that money is in fact invested in the very benchmark, and if the benchmark is destroyed after a short time, the investment does not pay off. Today we are also very interested to connect the points from the first precise levelling from 1886 – 1905, and the second one from 1951 – 1967, in order to establish the relation between former height systems and the new one to be. In the Nordic countries there is land uplift that can be calculated if we know those relations. This is an important task of the project, and that would be impossible if there should be too few or no old points left to connect. Therefore there are always lines located to the same route as the lines from the previous precise levellings.

In Sweden the law protects benchmarks established by NLS or a local user. It is not allowed to deliberately destroy a benchmark without permission from the owner. This law is very hard to practice, since a person who removes a benchmark can always say that he was not aware of the benchmark when he destroyed it. Road constructors are the people who destroy the largest number of benchmarks. That is quite natural since most points are located along roads. Therefore the road planning authorities are always asked about their long-term plans for bigger projects of road constructions. There are always small projects that can not possibly be foreseen, but many points can be saved by those interviews.

This means that we should try to locate the new points so that they are well protected from destruction. At the same time we want the points to be accessible. If we add the requirement for stabile foundation of the benchmark, we can easily see that we can have a problem. Where can we locate a point that fulfils all these demands? We will often have to compromise between the different demands. If we have to choose, the most important demand probably is to have a stabile point. Second choice is the permanency of the point and the third is the accessibility. It is no use to have an accessible point if the coordinates or the height given are not valid or if the point is destroyed shortly after the establishment.

All those considerations have to be done by the personnel who shall set out the benchmarks, and that really takes a lot of experience in ground analyse as well as in surveying. A great deal of patience and common sense is also valuable. Not anybody should be trusted to do this work, because the responsibility of the total quality of a network is really in the hands of the people who are doing this work. Unfortunately this responsibility is not seldom handed over to the youngest or latest employed assistant, who is sent out to set out the points. If the guidelines for the work are poor in addition, the whole network is more or less ruined even before any measurements have been done.

Foundation

In Sweden and the other Nordic countries we have ground frost, that in Sweden can be up to 2.0 m below the ground in certain types of soil, or even more when there is no snow in the winter. The ground frost can lift even a big boulder or a steel pipe that is used as a benchmark. This is a problem where we cannot use bedrock for the benchmarks, and in large areas of the country there is no bedrock. The depth of the ground frost of course is dependent of the temperature, but it also varies with the type of soil. The ground frost is deeper in soil that contains more water. Therefore clay and some types of moraine are more dangerous to use as foundation for a benchmark than sand or other well-drained types of soil.

The best kind of foundation is the bedrock, which is normally very stabile in Sweden. It is important that the bedrock has no cracks on the surface or right under the surface. If it is cracked the frost can burst the bedrock. This can easily be detected if we knock on the bedrock with a hammer. However bedrock is not available in all parts of the country. Therefore boulders or blocks of stone are also used as foundation if they are properly embedded in the ground. This is the most common type of foundation for height benchmarks in Sweden *(see table 2)*. If there are no blocks available, buildings, bridges or other constructions can be used, if they are founded well enough. Since points located in constructions are often horizontally mounted, it is important to assure that we really can set up the rod vertically on the point. Too often there is a leaning wall, a roof that is too low or some other obstacle that makes the points impossible to use.

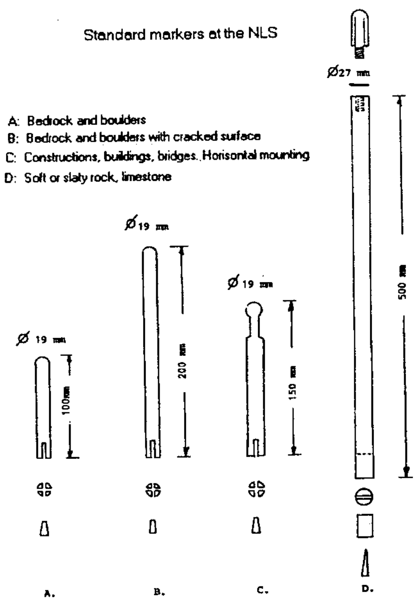
Type of foundation	Nr of benchmarks	% of the total number
Benchmarks in bedrock	16 768	36 %
Benchmarks in stone blocks	25 360	54 %
Benchmarks in constructions	2 674	6 %
Underground benchmarks	1 851	4 %

Table 2; The table shows the distribution among the different types of foundation in the Third precise levelling of Sweden up to 1997. The total number is 46 686.

When new measurements are connected it is important that the connection points are reliable. Otherwise we have to spend money on connection measurements, which should be unnecessary. In order not to get too many weak points in a row on the lines, in the third precise levelling of Sweden we say that at least in the urban areas every fourth point should be founded in bedrock. Since this cannot be fulfilled, we have another method to establish reliable points. This method is described below.

Choice of type of marker

The quality of a benchmark is not only dependent of the foundation. We also have to use a suitable type of marker for each type of foundation and for each type of measurement. In various exhibitions we can see many different types of markers, each one intended for a special kind of foundation and measurement.

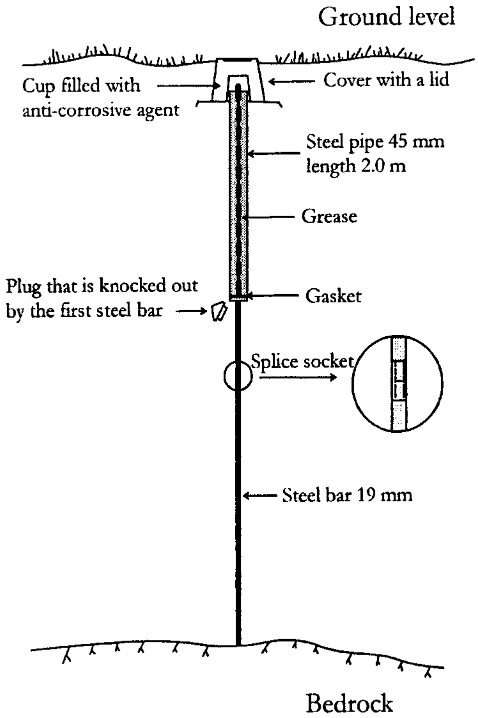


Fig;2 *Common types of markers in the third precise levelling of Sweden. All types are made of stainless steel.*

For levelling we want to have a type of marker with a distinct highest point. Therefore this kind of marker should have a spherical upper surface. In that way the bottom of the rod always stays on the highest point of the marker. For horizontal installation these markers can be shaped like a ball or a cylinder. The safest way to anchor the marker into the foundation is to have a wedge, flat or conical, that is expanding the bottom of the marker when the marker is hammered down into the borehole. In constructions other types of expanders are used *(see fig.2)*

Not only the ground frost can be a problem. When expanding a marker into a borehole we must prevent water from coming down into the hole. Otherwise the water stays on the bottom of the hole and when it is freezing in the winter it can lift the marker in the hole, even if it is fastened with an expander. This can easily be avoided by putting some waterproof silicone in the hole before driving the marker down.

To establish reliable levelling points where we have no bedrock, there is another method *(see fig.3 and 4.)*. It is done with a hydraulic hammer running along a portable rig. First we dig a small hole about 60 x 60 cm and 40 cm deep. Then the rig is set up in the hole and a 2.0-m long steel pipe with the diameter of 45 mm is driven down into the ground. In the bottom of the pipe there is a loose conical plug to prevent the soil to get into the pipe. The first steel bar later pushes out the plug. The pipe is filled with grease to prevent water from getting into the pipe. Inside the pipe a 3 -m long steel bar is driven down through a gasket in the bottom of the pipe so that the grease does not go out with the bar. When the bar is knocked down another one is spliced and then the joined bars are driven down until they stop. Hopefully we reach the bedrock, but even if we do not, the depth is enough to give a very stabile point. The medium depth of those points is about 15 m, but the variations are very big. The last steel bar is cut off above the top of the pipe and the top is shaped to a sphere, which makes the benchmark. The benchmark is protected with a cover with a lid on the ground level With this method we get a benchmark that is protected from the ground frost by the steel pipe. The frost can lift the pipe, but the steel bar inside is not affected since it has no contact with the pipe. The grease in the pipe stops the water from coming in and freeze between the pipe and the bar.



Fig; 3. Sketch of a benchmark where we have no bedrock

Fig: 4. An underground benchmark is set out.

This kind of points is considered almost as reliable as a point located in bedrock. They are quite expensive to establish though, since they take between two hours and half a day to produce, sometimes even more, depending on the type of soil and the depth. It also requires some equipment, and the material is quite expensive compared to a standard marker. However compared to the costs for future unnecessary control measurements, this is a good investment.

For horizontal measurements it is important to have a distinct point so that we can set up the instrument exactly over the same point each time the point is used. Often we want to set up a target of some kind over such a point. Therefore those markers mostly consists of some kind of pipe, where a stick with a prism or a signal can be set up. Sometimes it can be sufficient to drive a steel pipe 0.4 – 2.0 m down into the ground, depending on the risk of ground frost. In buildings a plate can be mounted on the wall, and then a special device is attached to the plate every time when measurements shall take place. That gives good protection to the points, but you must be equipped with that special device in order to be able to use the points. For all kinds of markers, it is a good idea to have a mark on the benchmark that tells who is the owner of the point. This can be done by having the name of the organisation punched on to the marker. Then a user will know who can give information about the point.

There are also markers for combined measurements. However they are often more suitable for either horizontal or vertical measurements, and seldom perfect for both purposes.

IDENTIFICATION

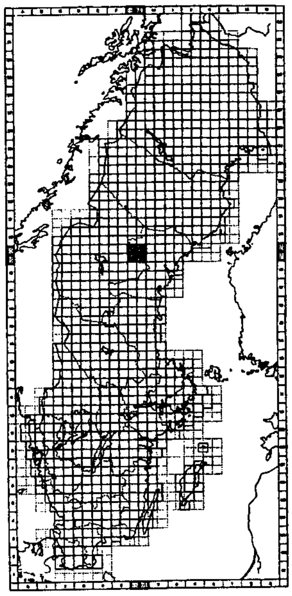


Fig:5. Map showing the mapsheet system and the point number system. Sheet N:r 19G =196 is marked.

Benchmark numbers

We also must have a "waterproof" system for identification of the points, and descriptions containing all information required, not making mistakes when we shall use the points. A point number system that gives unique point numbers is of course essential.

The point number system in Sweden is built up by the map sheet system *(see fig 5)*, where the sheets in the scale 1:100 000 contains 10 x 10 maps in scale 1:10 000.

Those sheets are numbered within each 1:100 000 map. To identify the area of such a map we use 3 digits to identify the 1:100 000-map sheet and two more to identify each 5 x 5-km area. For each of those areas we finally add two digits for consecutive numbers. This system allows us to have 99 points in a 5 x 5-km area. The most crowded squares contains about 30 points. There are points more than 100 years old in the system, so it will probably last another 100 years. In addition to that we have for practical reasons codes for different types of points included in the point numbers. So when a new point shall be established, we must always look at the archive map to check what is the lowest available number in each square

Benchmark descriptions

In order to find the points and to secure that the used points are the right ones, we need a benchmark description. The description tells where the points are located and contains at least two distinct measures to permanent details in the terrain. A sketch shows the location of the benchmark in relation to other objects in the surroundings. The description should also tell if the point is identical with an older one, a local point etc, or if there is such a point in the neighbourhood, that can cause a mix up between the points. The description also tells when a point has been established and used and what measurements it has been involved in.

The benchmark descriptions are drawn up at the same time as the points are set out. The heights are not given on the description though. They are printed out on separate lists. All the information is stored in databases, including the sketch. That makes it easy to update the descriptions. This work is done during the winter after the fieldwork. We want to induce the users to get the information on the points from our archive every time he wants to use them. In that way he can be sure to have the most actual information.

Benchmark maps

To complete the information about the benchmarks we need a benchmark map that shows all the benchmarks on a mapsheet and their point numbers. Different types of points are drawn with different symbols. The point numbers are determined from the archive map according to the system mentioned above. The archive map is stored in a portable PC, and the new points are digitised on the screen of that PC. Every new point must be inserted on the same copy of that archive map so that everyone working can see which point numbers are occupied. If there are more than one team working, they use the same copy of the database stored on a PCMCIA-card. That card is shifted between the computers in order to avoid errors with the numbers.

This work is done in the evening after each working day. Mistakes with numbering of the points will be detected during the process, but it is easier to correct the errors the sooner they are detected. Built in control functions in the digitising programme helps to avoid some mistakes. So when the fieldwork is finished for the season, we store the data from the computers into the original database, and then we can print out the maps at once. The maps are sent to the local users so that they can see the locations of the new points and decide if they need additional points.

MAINTENANCE

Building a network like the third precise levelling in Sweden can be seen like a gigantic investment that is calculated to pay off over a very long time. In spite of all the measures taken in order to preserve the network when it is built, things happens that in different ways is causing damage to the network. To protect the investment we must therefore maintain the work that is done. This is often an underestimated problem. Benchmarks are destroyed or damaged. The location of the points is changing and makes it hard or impossible to find the points. This process is going on all the time, and if nothing is done the value of the investment is decreasing very fast. What good is it to have accurate benchmark descriptions and exact heights if you cannot find the points or if there are no points left in the terrain? In 1992 an investigation was made that showed that about 1% of the points from the third precise levelling are destroyed every year in the urban areas, and 0,5% in the woodlands. The oldest parts of the network were at that time 13 years.

After that discovery a programme started in order to systematically update the network, even though the network is not completed yet. The ambition is to keep the network at the same standard as it was originally, so there is no extra points or lines established in this work. The local users are involved in this work in the same way as in the establishment of the network. We are now updating the urban areas 13 – 15 years after the establishment, and that means that 10 – 15 % of the points are gone in each area. All the points are visited. The destroyed or damaged points are replaced by new ones or local points, and all the benchmark descriptions are checked and updated, the text information as well as the sketch. All the changes are stored in the databases. To separate those new points from the original ones, the new points are given a special type in the benchmark number. They also have a special symbol on the benchmark maps to point out to the users that something has happened. If the new point is located close to the destroyed one, a user can make a mistake if he is not aware of the replacement.

Conclusions

When a measurement is performed we always need to connect it to other measurements in some way. For that purpose we need benchmarks that have preserved the quality of those former measurements. This will be the case also for the foreseeable future. Since measurements today gets more accurate, the requirements on the benchmarks must also increase. Otherwise the accurate measurements can not be utilised, especially in the long run.

For economic reasons we must plan the network so that the points are as useful as possible to as many users as possible. Measurements for connections and control measurements cost as much as ordinary measurements, and should be unnecessary if the points are reliable and located at the right place. The type of marker used should be suitable for the type of measurement that should be performed. The possibility to safely identify a point is essential. Descriptions should be distinct and contain all the necessary information. In order to protect the investment made in the network, updating must be done on a regular basis.

Unfortunately there are no formulas that can help us to judge the result from the benchmarking like we can when it comes to measurements. That is why we must trust the common sense by the personnel who is dealing with these matters. The more experienced people we use here, the better result we will get.

Taking these factors in account when networks are built and when the benchmarks are set out, the points will be a little more expensive to establish. Compared to the costs for just one extra km double run precise levelling for connection or control measurement, at one occasion, this extra cost is earned several times.

References

Projektgruppen för Riksavvägningen vid LMV (1976): Precisions- och huvudlinjeavvägningens förnyelse. (In Swedish).

Becker, J-M (1984): Uppbyggandet av Sveriges nya riksnät i höjd. Lantmäteriverkets Tekniska Skrifter, 1984:1, 12pp. (In Swedish).

Becker, J-M (1985): The Swdish experience with motorised levelling, new techniques and tests. Presented at the NAVD Symposium, April 21-26, 1985. Lantmäteriverkets Tekniska Skrifter, 1985:10, 54 pp.

Eriksson, P-O, Becker, J-M (1994): Updating of the Swedish primary height network. How to update? Results from test measurements. Presented at the Nordic Geodetic Commissions, 12 General Meeting, Ullensvang, Norway, 1994.

Becker, J-M (1997): Riksavvägningsarbeten i Sverige under perioden 1974 – 1995. Lantmäteriverkets Tekniska Skrifter, 1997:2, 23pp. (In Swedish).

Implementation of a new, common height datum in Denmark

S Stampe Villadsen

National Survey and Cadastre, Denmark

Abstract

The societies of today are becoming more and more complex, integrated and international. The demands of standardization are in general increasing, such as introductions of geodetic datums. The paper will focus on the practical aspects in measuring the detailed height points in Denmark, and the different society aspects needed to consider in implementing a new, common height datum.

A. Background

A 3. precise levelling was carried out from 1982 - 1993. 85% of the levelling were performed in the years from 1986 to 1991, with two motorized levelling teams observing about 5 month every year. The 3. precise levelling was finalized with smaller parts of line measured in between the main lines in 1992, 1993 and 1994.

The connection between Denmark and Germany from Rødby to Puttgarten, and from Sealand to Funen was carried out as Hydrostatic levelling.

The adjustment were carried out as geopotential numbers, fully tidally adjusted. Mean sea surface in 10 tide gauge stations in Denmark gave the absolute level of the datum. After the adjustment the result is given as ortometric Helmert height, with the height of the fundamental bench mark in Århus Cathedral set at:

5,565 m

5.0 cm lower than the old height system DNN GM from 1891.

The precise levelling showed, that the northern part of Jutland has raised about 1-2 cm, and the southern part of Jutland lowered about 15-18 cm in the last 100 year.

B. Getting the new height system out in the country.

Totally 4.200 km levelling line is observed in both directions in the 3. precise levelling. 9.8% of all the levelling is rejected. About 7.700 points have now a observed height in the new Datum DNN KMS90. But what about the rest of Denmark ?

Denmark has about 80.000 benchmarks. Approx. 24.000 is situated along main levelling lines the rest along minor roads and in town areas. The State, in practice KMS, are responsible for the main levelling lines, other users, mostly the municipality are responsible for the rest of the benchmarks.

The height system is build up over a long period. The main lines up till 1940, the densification in a very busy period through, - and immediatly after the second world war. In practice, it was the Geodetic Institute the predecessor for KMS that densified the height system, in a helping program for unemployed workers. Every densification was done after wish from, and in a cooperation with the local municipality. The money for the densification was taken from special sources for unemployed workers and from the municipality.

Since mid 50'is the levelling system is only kept alive with very little effort, and first with introduction of motorized geometrical levelling in Denmark after Swedish model in the 80'ies effort was brought into the levelling system again.

There is a strong need all over Denmark for good height information. Main part of the sewing system in the towns areas need a renovation, and many benchmarks are being destroyed, or worse, the height of the benchmark are not to be trusted because of local settings etc. Too, there is a need for bencmarks in all the new developed areas build since mid 50'ies. In total there is a need for new height information brougt out to approx. 110.000 existing and new bencmarks.

On top of the problem with trustworthy heights comes the new problem with changing of Height System, - how to solve ?

C. Technical solution

Since introduction of the motorized geometric levelling in Denmark the equipment is optimized, but there is an upper limit, given by the lenght and sight that sets the maximum of the production. Too, the results is in practice too good for the use of the bencmarks. Except for the precise levelling line, there is very few places that have a need for a height of a quality given by the geometric levelling.

From Sweden we adopted the idea of Trigonometric levelling, adjusted to danish purposes. In Denmark we use 2 cars both with totalstation, and in total 4 persons on a team. In practice all the levelling in Denmark is divided between trigonometric levelling and geometric levelling equipment. The main lines, or the State part, in the levelling is carried out as geometrical levelling, the rest with trigonometric levelling. The division is done to secure trustworthy heights all over Denmark, so the minor precise trigonometric levelling can fit in to ploygons measured with geometrical levelling. In total two teams are equipt with trigonometric levelling gear, one team with geometric gear.

In 1998 we have started production with automatic registration equipment in the geometrical levelling. The results of the use is promising. The Daily production is around 20 - 25 km with a RMS around 1.2 mm*√km.

In total the equipment, together with the equipment from the private surveyors, is in place to handling the height problem in a 10 - 12 years period

C. Political solution

Since 1992 where the main part of the precise levelling was finalized, on of our main task in KMS has been to go out in the country to tell about height, height system, problems with the existing height, and the new DNN KMS90. We have had talks with every municipality in Denmark about the behaviour of the country, how the crust is moving, and how local settings and loss of benchmarks has reduced the quality of the existing height system.

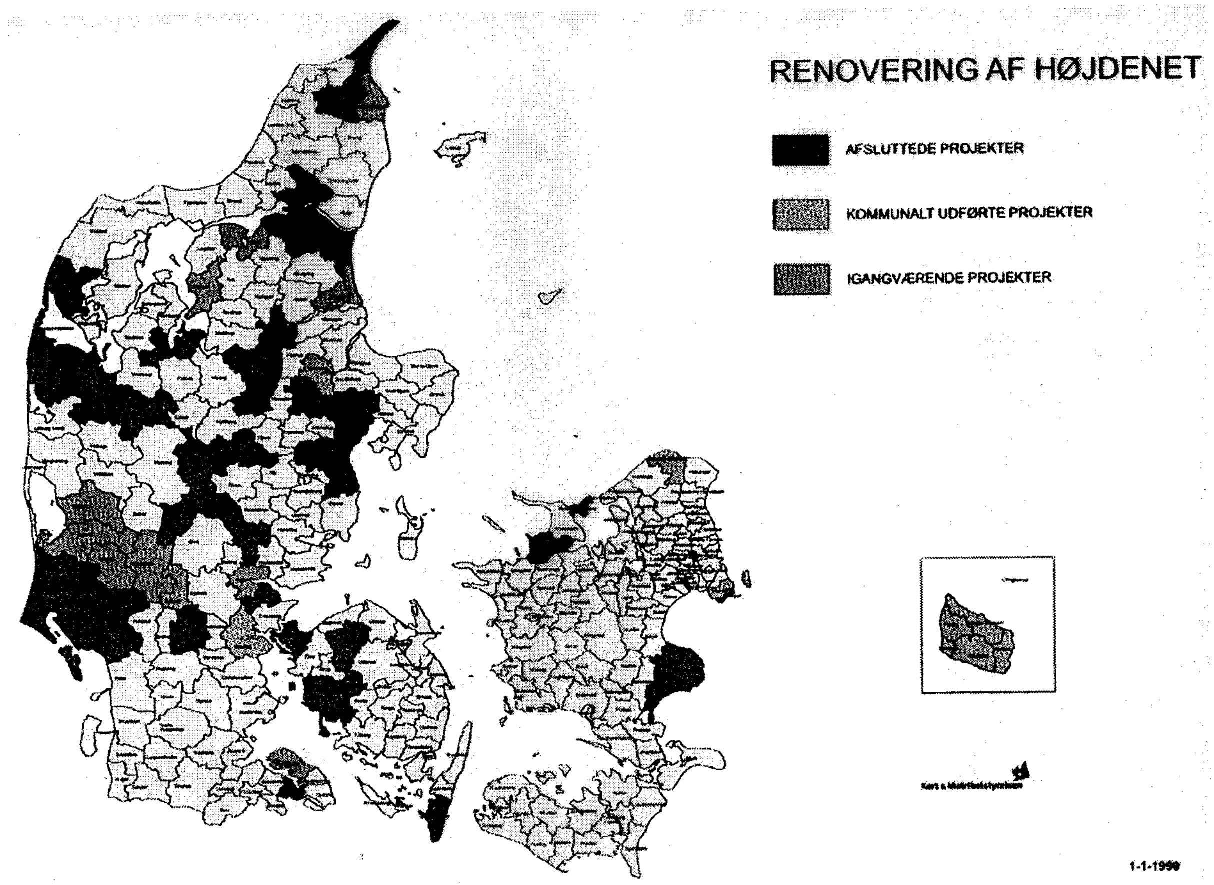


Fig. 1 Map showing where contracts with municipalities and counties of renewal of the levelling system is signed. The darkest areas shows the municipalities where the renewal is finished. The lighter areas where the renewal is started.

The practical solution to the height problems is new levelling, a solution that many of the municipalities are in agree with KMS about. The only problem is the financing. Because parts of the old network was build up with money taken from special sources for unemployed workers, many of the municipalities think that the renovation of the height system is a state problem. The discussion about this problem is still not finished, but in the years from 1992 till today about 70 of Denmark's 275 municipalities has signed a contract with KMS about renewal of their height system. In all contracts a remark is given, that in time the heights are to be calculated in DNN KMS90, but until that time comes, the heights are given in one of the 3 *) existing height systems in Denmark

Two major problems in changing is still to be solved. How and when ?

In Denmark all the important questions about the Reference Systems are discussed in the Council for Denmark's Geographical Reference Network. In the "Council" all the major user are members. KMS, Municipalities, Road Directory, Counties are the most important members.

Two years ago the Council established a working group about problems in introducing the new DNN. The working group saw the worst problem in changing in the sewing systems, both in the Databases and in the many analog informations given in this sector. The working group expressed however, that the problems in this sector was <u>not</u> in changing to a new height DNN, but in getting all the errors out coming from the inhomogeneous old height system.

The working group suggested that the database changes should be done in two steps. First a transformation from the old inhomogeneous system to a new renovated, but old system. The transformation was given by the differences in height to all benchmarks in the municipality before and after the renovation, with every "manhole" to the sewer system put in a triangle given by the three nearest benchmarks. Secondly the change in height datum would only be a simple transformation given by the differences in the Datums inside the municipality. This second transformation should first be done when a whole area was ready to change.

All transformation of the database can be done for 25 - 50.000 DKK pr. municipality.

D. Final Remarks

The new DNN is except in KMS only introduced to one major user, the Cost Directorate, who is responsible for the security of the West-Coast of Denmark. The new DNN together with a new geoid give them special good possibilities in all their GPS-measurements.

Before given the final advice to all other users of how and when to change, if they want, a questionary was send out to all the major user in Denmark including the counties and municipalities. The main question was how and when the change.

From counties 8 out of 14 answered. Of these eight, seven of them wanted to change to a new height system in a few years time, but regional or national. Only one of the counties was not sure of the benefit in changing. From municipalities the answers were more differentiated. 106 of 275 has answered the questions. Of these 106 municipalities 97 has answered the question of if, or how to change.

National change to DNN KMS90	44
Regional change to DNN KMS90	40
Change only depending of the municipalities own wish	24
Don't want to change to DNN KMS90	23

In the scheme there are one or more answers from each municipality. Of the 23 municipalities who don´t want to change, 13 of them has also marked, that they want a national or regional change. The answer from these municipalities could be, that we don't want to change now.

In addition to the positive answers one can add 41 municipalities who in their contract with KMS has agreed in changing of height to the new DNN KMS90.

The result from the questionary is now given back to the "Councils" working group, and we expect them to give the advice to the users of how and when to change at the end of this year. It is important to say, that we in Denmark until now not are able to give specific instructions of use of a specific height system. We had to convince the users that it is a good idea, and a benefit for us all.

The societies of today are becoming more and more complex, integrated and international. The demands of standardization are in general increasing, such as introductions of geodetic datums. A good way of thinking of our work is in fact to think of it as a standardization process. What we are doing is to give new standards in heights with an optimal fit to the surface of the earths crust, and using this specific new height standard gives a benefit to the whole society.

* In Denmark in addition to the new DNN. KMS there are in fact three different official height systems. DNN. GM91 covering Jutland, and DNN. GI44 covering Sealand and Funen. The third system still in use is the oldest system KN44, covering Copenhagen and some of the municipalities around.

Changing of Coordinate system is difficult. A lot of discussions and decisions has to be made. Many users good advice has to be taken into account. The amount of data is enormous, but the problems is still to be handled. The users we deal with in coordinate discussions and problems are almost everybody professionels, e.g. surveyors, map people. The ciffers tells us about what kind of coordinate system we are dealing with, and the data we have to change is digital data.

The problems in handling change af Height System is much more complicated. Most of the user are in a way amateurs, e.g. sewerage people, road people. A heights are not changed in a way, that one could see the differences between a height in the old and the new system. A lot of data is still analog data, and the heights here are mostly given without any label that tells the user wich system the height is given in. Every mistake cost a lot of money.

The best answer to give about how and when to change is information. From now on, an untill the day a change is done in 2002 - 2004 informations had to be spred around to all users, to get them prepared to the change, so they can prepare their own data and their users to the new DNN.

The rest is just a tecnique-question. Recalculate the part of the contry in DNN KMS90 where new levelling has been performed, and transform the rest of the contry in to the new DNN.

About 26.000 benchmarks including the precise levelling line is relevelled since 1992. This year approx. 5.000 benchmarks are relevelled or will be levelled for the first time.

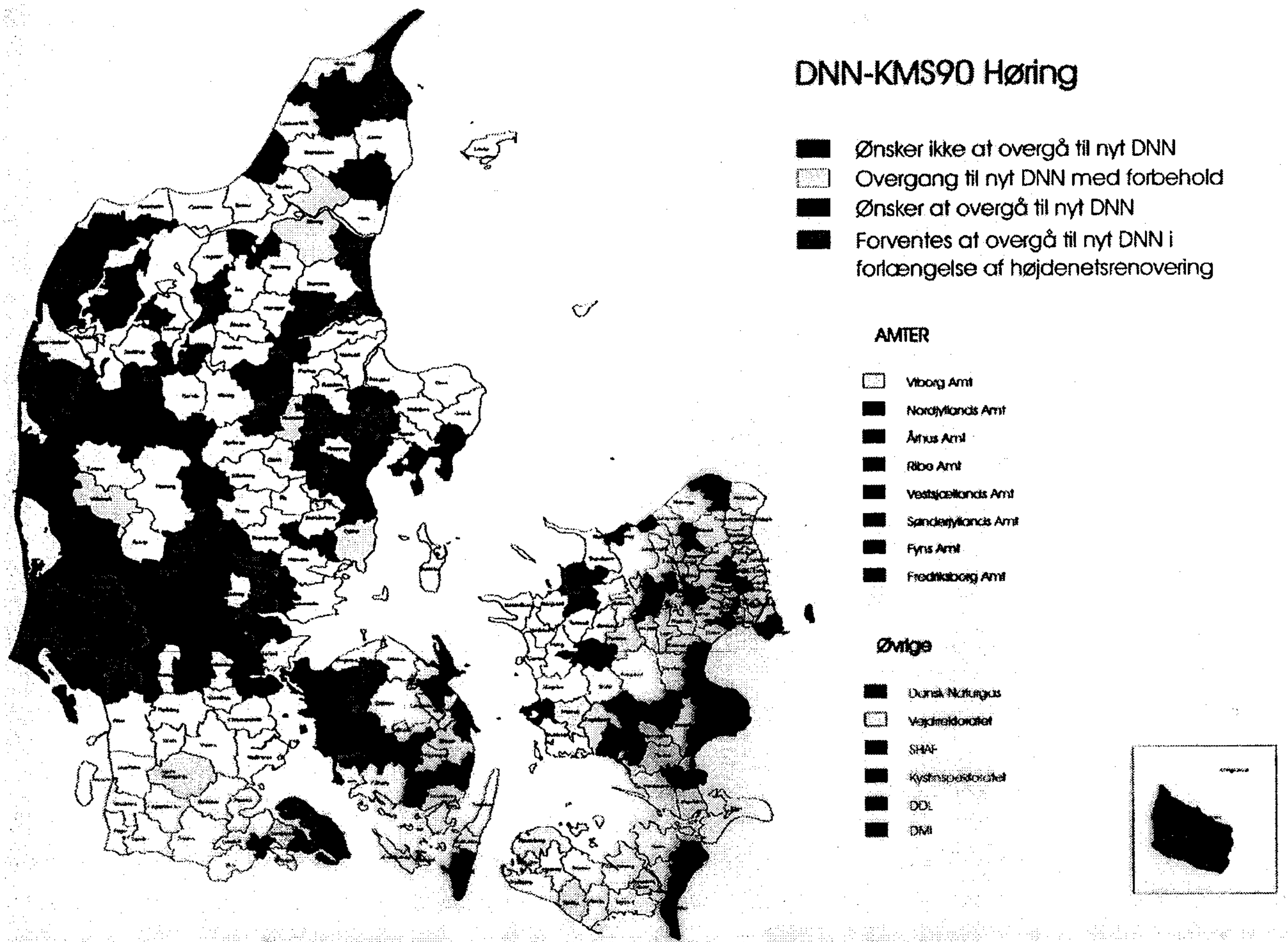


Figure 2: Answers from the major users about changing to a new height system.

Best use of different altimetric determination techniques

Prof. Michel Kasser

ESGT, 1 Bd. Pythagore, F-72 000 Le Mans, France

Fax : +33 2 4343 3102, e-mail : kasser@esgt.cnam.fr

Abstract

The surveyor, thanks to the rapid evolutions of the available equipments, has today a wide range of possibilities opened to him when he has to perform altimetric determinations. The present paper presents some considerations regarding the optimisation in different situations.

Resumé

EMPLOI OPTIMAL DES DIFFERENTES TECHNIQUES DE MESURES ALTIMETRIQUES

Le géomètre dispose actuellement d'une grande variété de procédés de mesure des altitudes et des dénivelées. Le présent article présente une analyse comparative des solutions qui apparaîssent les plus adaptées pour quelques cas courants.

1. A short review of available methodologies for altimetric determination

The different techniques for altimetric determination are well-known. For each of them we shall recall their specific advantages and drawbacks.

A/ Direct (or Geometric, or Geodetic) Levelling.

Direct levelling is performed with a level and one or two graduated rods. The various errors are described in many papers, e. g. see Kasser & Becker, 1999, in this seminar.

- The level may be opto-mechanical or digital, which implies different levels of security regarding possible blunders, and also different levels of precision. The precision may range from 0.3 (in exceptional conditions, with very specific instruments and field procedures) to 3 $mm.km^{-1/2}$ and more.

- The equipment has to be used at least by 1 observer + 1 helper for the rod. For maximal precision it requires 1 observer and 2 helpers for the staffs. When the team works along roads, it is often mandatory to have one extra worker to protect from the traffic. And the equipment may be mounted on vehicles to improve the efficiency (motorised levelling). Thus the team varies from 2 to 4 people.
- The daily production depends strongly on the equipment and the composition of the team, from 4 km/day to more than 30 km/day.

B/ Indirect (or Trigonometric) Levelling

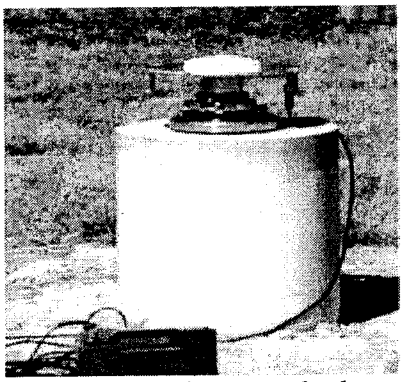
It relies upon the use of theodolites and EDM, in order to measure the zenith angle and the slope distance from one station to another. This methodology is generally much faster than direct levelling and of lesser accuracy due to refraction effects. An exception is the trigonometric levelling using simultaneous reciprocal measurements. This method can be motorised and has been widely developed and used at IGN-France since 1982 for the national levelling network (NGF). Its main features include :

- the possibility to have a large variation of production cost between low and high accuracy measures. The specification of maximum sight line length has a very important impact on the accuracy and on the daily production.
- a very limited reduction of production in mountainous areas.

The use of a tacheometer allows for rapid levelling operations with a limited accuracy if the employed ranges are long. But if the tacheometer includes a reflectorless EDM, this will provide a very convenient situation for a 0.5 to 1 cm accuracy height determination of natural topographic details close to the station.

C/ Use of GPS

GPS may be used for heighting. Its main features for such operations are :



- The benchmarks do not have to be along roads, but require an open sky above them, which is not suitable in dense urban areas.
- The error determination is comparably large and depends from the duration of the measurements, hardly better than 1 cm and generally close to 2 cm rms (one should not assimilate the internal consistency provided by computations with the accuracy), but the dependence with distance between stations is very low.
- An excellent knowledge is required of the Zero-Altitude Surface ZAS (close to the geoid and often wrongly presented as the same thing), as GPS provides only geometric observations, and height is a geopotential information. Only in a limited number of countries (among which most of European countries) is this information available with a precision comparable with GPS vertical component's one for 2 hours long sessions.

If the ZAS is not available, the surveyor will have the possibility to use GPS on a limited zone by measuring the discrepancy between the official altitude and the ellipsoidal height. For that he will get GPS measurements over a set of benchmarks from the national network, with a density as homogeneous as possible in the zone (typically 1 benchmark every 3 / 4 km may be correct if the area is not too mountainous ; if the area is mountainous, the precision requirement will probably be lower so that such a density may also be acceptable). If the discrepancy has only a variation of a few cm, a simple mathematical interpolation model between the benchmarks will provide the necessary correction, with an accuracy compatible with the 2 cm rms of the GPS vertical component.

The use of GPS for topographic applications is now sometimes proposed in real-time differential configuration, which means a more expensive equipment, but no post processing work. The main feature of this configuration will be the possibility to have a correct radio-link (emission authorizations, topography allowing a correct reception far from the emitting station). But it must be taken into consideration that post-processing GPS data allows sometimes to benefit a posteriori from data that in real-time did not work properly (ambiguity resolution after an interruption of reception), which means that real-time applications must be used only when it is requested, and sometimes may not be the best choice.

2. Typical height determination situations for surveyors.

Almost in all cases, high precision altimetric operations are requested as soon as, at least potentially in some part of the area, water has to flow driven by gravity only (e. g. sewerage, irrigation, drainage). Moreover, all national levelling networks have been set up for these reasons too.

We shall select typical works where surveyors are requested to perform levelling production.

A/ Fundamental Levelling of the National Network

Although such an activity is generally done directly by a national office, it may be in some countries at least partially observed under the control of this office, and this highly specialized activity is interesting to analyze. The goal is to provide benchmarks everywhere in the country, with a variation of density for benchmarks close to the population density, a millimetric local precision and a long range error figure as low as possible. This network must be observed at the lowest cost (compatible with this precision) possible, and regularly checked because of benchmarks destruction. The information about altitudes must be widely accessible at the lowest cost possible, every surveyor being encouraged to use this unique national height system so as to maximize national economy and synergy between various public and private surveying operations.

B/ Urban Densification Network

The goal is to provide levelling over a large number of marks, some of them being often natural ones (sewer plates, sidewalk borders, etc...), the other ones being benchmarks with special attention paid to their conservation. The applications are mostly related to water driven by gravity (sewerage systems for example). In most cases, the requested accuracy is high (1 mm to 5 mm relatively to the national levelling network). The client is the technical service of the town, and generally he will look much more at the density, the cost and the conservation rather than the precision.

C/ Semi-Urban Network.

Such networks will be requested for the preparation of new works, town housing developments, implantation of a new plant, extension of sewerage network, setting up new benchmarks for a new road, highway, or fast train (TGV) line, etc... The required accuracy will be of the same type (0.5 to 1 cm relatively to the national network), but the density of the benchmarks will be low, using classical benchmarks.

D/ Rural Height Determinations

They may be requested because the national network is not dense enough, if some new water organisation is planned (e. g. in flat areas, for drainage, in villages for water supplies, etc...). The density will be low, but the references will be perhaps very far from the site.

E/ Stability Monitoring

In order to check the movements or deformations of a bridge, a dam, a high building, or for common buildings during an underground tunnel boring, the main point will be the highest accuracy possible, with local references established only for these works, possibly with no link to the national network.

F/ Control and Real Time Guidance of Construction Machines

This goal appears more and more important for future productivity gains in civil engineering, and especially for the construction of roads, highways or train lines. There are many possible specifications of precision. The base layer thickness for roads should be monitored within 5 cm, and the last layers, that are formed with quite expensive materials, should have a thickness control to within 5 mm. Increasingly it is requested that any geometric control be permanent, without any interruption for setting up the instruments elsewhere in a new section, and be perfectly reliable whatever the profiles to achieve.

3. What technique is optimal today for these tasks?

For the **case A,** a large part (if not all) of the network should be observed with motorized levelling (cf articles from J.-M. Becker & M. Kasser) or trigonometric motorized levelling for sections in mountainous areas. But the question arises about the possibility to use GPS in parts. One must remember that the various "orders" for levelling are due to the enormous difficulties that geodesists experienced in the past with the least square adjustments of even modest systems of equations. The "first order" goal was to provide the national reference system with a density acceptable for letting the further densification in user-oriented benchmarks not too demanding in terms of observations and computations. The first order was up to now a technical necessity, but its benchmarks were not particularly valuable for the normal users. In some countries, these benchmarks may even be quite difficult to exploit : in France up to 20 years ago, most of them were along railways lines, and thus were quite dangerous to use at the era of the TGV. If there exists in the country a good geoidal computation providing a centimetric or sub-centimetric ZAS, we should now consider that the first order notion be replaced by an equivalent notion of reference national height network based on stations observed with GPS and the highest precision methodology possible, of course with ZAS corrections, but these stations being regularly spaced without any terrestrial observations between them. The mean distance between them could be from 50 to 100 km, their global precision being around 2 cm (with a much better repeatability, around 3 to 5 mm, but who cares really about repeatability ?...). This would provide a zero surface much more horizontal than commonly achieved with classical methods, and thus very low bias, at the cost of a higher standard deviation. But the general goals of the national network would be fulfilled at a much better cost than today.

For the **case B**, GPS will not be profitable : too many situations exist where the sky is not fully visible (close to buildings, trees, etc...), and too many benchmarks impossible to pick up directly with the antenna, so that an auxiliary tacheometer will be requested, limiting the benefit of the GPS advantages. And the real-time differential equipments will generally not work properly between the buildings, with their shadow zones. Our opinion is that trigonometric levelling with a tacheometer using a reflectorless EDM will be the best device, as :

- it allows to measure natural objects (sewer plates, marks on concrete borders, etc...) which is often required, if necessary with only 1 people,

- the accuracy obtained will be acceptable,
- the cost of the equipment is compatible with the economic activity of surveyors, tacheometers being the everyday tools of most of them.
- The use of a very high tripod (> 2.2 m for example) or of mural benchmarks set up very high on the walls is a very useful feature, due to the difficulty to get the optical axis unobstructed by passing-by people, trucks or cars.

Another solution would be the use of a digital level with one cheap fibreglass rod (invar rods are much more expensive), but this will prove less efficient if the density of points to survey is high.

For the **case C,** considering the low density requested, we may consider the use of digital levels because of their low cost, or the use of high precision tacheometers with reciprocal simultaneous angle measurements if the equipment is available. The latter would be preferable if the area is large (or very long), and/or with difficulties of communications (for example for a new highway where there are no roads to go from a station to the next one).

For the **case D,** the GPS will generally be the best economical solution, as soon as the work to be performed is not too small an area. Of course the use of real-time differential GPS may be considered if the topography allows for it : it will provide a better security for the quality of satellite measurements and the integrity of the collected data will be tested before leaving the zone. Thus it will be more interesting in situations where the cost of a remeasurement due to a lack of data integrity would be high.

For the **case E,** the use of optico-mechanical levels should probably be preferred for their unsurpassed precision. And as a complement we may note that for stability controls, digital levels and GPS receivers may be used as automatic continuous monitoring devices :

- For digital levels, the required length of rod may be fixed, for example to a building, and monitored automatically by the digital level controlled by a PC. Multiple targets may be surveyed if the digital level is motorised (one command for the direction, one command for the focus), and the accuracy of such measurement reaches easily the 0.1 mm level, even for distances ranging beyond 20 metres.
- For GPS, the requested receiver will have at least a single frequency capability, but of course phase measurement and if possible a large internal memory. Such an equipment may then be permanently installed on a given device, with a reference station not too far away (e. g. less than 1 km when monitoring a bridge), a power supply and if necessary a data link. Considering the possibility to filter the results, even vertical movements as small as 2 to 5 mm may be detected over periods of several days.
- For the **case F,** three methods may be considered : GPS, laser equipments and automatic (unmanned) tacheometers. All of these have been tested, but clearly the "pros and cons" are not the same for each of them. For example :
- GPS, in real-time differential mode with multiple antennas on the machine and its blade, may provide an excellent permanent control as long as there is no problems of "shadow" zones where the satellites cannot be received (high trees, high buildings, bridges or tunnel sections). But generally its accuracy is not sufficient for the last layers, as it cannot guarantee better than 1 cm (and in good situations !...), and up to now the cost of the equipment is high. But it will be perfectly compatible with even very complicated profiles. The CIRC European Project has provided quite useful data about this methodology.

- Motorized automatic tacheometers provide a much better precision, and may achieve millimetre accuracy, even in zones with "shadows"where GPS could not be used. But new stations have to be set up every 50 to 200 m (depending upon the topography, as from the stations nothing must limit the sight on the machine), and the continuity of the work requires at least two fully operational equipments. But the cost of the equipment is probably lower than for the GPS, and it is much more versatile and usable on many different situations, not only in guidance of construction machines.
- Laser equipments also allow to achieve a millimetre accuracy, and their ease of setting up is quite appreciated ("2-slopes" configuration, an improper terminology but an efficient technique), and their cost is low but they do not allow for complicated slope or profile variations and their range is limited, which requires the permanent management of at least two instruments (and more generally three) if the continuity of the guidance service is requested.

In any case, a careful estimation of the effects of refraction should be performed, as tacheometers and laser equipments may be sometimes used on very long ranges (more than 500 m is an achievable range for some lasers, and an automatic tacheometer may easily work much farther). Thus it must be pointed out that on such ranges, the errors induced by refraction are often larger than instrumental errors.

Methodology **Case examined**	**Direct levelling**	**Trigonometric levelling**	**GPS**
A/ National network	xx (motorized levelling)	x (mountainous areas), reciprocal simultaneous zenithal measurements	x (limited number of reference stations)
B/ Urban network	x	Xx	
C/ Semi-urban network	x	Xx	
D/ Rural network			x
E/ Stability controls	x (high precision, possibility of continuous measurements)		x (low precision, continuous measurements)
F/ Construction machine guidance	x (laser equipments)	x (automatic motorized tacheometers)	x (low precision)

4. Conclusion

Each given type of work requires a careful analysis, as usual, and a regular re-evaluation to the method that is optimal at a given date. But surveyors will have noticed that since a few years, "precise height determinations" are not always equivalent to "direct levelling". Here we have presented a few examples : the relevance of the analysis presented is probably quite dependent on the economic conditions in each country. But we consider that sometimes the GPS may be used, sometimes not. The same applies for the use of tacheometers. Thus we encourage the surveyors (i) not to overestimate the accuracy of GPS (this paper does not want to emphasize this classical question of the vertical precision of GPS, but any surveyor must be aware of the large discrepancy between the repeatability of GPS - a few mm - and its real precision - generally more than 2 cm -) and underestimate the problems posed by the different reference frames of GPS and national levelling network, and (ii) to have in mind for each work a clear and regularly updated idea about the economic and precision aspects relative to the methods available.

Acknowledgements

This paper has benefitted from many useful comments and reviews from members from the FIG WG 5.2, among which I want to thank at least Jean Rüeger, Jean-Marie Becker, Henri Duquenne, Hilmar Ingensand and Reiner Jaeger.

Some Trends in Geomatics - Multi-sensor Systems and Global Georeferencing.

K.P. Schwarz

Department of Geomatics Engineering

The University of Calgary

E-mail: Schwarz@ensu.ucalgary.ca

Recent developments in kinematic measurement technology have made it possible to conduct local surveys from moving vehicles and express the results in an accurately defined global reference systems. Such systems can be deployed everywhere on the globe without the need for identifying existing ground control or establishing new control. They can be used on airborne or vehicle platforms and provide a mobile survey tool to rapidly cover large areas including urban centers. Systems of this type have become known as mobile multi-sensor systems. It is the objective of this paper to discuss the basic concepts behind such systems, outline the underlying mathematics, review the state of the art by presenting some of the existing systems, and show their potential by looking at some new applications. The presentation is didactical in nature and will emphasize the principles. A detailed bibliography is given to assist the interested reader in finding specific applications.

Author's Note:

The paper is not included in this volume, because it has been published in similar form in:

Schwarz, K.P. (1998). Mobile Multi-Sensor Systems - Modelling and Estimation. Proc. of the Int. Symp. on **Geodesy for Geotechnical and Structural Engineering,** Eisenstadt, Austria, April 20-22, 1998, pp 347-360.

Different solutions adopted to modernise the height networks in the Baltic countries

A. Ellmann (National Land Board, Estonia)

A. Torim (National Land Board Development Centre, Estonia)

N. Abols (State Land Service, Latvia)

J. Kaminskis (State Land Service, Latvia)

E. Sleiteris (National Service of Geodesy and Cartography, Lithuania)

ABSTRACT

The history of precise levelling in the Baltic countries dates back to the second half of the 19th century. The analyses of high-precision re-measurements and concurrent studies performed by different institutions have identified the character and velocity of recent vertical crustal movements on the territory of the Baltic countries.

Considerably high values of vertical crustal movements (in different regions from -4 mm up to +2.5 mm/year) and destruction of the considerable amount of benchmarks require a continued and all-round attention to the problems of height networks.

New political status of the Baltic states, participation in international study projects (BSL and EUVN GPS campaigns), implementation of new technologies and skills have created a need as well as possibilities for a co-ordinated improvement and modernisation of the height network in the Baltic region.

INTRODUCTION

The first precise levelling network covering the whole Estonia, Latvia, Lithuania was established between 1871 and 1913 by the Russian Corps of Military Topographers. The total length of precise levelling lines was 4700 km of which 765 km was measured twice (RANDJÄRV, 1993). Due to relatively low accuracy of these works (random error of 1 km levelling ± 2.6...4.5 mm/km, systematic error ± 0.3...1.0 mm/km) the above measurements are only of historic value.

In the development and co-ordination of geodetic activities in the Baltic Sea countries an important role was played by the Baltic Geodetic Commission (BGC) that was founded in 1924 and worked actively up to 1940. The Commission included all countries around the Baltic Sea.

On the initiative of the BGC a uniform height network was started to be established in the Baltic Sea countries the first stage of which was the establishment of international high-precision levelling line aiming at connecting tide gauges. Scientific objective of the planned levelling networks was to study vertical movements of coastal areas and to determine the level surface of the Baltic Sea.

Proceeding from objectives set by BGC every country designed the location of national levelling lines, developed instructions, requirements and methods for levelling. At the same the establishment and measurements of high-precision and precise levelling networks were started. The levelling networks are characterised in Table 1.

Table 1. *Main characteristics of the levellings in the Baltics in 1927-1943 (RANDJÄRV, 1993)*

Location of the Network	**Years of levelling**	**Length of line L km**	**Average distance between benchs/marks km**	**Accuracy estimates**		
				Fh_{add} mm/km	**accidental error mm/km**	**systematic error mm/km**
Estonia	1933-1943	1770	1.5/10-15	$\pm1.5\sqrt{L}$	±0.32	±0.03
Latvia	1929-1939	4276	2/6-7	$\pm3.0\sqrt{L}$	±0.47	±0.12
Lithuania (except SE)	1930-1939	1700	2/10-15	$\pm1.0\sqrt{L}$	±0.35	±0.10
SE Lithuania and NW Belorus	1927-1937	1215	2-3/8-10	$\pm1.5\sqrt{L}$	±0.31	±0.05

Notice: *In Table fh_{add} is admissible divergence of relative altitude, received from back and forth traverse.*

The obtained accuracy of high-precision levellings performed in 1927-1943 in Baltic States is remarkably good and the established levelling lines have, with some minor changes, served as basis for following levellings as well as for improvement projects of modern levelling network.

HIGH PRECISION LEVELLING IN 1945-1991

In the period of 1945-1991, the height networks of Estonia, Latvia and Lithuania, which were incorporated into the Soviet Union, were similarly to the height networks of East-European countries connected to the Soviet Union's height network. Results of high-precision levelling were adjusted as a united system of levelling lines known also under the name of UPLN (United Precise Levelling Network).

Official high-precision (I order) levellings were performed by the Main Board of Geodesy and Cartography (MBGC) of the former Soviet Union, whereas the lines and benchmarks used by them in the Baltic countries (1946-1954 and 1967-1971) to a great extent coincided with the levelling lines established in 1927-1943.

Simultaneously with the works performed by the MBGC, national research and educational establishments of Estonia, Latvia and Lithuania carried in 1950-1991 on regular basis out high-precision re-measurements on levelling lines established in 1927-1943. The aim of these re-measurements was to study recent vertical crustal movements. The accuracy of levelling was the same as the accuracy of the official high-precision (I order) levelling (the average random error of 1 km levelling ±0.5 mm/km). The results of these measurements have been summarised by J.Randjärv in his paper (RANDJÄRV, 1993).

The analyses of above high-precision re-measurements and concurrent studies performed by different institutions have identified the character and velocity (from -4 up to+2.5 mm/y) of recent vertical crustal movements on the territory of the Baltic countries. According to studies the most intensive land uplift area is North-Western Estonia and vast sinking area including SE Estonia, most of Latvia and Lithuania.

During this period normal heights started to be used, similar to East and Central Europe Kronstadt zero was used as initial benchmark and the levelling results were given in the Baltic height system of 1977.

INTERNATIONAL CO-OPERATION

Since re-gaining the independence in 1991 Estonia, Latvia and Lithuania have been participating in the realisation of several international co-operation projects.

So far the most important project has been the implementation of new European Terrestrial Reference System.

In order to support the establishment of new geodetic reference system in the Baltic countries, the GPS campaign EUREF.BAL'92 was performed in the co-operation of Nordic and Baltic countries in 1992. The coordinates of 13 points on the territory of Baltic countries (MADSEN & MADSEN, 1993), were accepted by the EUREF Commission as an extension of the EUREF in 1993.

Relying on the approved values of coordinates, all Baltic States have started to implement the new European Terrestrial Reference System ETRS-89

As the ongoing land reform in the Baltic countries requires extensive land surveying, a further national densification of EUREF by using GPS technology has been necessary in all three countries. In our countries thousands of points have been monumented and measured (approx.12 000) in the past seven years.

Today the coordinate system ETRS-89 is adopted in the Baltic States as a national coordinate system. As a rule data of all topographic maps and national geodetic networks is given in the coordinate system ETRS-89.

Currently national improvement of previous results of EUREF.BAL'92 is ongoing or foreseen.

In recent years with international support 4 permanent GPS stations have become operational (Irbene, Riga, Suurupi, Vilnius), in near future some more stations are to be established. The data of these stations is used and available at international research and computing centres.

The Baltic countries have been involved also in several GPS campaigns that have addressed the problems of height systems and height networks. Here can be mentioned e.g. GPS-campaigns BSL93&97 that aimed at the determination of the level surface of the Baltic Sea and the vertical connection of tide gauges (in 1997 measurements were performed on 9 points in the Baltic States of which 4 were located in the vicinity of tide gauges), and EUVN97 (European Vertical GPS Reference Network) GPS Campaign performed simultaneously with BSL97 campaign with the aim of unifying European height networks and systems (10 points in the Baltic States, some of which were used also in BSL97 calculations).

The published results of these GPS campaigns may be necessary for developing improvement projects for Estonian, Latvian and Lithuanian vertical networks.

In 1995, the Finnish Geodetic Institute (FGI) performed gravimetric absolute measurements in the Baltic countries determining absolute values of gravity on three stations in each country (MÄKINEN, et al 1996), which made it also possible to specify the gravimetric data of Baltic countries.

Specialists from the Baltic countries could improve their knowledge by participating in the calculation of the Nordic-Baltic geoid models in which also gravimetric data from our countries were used. One of the most recent collaboration was the calculation of Baltic geoid model within the framework of successfully completed Danish-Baltic co-operation project.

Levelling data is being digitised for the purpose of analysis and further processing.

Considerably high values of vertical crustal movements and destruction of the considerable amount of benchmarks require a continued and all-round attention to the problems of the height networks.

New political status of the Baltic states, participation in international study projects, implementation of new technologies and skills have created a need as well as possibilities for a co-ordinated improvement and modernisation of the height network in the Baltic region.

At present specialists from the Baltic countries hold joint meetings trying to solve the problems concerning height networks. In this connection it is necessary to develop, review and co-ordinate the design and basic requirements of national height networks, to specify the connection to height networks of neighbouring countries, the methods of levelling and a preliminary time-schedule.

Below are presented views and ideas of specialists from all three Baltic countries about the improvement of national height networks.

ESTONIA

High-precision levelling of previous years in Estonia

Estonian Height Network (EHN) is consisting of the national system of levelling lines (I, II, III order) and benchmarks, which are based upon levelling lines established during the years 1933-1943. Before the World War II the zero point of Estonian levelling network was the mean sea level height of twelve years (1923-1934) from Tallinn tide gauge station and after the war it was replaced with the so-called Kronstadt zero. The present heights of EHN benchmarks are expressed in the Baltic Height System of 1977.

The first levelling network as a system of high-precision and precision levelling lines covering the whole Estonia was established within the framework of the Baltic Geodetic Commission in 1933-1943 (TORIM, 1993). As much as 1151 benchmarks were monumented in the course of establishing the all-Estonian levelling network. All knot points and end points of the levelling lines were supplied with fundamental benchmarks (in total 23). The established levelling lines with slight modifications have served as basis for high-precision levellings of following years.

The main characteristics of the high-precision levellings in Estonia are given in Table 2.

In addition to the high-precision levellings, the precision levelling (in total 728 km) was carried out in 1936-1943 and the second order levelling (1380 km) was performed by MBGC in 1981-83.

Table 2. Main characteristics of the high-precision levellings in Estonia in 1933-1991.

Institution	Years	Levelling lines	The 1 km mean error values random mm	systematic mm
Cadastre Department	1933-1943	6 loops in mainland, in total 1800 km	± 0.32	± 0.03
MBGC	1948	The line Narva-Tallinn-Pärnu-Ikla (505 km)	± 0.5	± 0.05
Academy of Sciences	1951-1969	Mainly at the lines established in 1933-1943 (2067 km.)	± 0.48	± 0.08
MBGC	1970	The line Narva-Tallinn-Pärnu-Ikla (505 km)	± 0.53	± 0.06
Academy of Sciences	1970-1991	6 loops on the continental part and 2 on the islands, in total 2280 km	± 0.46	± 0.04

Recent vertical movement of the Estonian territory

Based on the data of repeated levelling, maps about the vertical movement of the Earth's crust by geodetic method have been compiled for Estonia, the most recent one in 1988 (VALLNER et. al., 1988). According to these researches the most intensive land uplift area is North-Western Estonia and vast sinking area SE Estonia.

The research work carried out in Estonia has shown that the gradient of the recent movement of the Earth's crust is maximal in the direction of north-west – south-east, ranging annually for up to 3.6 mm. When it takes years for closing the loops of the levelling network, then the closing error will increase because of tectonic vertical movements and the respective correction should be introduced in the levelling data. When using levellings performed in different years, the data have to be reduced to a definite moment of time (epoch) and the respective transfer corrections have to be introduced to the calculated heights. The transfer to a given epoch will take place by introducing corrections caused by relative vertical movement to measured heights.

Renovation of the Estonian levelling network

The renovation of the Estonian levelling network is carried out in connection with extensive works related to the reconstruction of national high-precision geodetic (GPS) network that was started in 1996.

The national geodetic GPS network consists of 212 points, the average distance between neighbouring points is ~15 km (RÜDJA, 1996). The network was monumented in 1996, observations were performed in 1997. In the computations also the data from permanent GPS stations of neighbouring countries is used. The results will be presented in 1999.

In order to establish an integrated geodetic network, the design of GPS network has taken into consideration the location of national levelling lines and that of points of the gravimetric remeasurement network. On the high precision levelling lines there are approximately 120 GPS points. It enables to include a number of GPS points directly into the improved vertical network if necessary.

As the first stage of the establishment of the integrated geodetic network 20-25 points of the national geodetic GPS network were connected with the high-precision levelling and gravimetric networks in 1998.

The aim of the renovation of the Estonian levelling network is the updating of the existing levelling network on the basis of the principles of establishing an integrated geodetic fundamental network. That would set up premises for the compilation of a new levelling catalogue reduced to a definite moment of time, specification of the geoid model, integration of the EHN to the unified regional Height Network and connection of different national levelling systems.

As planned the first all-Estonian system of high-precision and precision levelling lines that formed 6 loops and were established in 1933-1943 will be taken as basis in renovating the EHN. It is rational to establish the Estonian levelling network as a system of loops covering the country, with 10 loops in the mainland and 2 loops in the islands (figure 1). This will result in an integrated and dense levelling fundamental network that would cut the present large levelling loops into two by replacing existing lower order levellings with high-precision levelling and foresee the establishment of new networks in the islands of Saaremaa and Hiiumaa, where high-precision levelling was performed in 1962 and 1966 respectively.

Within the range of the Tallinn circle there will have to be established a new high-precision line, which is necessary for connecting the levelling lines heading for Tallinn knot point, and for connecting the Suurupi initial benchmark (co-located with the absolute gravity and GPS permanent Stations) and the Greater-Tallinn levelling network.

As a result of the above-mentioned the perimeters of the new high-precision levelling loops in the mainland are in the range of 213-310 km, which is in conformity with the requirements of the instruction of I, II and III order levelling (TORIM, 1994). At the same time, the demarked lines are covering the territory of Estonia quite evenly. The main line of Narva-Tapa-Tallinn-Pärnu-Ikla enables us to be linked in the research programme of the Baltic Sea level.

Investigation showed that 25-30% of the benchmarks have perished during the last 65 years. Therefore there is need to mount 250 new wall and 16 fundamental benchmarks.

Situation of the EHN (long levelling period, destruction of benchmarks), participation in international study projects and implementation of new technologies required co-ordinated improvement of the Estonian height network. The best variant for the renovation of the Estonian levelling network would be the establishment of an integrated geodetic fundamental network by a new repeated levelling of all levelling lines and the connection of selected high-precision GPS points together with the gravimetric survey of the levelling lines.

Considering the velocities of the land uplift, if possible it would be reasonable to level the whole network quickly, within 5 years at least.

The initial sea level could be calculated from the same period.

During the new repeated levelling for the purpose of determining the initial level of the EHN and participating in the continuing international projects EUVN and BSL, the main tide gauge stations situating on the coast of Estonia will have to be connected properly to the high-precision levelling benchmarks, which has been done inadequately to this day.

The intention is to establish a new modern tide gauge in the proximity of the Suurupi permanent GPS station.

LATVIA

The geodetic reference network

Reference frame for Latvian GPS network is based on the results of the EUREF.BAL`92 GPS campaign. Nowadays the Latvian GPS network consists of zero, first, second and third order networks, all together 2200 points.

The original results (MADSEN & MADSEN 1993) of the EUREF.BAL'92 GPS Campaign have not been improved in Latvia, but it can be done by taking into consideration the results of the 5 Latvian EUVN97 stations.

There are two operating permanent GPS-stations in Latvia - Riga and Irbene.

Riga station belongs to the EUREF Network of Permanent GPS Stations, besides it is co-located with Satellite Laser Ranging System, absolute gravity station, ground water registration spot. Since May 1998 the second permanent GPS-station in Latvia Irbene has been operational and it was included in EUVN97 project (ZHAGARS & KAMINSKIS, 1998).

The future levelling plans are made with the consideration of possibilities. It is planned to perform 80 km of the 1 order levelling in 1999. First and second order levelling network catalogues are already digitised in Latvia.

In 1998 new gravity reference network measurements were started in order to create and complete gravity reference network and to continue gravity measurements, verify and improve Latvia's gravity database. For that purpose the new gravimeter Scintrex CG-3 is used.

New points are selected along the roads with the distance between points less than 5 km. Trigonometric points or levelling benchmarks are going to be used in the detailed survey. For gravity surveys the three absolute gravity points measured by the Finnish Geodetic Institute (MÄKINEN et al., 1996) will serve as initial points.

In the future the improvement of national gravity reference network can be obtained from absolute gravity observations, because such observations during last years have become more accurate and less costly. From the possible future participation in Unified European Gravity Reference Network (UEGN99) project, Latvia obtains homogeneous station gravity values allowing inclusion of Latvia in the common European framework. Geodynamic changes of network geometry influence surface gravity values, which makes it important to continue with ongoing improvements and research in reference networks.

Geoid model

As an indicator for testing consistency of local or global geodetic reference networks an appropriate geoid model could be used. The geoid model can be used for testing the Vertical Geodetic Network and for determining the quality of national GPS zero, first and second order network in connection with the Latvian geometrical levelling network.

As a starting point of using geoid for practical application the geoid model NKG89 should be mentioned. Later the Baltic`96 geoid has become available, too.

In 1996 at the National Survey and Cadastre of Denmark (KMS) the Latvian Gravimetric Geoid (LGG96) as a part of Nordic Geoid (NKG96) (FORSBERG et al., 1997) was computed by using GRAVSOFT package. The geoid model was evaluated by using different gravity data sets (including gravity data on the Baltic Sea). In calculations the data of the 12 000 gravity points originally from the 1960ies were used. The gravity data corresponding to the system of International Gravity Standardization Net 1971 was digitised for that purpose.

For the fitting of the geoid the GPS zero, first and second order sites together with first and second order levelling data are used in Latvia, which makes the base for a 4-parameter empirical datum fit to GPS/levelling stations.

The regional and irregularly distributed differences between the European Gravimetric Geoid solution EGG97 and the local geoid were determined (indicated on the figure 2) in the test.

Also the local geoid model was tested by using values of 32 zero and first order GPS points and standard deviation of 8 cm for LGG96 after fitting of the geoid was obtained (KAMINSKIS & FORSBERG, 1996). Levelling data at the reference GPS sites is rather old, because the last levelling of those sites in Latvia was carried out in 1950-ies.

Calculated reliable geoid model could be used with modern and efficient survey techniques and might consolidate the national height system.

For the maintenance of the Latvian geodetic reference system and needs of surveying the local geoid will be authorised by national geodetic community for the official use.

LITHUANIA

GPS activities and geoid

The establishment of the Lithuanian National GPS Network as the basis for all geodetic, cartographic and cadastre works was started in 1992. The network consists of zero, first and second order points, all together 1078 points.

To ensure necessary accuracy of the GPS networks points coordinates and reliability of the connection between coordinate systems and to estimate regional and local geodynamical processes, it is necessary to supplement the existing GPS network with permanent GPS stations and to perform repeated measurements of zero and first order network. Geodynamical researches require the same terms.

Taking into account the accuracy of the new Lithuanian GPS network and possible rates of the horizontal and vertical earth crust movements and their influence on the geodetic network distortion, it is necessary to repeat Lithuanian first order GPS network measurements after 10 years, i.e. in 2002-2003.

It is necessary to include the established permanent GPS stations in Lithuania in international network. That would simplify Lithuania's participation in different works which are carried out within the framework of international projects and which are important not only for Lithuania or the Baltic Sea region but also for the whole Europe in solving such problems as acceptance of coordinate systems, geoid determination, establishing of up-to-date geodetic networks, geodynamic researches and navigation.

The geoid model for Lithuanian territory has been calculated by the Geoid Determination Working Group of the NKG three times, most recently in 1998. Gravity data of the terrain, Baltic Sea and Kursiu lagoon, totally about 12 000 points (1 point per 5 sq. km, measured in 1954-1974) and the digital terrain model with a grid of 1 km has been used. Also some data from surrounding areas were involved, totally about 3000 points. The Lithuanian GPS Network ensures the accuracy of derived ellipsoidal heights. 20 mm accuracy calculated from the differences of the double GPS measurements was determined.

The GPS network served as basis for positioning the benchmarks of the first and second order levelling lines. Totally 120 benchmarks were coordinated, from which 70 most reliable ones were chosen for the geoid fitting. The GRAVSOFT software package was used for calculations. The fit of Lithuanian geoid model at the 3 cm level was obtained (KAZAKEVICIUS & PARSELIUNAS, 1998).

Establishment of the Lithuanian National Vertical Network

The main goal for the establishment of the vertical geodetic network is to determine geopotential heights and to form levelling base for the Lithuanian territory. The data of geodetic, satellite geodesy and gravity measurements are used for height determination. The levelling network is being established in two accuracy orders.

The Lithuanian national vertical network is being established according to the following principles:

- network meets national and international levelling network requirements;
- precise levelling and gravity data would be used for geopotential height determination;
- ellipsoid heights will be received by satellite geodetic methods;
- gravity will be determined by gravity measurements;
- data of vertical network will be used to determine quasi-geoid of Lithuanian territory and to specify European continental geoid;
- on the basis of the established vertical network, a quasi-geoid model and results of satellite geodesy measurements the heights of needed accuracy for geodetic, cartographic and others engineering works can be calculated;
- networks must meet requirements of geodynamical researches;
- network must be co-ordinated with lito-monitoring programme of Lithuanian territory;
- Lithuanian vertical geodetic network will be included in the Baltic and European united vertical network through connection with the points of international projects EUVN and BSL existing in Lithuania;
- existing geodetic marks will be used where possible;
- the mean square error of the 1 km line measured height difference of the vertical network shall not be bigger than 0.5 mm in the first order and not bigger than 1.0 mm in the second order network.

The first order network scheme (Fig.3) is prepared to establish Lithuanian Vertical Reference Network. The new network consists of five closed loops. The network perimeter is about 1800 km. In designing the scheme it was aimed to distribute network points equally over the whole territory with using former first order and part of the second order levelling lines. The Lithuanian vertical network connection with vertical networks of the neighbouring countries – Latvia, Belorus, Poland and Russia is provided in the scheme too. The total length of the second order vertical network lines is about 4500 km.

It is intended to carry out Lithuanian national vertical network measurements in five years by doing works of first and second orders simultaneously, for that purpose it is necessary to implement the digital levelling technology and use invar precise rods.

REFERENCES

Forsberg, R., J. Kaminskis, D. Solheim (1997): Geoid of the Nordic and Baltic region from gravimetry and satellite altimetry. In: Gravity, Geoid and Marine Geodesy, IAG Symp., Vol. 117, 540-547, Springer, Berlin, Heidelberg, New York.

Kaminskis, J., R. Forsberg (1996): The geoid determination in Latvia from gravity and satellite altimetry. In: Reports of FGI, No. 96:2, 149-155, Helsinki.

Kazakevicius, S., Parseliunas, E. (1998): Specifying the parameters of Lithuanian territory quasigeoid in 1998.

Madsen F., Madsen F.B. (1993): A new GPS-network in the Baltic Countries. Gubler E. and Hornik H. (Eds): Report on the Symposium of the IAG Subcommission for the European Reference Frame (EUREF) held in Budapest 17-19 May 1993, pp. 83-91, Veröffenlichungen der Bayerischen Kommission für die Internationale Erdmessung, Heft Nr. 53, München 1993.

Mäkinen, J., Petroskevicius, P., Sildvee, H., Kaminskis, J. (1996): Absolute-gravity measurements in Lithuania, Estonia and Latvia in 1994-1995. European Geophysical Society, Annals Geophysicae, Supplement I to Volume 14. 1996.

Randjärv, J. (1993): Vertical movements of the Earth's crust in the Baltic region. Reports of the Finnish Geodetic Institute 93:2, Helsinki, 1993.

Rüdja, A.(1996): Project of national geodetic network (in Estonian). Planserk AS. Tallinn 1996.

Torim, A.(1993): Levelling performed in Estonia (1868-1943). Geodeet no. 3 (27), Tartu, 1993.

Torim, A. (1994): Instruction of the I, II and III order levelling. National Land Board of Estonia. Tõravere, 1994.

Vallner, L., Sildvee, H., Torim, A. (1988): Recent crustal movements in Estonia. Journal of geodynamics 9, 1988.

Vallner, L., Zelnin, G. (1975): The new isobasis map of the territory of Estonia. The Estonian Academy of Sciences, Recent movement of the Earth's crust in the Baltic region. Tartu, 1975 (in Russian).

Zhagars Y., Kaminskis J. (1998): A new geodynamic site in Latvia - Irbene LV-04. Latvian Journal of Physics and Technical Sciences, vol. 6, 1998 (ISSN 0868-8257).

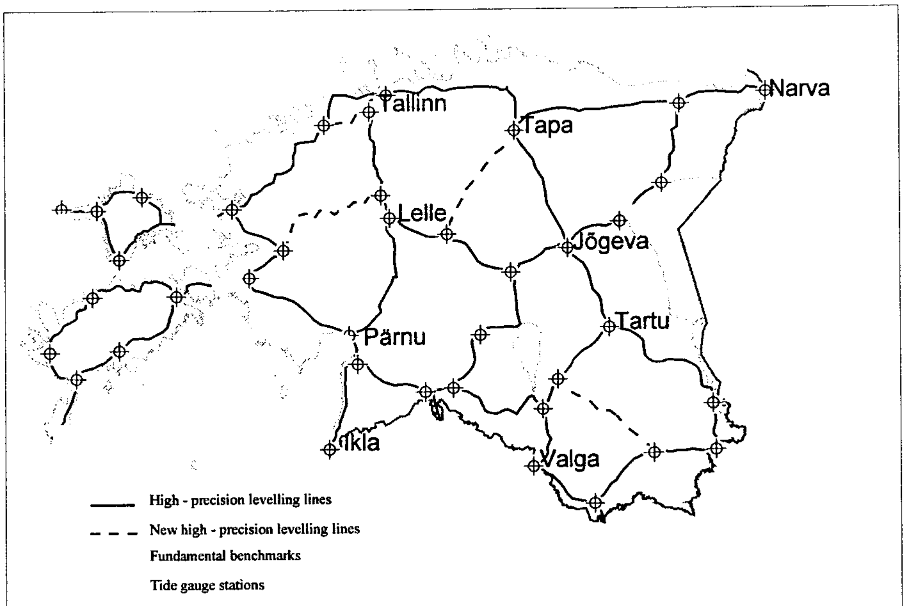


Figure 1. The Estonian high precision levelling network

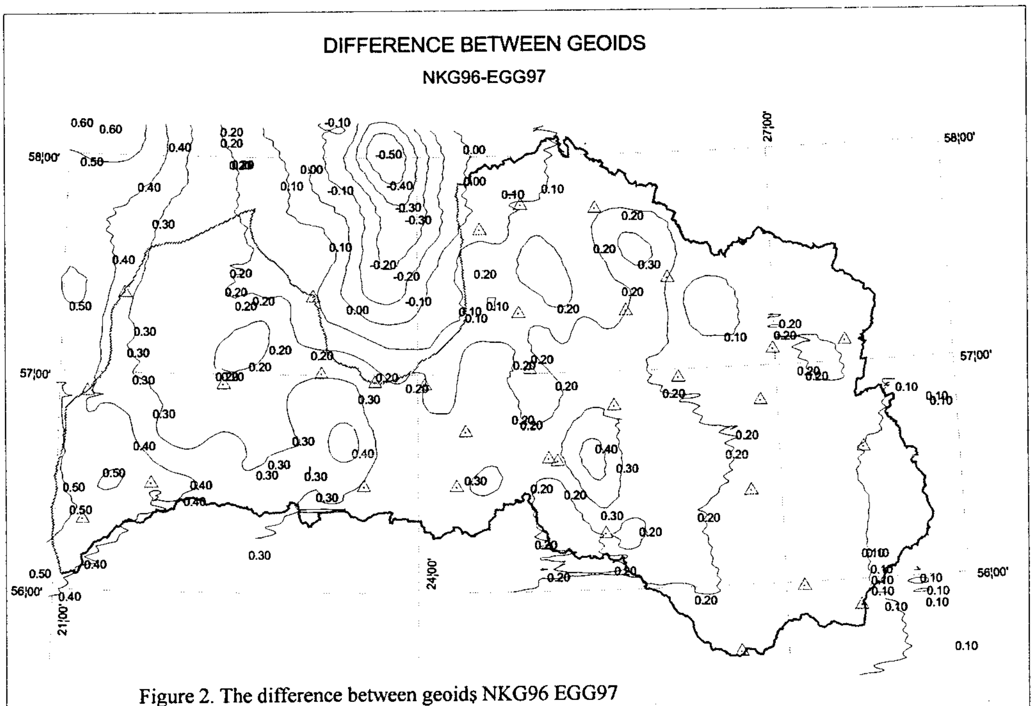


Figure 2. The difference between geoids NKG96 EGG97

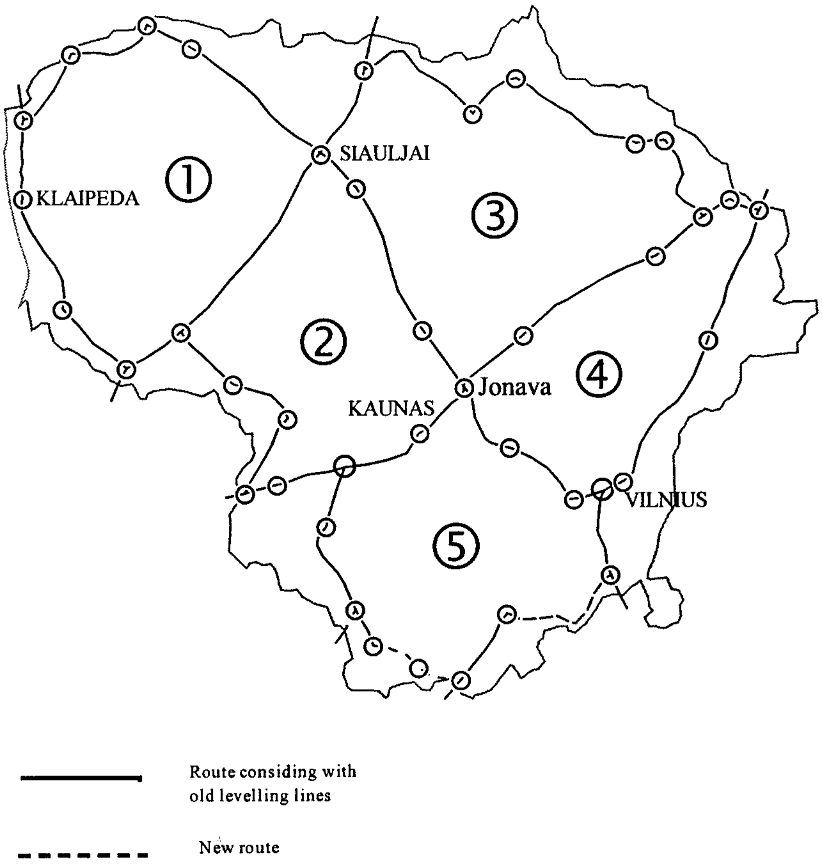


Figure 3. Lithuanian National Vertical First Order Network

High precision GPS presents challenges in the realisation of Reference Systems

Poul Nørgård

M. Sc. Geodesy & surveying

Road Directorate

Ministry of Transport - Denmark

1.0 Demands on a high precision reference system

When the compass was recognised as a navigational aid at sea, all nautical charts were re-adjusted to the Mercator projection, so that they were true to the compass. Today, not only nautical charts, but all surveys using GPS must be expressed in one and the same global projection.

This change introduces a number of demands for new projections, surveys and tools to express and make conversions between various projections and datums. This memorandum treats the considerable challenge posed by this task in connection with precision surveying.

1.1. EUREF89 as defined in Europe

During the past years a thorough survey using GPS has been done, making it possible to implement WGS84 as a projection usable in most European countries. EUREF89, which will form the background for national realisation of these datums, fulfils on the whole all the requirements for a European datum. Both scale and orientation are carried out in such a way as to considerably ease the practical work of surveying, and give better possibilities for precision surveying.

1.2 A really good prediction in the field

Like all other datums, EUREF89 is no better than the prediction expressing it. The requirements for a good prediction are:

- Enough points so that GPS/RTK surveys can be carried out directly from the main control points/bench marks.
- The physical stability and survey accuracy of the points must be superior to surveying of detail points /minute surveying with GPS, e.g. 0.5-2 cm + 1 ppm.
- The points must be suitable for satellite surveying and be easily accessible.

1.3 Well defined 3D connection to local datums

If the above requirements are fulfilled, surveying with GPS is considerably easier, but the fulfilment of one important requirement still remains, namely the linking to the existing datums, which normally are terrestrially determined. This link can be made in many ways. In Denmark, the National Survey and Cadastre publishes very high quality system-shifts. On the horizontal plane, the shifts are partly based on mathematically determined shifts, partly on classic transformations and polynomials to the local datum System 34.The shift between EUREF89 and System 34 can be made with approx. 2 centimetre's absolute accuracy, and considerably better relative accuracy. The vertical/height shift is expressed in a geoid model, with an MSE/standard deviation at level of 1-3 centimetres. This means that there are extremely good and well-described tools for system shifts, but nevertheless, as will be seen in the following, other means must be used in precision surveying.

1.4 A good description of all systems and datum shift(s)

In order to work with good local accuracy – and for in order to use GPS at all effectively, it is important that good written documentation is available for datum and the necessary transitions/links.

The availability of this documentation is also important, since there are, unfortunately, a number of cases where the use of out-of-date documentation and inexact solutions has caused serious problems.

2.0 Local - contra global accuracy

2.1 All construction and cadastral work needs local accuracy

Construction work often requires great local accuracy. For example in laying asphalt, a centimetre's inaccuracy over some meters is a serious problem, but completely unimportant over longer distances

Cadastral surveys in Denmark are to a great degree based on local accuracy, expressed as relative accuracy.

2.2. GPS are based on global accuracy

GPS is, per definition, a global survey method, but in practice, far the most important need is for relative accuracy over short distances. Understanding of these differences and the need for local adaptation should be regarded as the main problem for a great number of surveying tasks with high accuracy.

2.3 Orientation problems with GPS

2.3.1 Local orientation on processed vectors

When networks are surveyed with GPS, the adaptation to local networks can take place either by fixing known points, or by calculating minimum constrained networks, which subsequent are adjusted with a 3D transformation.

If optimal accuracy is desired, the orientation of GPS surveys can, however, cause problems. In calculating the vectors, the orientation element is often determined in different ways. This uncertainty on orientation is normally constant, but will, in fixed networks appear as a tension, while the source of error can normally be eliminated by adjusting the surveys as minimum constrained networks, followed by a transformation.

In RTK-surveying, it is, however, more difficult to remove the contribution from an orientation error. If surveying is done over long distances, 10 - 15 kilometres, it is difficult to control this contributed error, especially in the vertical/heights. To obtain maximum accuracy with

RTK, it is therefor important that all surveys are adjusted with local transformations where the RTK-data are surveyed inside of a period of 10-15 min.

3.0 Tools and methods datum and datum fitting

Detection of basic accuracy

In order to know how much accuracy can be obtained, it is important to know the GPS receiver's capacity under practical conditions and without any influence from possible contributed error from transformations. Our GPS equipment is therefore tested both statically and dynamically.

The static test takes place on a test-course for EDM-instruments. The test course consists of 7 pillars in a row, with all the points at the same height. A long series of surveys here has given us good knowledge of the capacity of the instruments under static conditions.

A series of our surveys indicated that poorer accuracy was obtained under dynamic conditions, but it was difficult to determine how much less accurate. We therefore had a carousel made, where we can test the various receivers at different speeds.

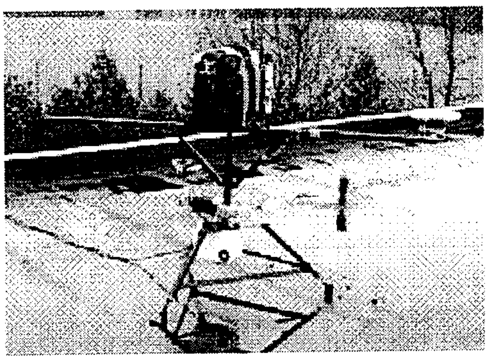


Fig. 1

Carousel for testing dynamic accuracy

Experience from these two types of test forms the basis for our differentiation between contributed error from GPS receivers and various transformations.

Generally transformation parameters can solve 95 % of all shifts, down to an accuracy of 2- 5 cm.

As described above, excellent overall shifts can be made between different datums, and it is estimated that these tools can cover at least 95% of surveying needs.

Locally/in the field 2- or 3D transformation is the solution for local accuracy down to mm.

Road Directorate experience has, however, shown that the accuracy of RTK surveys can be markedly improved by carrying out adjustments with local transformations. In the following, examples are given of how the Road Directorate suggests solving the problems in practice.

3.0 Examples of solutions

3.1 System DKS for Øresund

The Road Directorate was given the task of carrying out a projection for the construction of the fixed link between Denmark and Sweden

3.2 The DKS system and the prediction

Different datums are used in the two countries, and both UTM and WGS84 were, at the beginning of construction, defined differently in the two countries. A local transversal Mercator projection, called DKS, was therefore constructed for the working area. The definition of normal zero was also different, and therefore a common DKS normal zero was also defined. Based on this datum and an implementation on a series of pillars in the field, overall vertical and horizontal system shifts were created.

3.2.1 The DKS file

All the many datums and their correlations were described in a binder, which was distributed to everyone working on the project. The binder contains a verbal and mathematical description of all datums; and the best possible system shifts are also thoroughly described. To ensure the proper use of the material, a floppy disc is included in the binder, expressing the descriptions as a system-shift program. All binders are numbered and are kept updated.

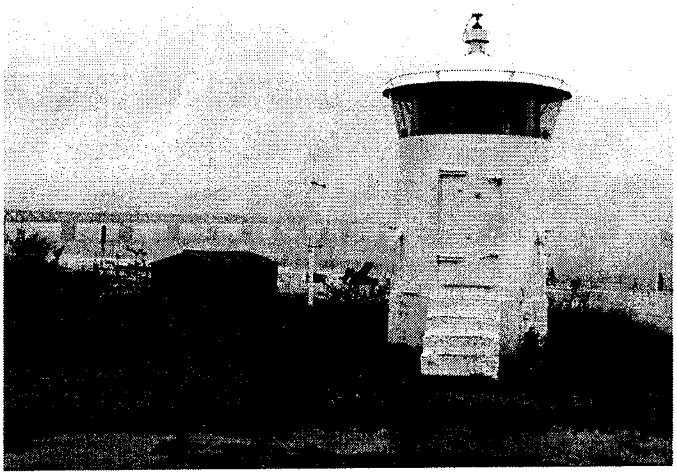


Fig. 2

RTK referencestation in Sweden. (The new bridge in the background).

3.2.2 The RTK system

Six permanent GPS/RTK reference stations are positioned at Øresund. These points act as electronic fixed/fix points and in this way supplement the pillars traditionally used to implement the DKS system.

On establishment of the RTK chain, the need for optimal orientation between the local datum and WGS84 was extremely important. A thorough survey of all main control points and the new electronic fixed points was therefore carried out. The survey was afterwards linked to EUREF89 points in Sweden and Denmark. Based on the surveys, all main control points were given new co-ordinates in EUREF89.

3.3 The Road Directorate's surveying on roads and bridges

3.3.1 New co-ordinates on all fix points for roadwork in S34

To obtain the best possible accuracy for fix points in connection with work on motorways, a comprehensive net/network is surveyed, using GPS. The vectors are adjusted as minimum constrained networks in EUREF89. Then KMS-trans is used to shift to the Danish system S34.

Finally the points are adjusted horizontally by a 2D transformation. At the moment, all heights are based on traditional levelling, but we are working on finding the best methods of combining GPS and levelling.

3.3.2 Relative GPS measurements on bridges connected/linked to old measurements by 3D transformation

The Road Directorate carries out control surveys on all larger bridges in Denmark. As a starting point, all these surveys are carried out in local co-ordinate systems. We are working on the introduction of various forms of dynamic and static control surveys, using GPS. When GPS is used, and it is not necessary to compare the surveys to older survey results, relative determinations are used.

3.4 DyRoS

Dynamic Road Surveying is the Road Directorate's new universal GPS-based survey vehicle. The concept was originally developed for levelling when renovating existing roads. The system has since been developed further, so that it can be used for surveying system lines and road technology along the roads. DyRoS is equipped with RTK/GPS for positioning, and with the help of a EDC (Electronic Digital Compass) and an inclinometer, the system lines can be surveyed eccentrically, and the oscillation of the antenna can be compensated for. The vehicle is also very well suited to the surveying of photogrammetic ground control points.

Without compensating for incline, the system lines can be levelled with an MSE of 1 centimetre, and the centre line can be determined with typically 10 centimetres while moving, and better if the

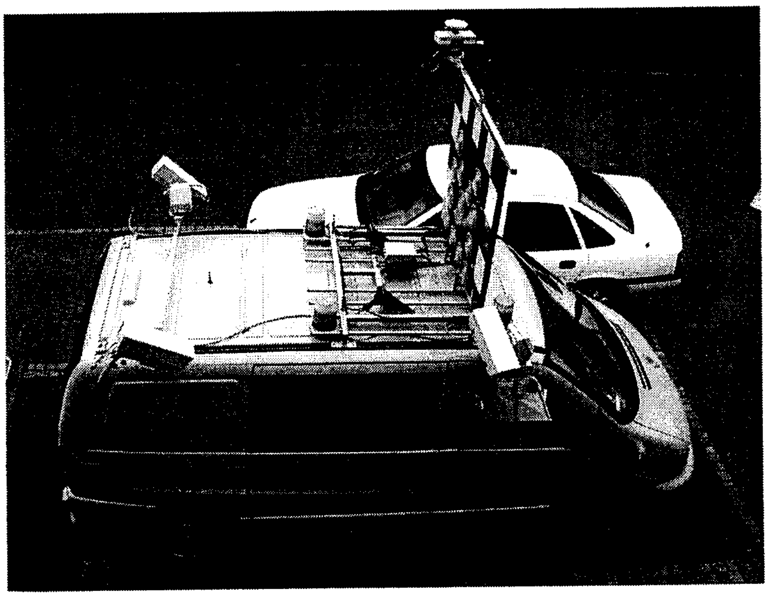


Fig. 3

The DyRoS multisurveying van, with triple video, GPS/RTK and DMC compenation

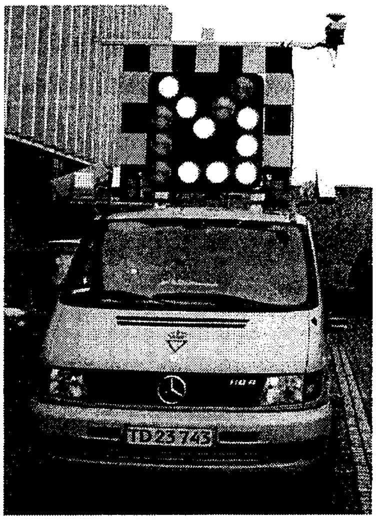
vehicle is stationary. We are currently working on documenting the accuracy of using EDC and inclinometer

3.4.1 Use of predefined transformation for system line surveying

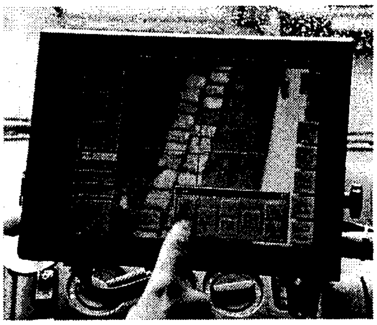
When we survey system lines, (road centre lines) 10-15 centimetre's accuracy is required, for which reason we only use overall system shifts, which can be carried out with an accuracy of a few centimetres in Denmark.

3.4.2 Local 3D transformation for system-line levelling

To fulfil the requirements for levelling of road centre lines, a different approach is made to surveying, as there are heavy demands on accuracy here. The benchmarks for the project are surveyed from the reference station. Based on these surveys, a transformation set for the current task is created. "Interpolate" is chosen as a transformation, because it makes an elastic transformation and for that reason gives the best local accuracy.



Red line shows axis for surveying of systemline



Control panel for video , GPS and

Direct access to the French digital height databank via Minitel

Michel Le Pape

Institut Géographique National, Service de Géodésie et Nivellement, 2, avenue Pasteur, 94165 Saint-Mandé Cedex, France

The French levelling network consists theoretically in 452 400 benchmarks distributed along 298 300 kilometers of railways, roads, waterways and rivers. The identification sheets of these benchmarks are captured in databank since 1983 ; the capture of the whole network is now achieved. The direct access to this databank started in 1991 with Minitel, home terminal of the french telecommunications system, which has been developped in the eighties. By this way, about 50 000 benchmarks per year are asked. The advantages for the users are the direct and permanent access, the choice of selecting criterions to get the benchmarks they need. The access by Minitel is well appreciated by the users and there is no question of giving it up but it was decided to make the informations of the database also accessible, from 1999, through the Net.

1- The French geodetic and levelling networks

The French National Geographic Institute (IGN : Institut Géographique National) is in charge of the setting-up, the maintenance, and the data distribution of the geodetic and levelling networks.

The French geodetic network (RGF : Réseau géodésique Français) is made up of about 80 000 sites. It comprises 3 parts :

- the reference network : RRF ("Réseau de Référence Français") - 23 sites established in 1993
- a first level of densification : RBF ("Réseau de Base français") - 1009 sites established from 1994 to 1996 by GPS
- the previous network established by triangulation method until 1991.

A permanent GPS network is being developped (RGP : Réseau GPS Permanent) already including 8 stations. It shall become active in the next months.

The French levelling network is divided up as follows :

order	Length (km)	count of benchmarks	precision in mm	mean date for observations
1st	13 800	22 400	2,0 x $(D\ km)^{1/2}$	1964
2nd	18 500	30 000	2,3 x $(D\ km)^{1/2}$	1980
3rd	46 000	76 000	3,0 x $(D\ km)^{1/2}$	1980
4th	170 000	264 000	3,6 x $(D\ km)^{1/2}$	1970
profiles of rivers	50 000	60 000	4,0 x $(D\ km)^{1/2}$	1950
total :	**298 300**	**452 400**		

This network contains theoretically around 450 000 benchmarks along 300 000 km of railways (mainly 1st order), roads, waterways and rivers. In fact, less than 400 000 benchmarks are accessible because some of them are destroyed or are doubtful. Nowadays, IGN maintains only the network from order 1 to order 3.

2- The users

Their needs are expressed in France through a specific committee called CNIG (National Council for Geographic Information). The last recommendations about levelling have been made in 1994 and completed in 1997. In result, IGN has :

- to restore periodically (10 years) the network.
- to continue and improve the on-line data distribution.

3- The geodetic data bank

The figures 1 and 2 show the evolution of the description sheet of the same benchmark from the 60'ies (fig. 1) to the 80'ies (fig. 2) and the figure 3 is an example of what you get finally by fax using the Minitel. The difference is that the sketch is replaced with a literal description of the benchmark location.

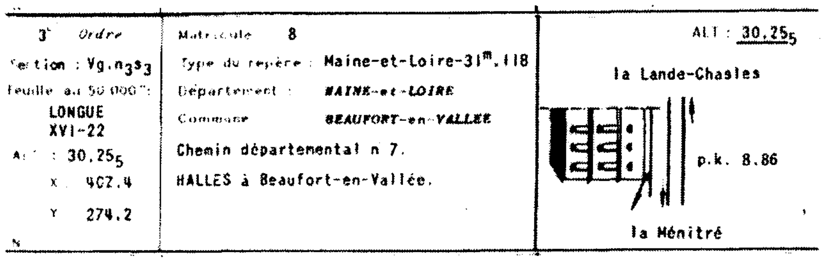

3ᵉ Ordre
Section : Vg.n_3s_3
Feuille au 50 000ᵉ :
LONGUE
XVI-22
Alt : 30,25$_5$
X 407.4
Y 274.2
Matricule 8
Type du repère : Maine-et-Loire-31^m.118
Département : MAINE-et-LOIRE
Commune BEAUFORT-en-VALLEE
Chemin départemental n 7.
HALLES à Beaufort-en-Vallée.
ALT : 30,25$_5$

figure 1

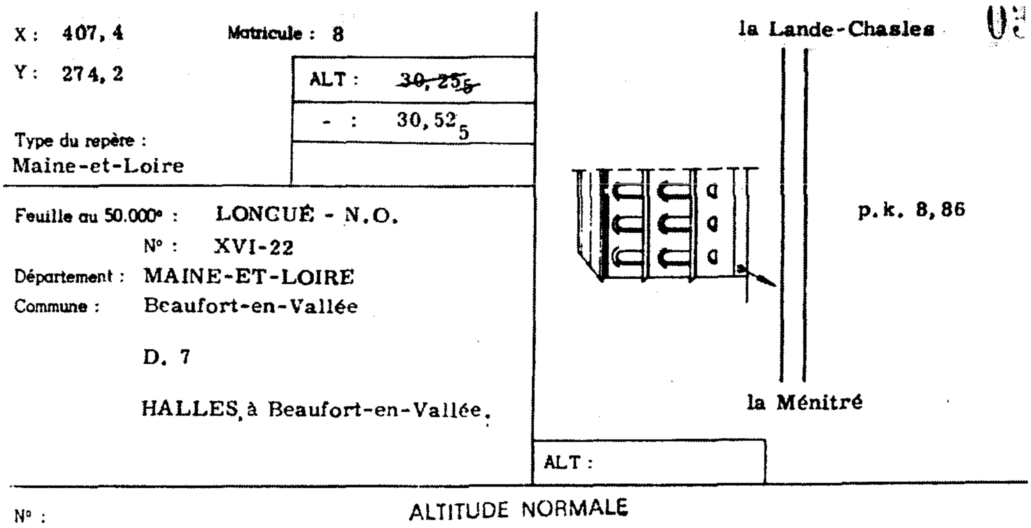

X : 407,4
Y : 274,2
Matricule : 8
ALT : ~~30,25$_5$~~
- : 30,52$_5$
Type du repère :
Maine-et-Loire
Feuille au 50.000ᵉ : LONGUÉ - N.O.
N° : XVI-22
Département : MAINE-ET-LOIRE
Commune : Beaufort-en-Vallée
D. 7
HALLES, à Beaufort-en-Vallée.
ALT :
N° :
ALTITUDE NORMALE

figure 2

Serveur Minitel : 08.36.29.01.29 11 MARS 1999 Page 2 / 2

INSTITUT GEOGRAPHIQUE NATIONAL RESEAU FRANCAIS DE NIVELLEMENT DE PRECISION

REPERE : V.G.N3S3 - 8

TYPE : B MAINE-ET-LOIRE Observé en 1961 Calculé en 1985
DEPARTEMENT : MAINE-ET-LOIRE
COMMUNE : BEAUFORT-EN-VALLEE (N° INSEE : 49021)
FEUILLE AU 1 : 50 000 : LONGUE (N° 1622) - Quart : Nord-Ouest
VOIE SUIVIE :
D.7
de BOIS-MAUDET à LA MENITRE - côté DROIT
PK : 8.86 km
COORDONNEES : en Lambert 2 : X = 407.40 km Y = 274.20 km
LOCALISATION AU REPERE :
DANS L'AGGLOMERATION , RUE DU COMMERCE
BATIMENT DES HALLES
SOUBASSEMENT DU MUR DE FACADE EST, FACE ROUTE
A 0.36 M DE L'EXTREMITE SUD
A 0.29 M AU-DESSOUS DE L'ARETE SUPERIEURE

ALTITUDE NORMALE : 30.512 m SYSTEME D'ALTITUDE IGN 1969 © I.G.N. Paris 1999

figure 3

The geodetic databank was conceived in the 8O'ies in order to have a tool for a better management of data and informations about geodesy, in the general meaning. This data are relative to geodetic, levelling, astronomical and gravimetric networks and consist in descriptions, observations, coordinates and auxiliary data such as ellipsoid parameters, transformation parameters between reference frames, etc .... .

It is a very useful tool for data updating, storage and distribution.

4- The Minitel

Minitel is the name of the terminal attached to the Teletel system which is the specific French videotex system implemented in the early 80'ies. Teletel (usually called Minitel) can be defined as a nation-wide intranet that is centrally managed and in which the admission is by membership only.

Nowadays, 20 millions people in France have access to this system trough 6 millions of Minitel terminals or trough Internet. The good sucess of Minitel is due to the fact that at the beginning, it was distributed free of charge to all French telephone subscribers, it is very simple to use it, it allows to research phone number from an on-line update base, to make reservations for train, plane, to manage bank accounts, ... Today, 25 000 on-line services are available.

5- The data distribution by Minitel

It is possible for anybody, having a Minitel terminal, to have access to the levelling databank. The first thing to do is to dial the specific phone number 08 36 29 01 29 in order to be connected to the geodetic and levelling databank. The benchmark research is possible by 3 ways : the benchmark identification, the parish name or the definition of a circular area. When the choice is made, the description of the selected benchmark appears on the screen and since 1994, the sending by fax is proposed. Around 6 minutes are necessary to get such a description, it corresponds to the time to display the successive screen pages leading to the altitude (see some examples fig. 4). The present price for the users is 9,21 French francs per minute (1,40 euros).

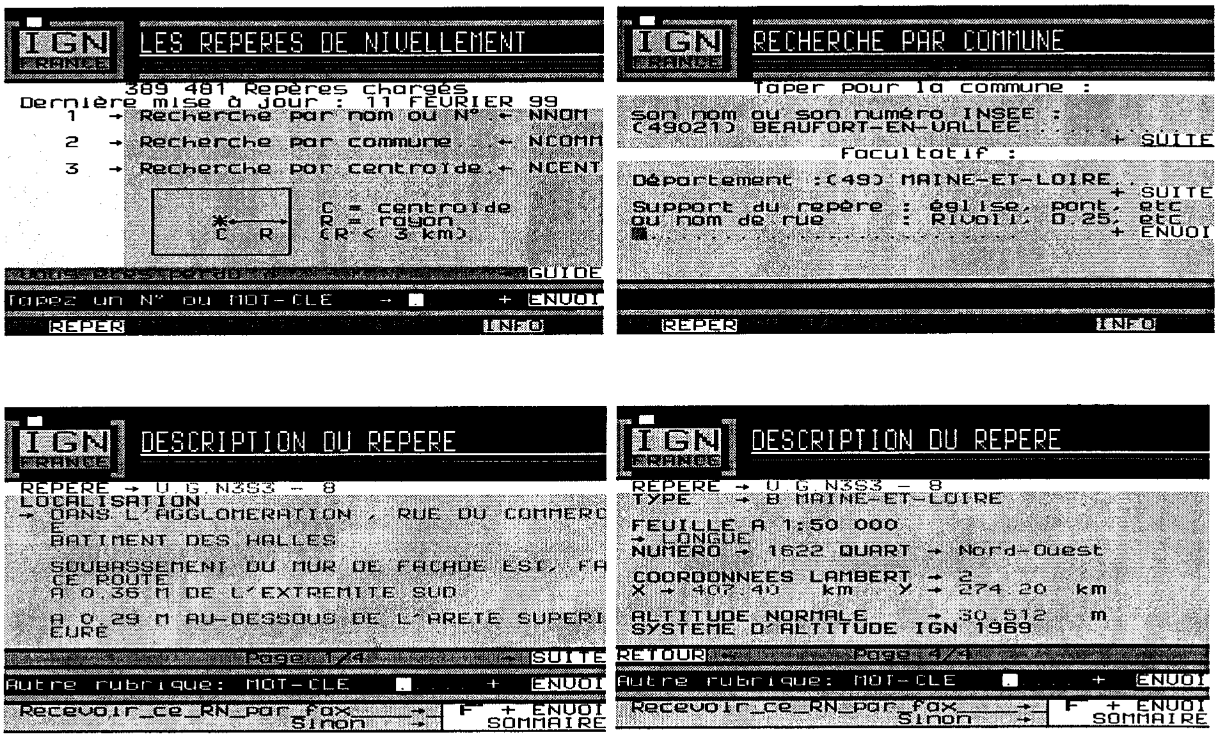


figure 4

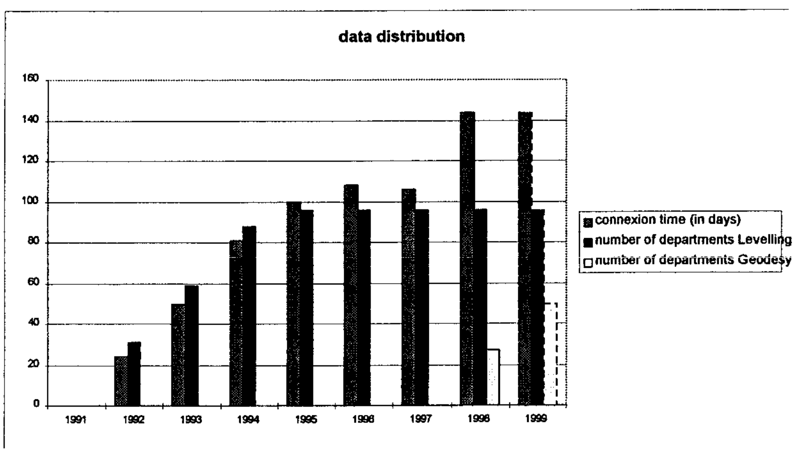


figure 5

The graph (fig.5) presents the evolution of the data distribution by Minitel for benchmarks and geodetic points from the beginning in 1991. The connexion time is given in days as unit in order to compare it with the number of counties for which all of the benchmarks are available. It can be noticed the introduction of the the fax at the end of 1994 and the availability of geodetic points since 1998.

6- The future

There are mainly two questions to deal with the levelling network.

- The first one is that until now, 400 000 benchmarks are accessible trough Minitel although 270 000 of them are not maintained on the field (4th order) ; the final question is : do we need to maintain such a network density whereas altitudes can be obtained using GPS and a good geoid model ?
- The second one is that Minitel is well established, but only in France where now, Internet is growing very quickly. Comparing to Internet, Minitel has some weak points : it looks primitive, it can display only text, it is slow and it can be considered as expensive (the time connexion can cost till 10 French francs per minute) but Minitel has some strong points : it is very easy to use because it is simple, and it is reliable and secure.

But probably the answer will come trough the governmental decision from 1998 : all of the public data have to be accessible through the Net by the end of 1999.

Requirements from urban users on heights, examples from Stockholm

Dan Norin, Stockholm City Planning Administration

ABSTRACT

In a city like Stockholm you can find a great need for height information within the building and construction industry. For existing buildings, heights are needed to control vertical deformations. Of great importance is the possibility to document the three-dimensional position of water pipes, sewer systems, electric cables, etc. It is vital that an unambiguous height system is used for all applications, especially when data are stored and exchanged in a data base. The Stockholm City Planning Administration is responsible for providing such a height system and as such maintains a series of bench marks throughout the city, making connection to the system possible. Different users have different requirements of accuracy and availability of the bench marks.

Several city administrations and other organisations require access to a large scale digital base map. Such a product covering the entire city is available from and maintained by the City Planning Administration. For example, city planners are using the base map to visualize new building projects. Now there are requirements of three-dimensional visualisation using computer animation. The height information in the base map is today limited to contour lines and some street elevations. Several methods have been tested in an attempt to find the best way to convert the base map into a three-dimensional city model.

1. THE CITY OF STOCKHOLM

Stockholm is the capital of Sweden and has functioned as an urban centre with a civic organisation and local government from as long ago as the middle ages. Stockholm was first granted a town charter in about 1250. Today the population is 736 000 and the area covers around 200 square kilometres. Together with the surrounding municipalities, the area of "Greater Stockholm" has a population of approximately 1.7 million.

1.1 The City Planning Administration

Over the centuries, the shaping of Stockholm through planning and construction has always been regulated and controlled by the city. The first City Engineer was appointed in the year of 1636 and the first City Architect in 1661.

The City Planning Administration has a comprehensive role in the development of the city. It is responsible for city planning in Stockholm (in general and in detail), for granting building permits, for making housing accessible to the handicapped and for matters concerning surveying. The administration is also in charge of land parcelling and registration of property. All the work involves weighing public and private interests and protecting the beauty of the city by safeguarding its profile and landscape.

Among the surveying matters, map production is essential. Maps in varying scales are produced, where the base map is a digital large scale product covering the entire city, that is updated on a three years basis. Buildings and properties are however updated continuously. The base map is used both inside and outside the administration for various purposes such as planning, building, documentation, etc. The field crews perform different kinds of surveying tasks and last, but not least, control networks are maintained.

1.2 Control Networks

Today there are approximately 2100 bench marks in the city that have been surveyed using precision levelling. They are stored in a database and form a well-distributed base for all height measurements. A typical bench mark is a steel bolt, preferably drilled into bedrock or into the foundation of a building standing on bedrock, and can be seen in figure 1.1.

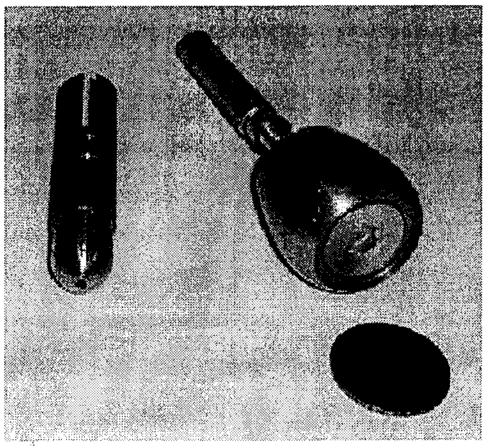


Figure 1.1: Typical bench marks determined by precision levelling.

The height system is connected to the same zero point as in the first precision levelling of Sweden, which is situated on the island of Riddarholmen in the central part of the city (figure 1.2). In the report from the first precision levelling of Sweden it says that "This so-called normalhöjdpunkt is given by the zero point of a graded scale in silver, which is particularly firmly and accurately fixed to the bedrock itself, which here consists of gneiss." (Rosén, 1906). An older height system were in use in Stockholm in the 19th century and in the beginning of the 20th. This makes it important to be attentive on old documents.

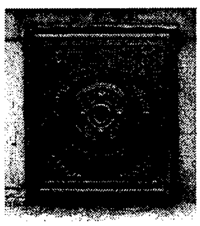


Figure 1.2: The zero point of the height system, as well for the height system of the first precision levelling of Sweden (RH 00).

Bench marks are also determined by trigonometric levelling. This is often in connection with the staking out of new buildings. The bench marks will only be given a centimetre value, but they will also be stored in the database for future use, which increases the access to the height system. Today there are approximately 3300 existing bench marks of this kind in the database. These bench marks with centimetre values will have a point number in the range 400000-499999, which will make it easy to seperate them from the precised levelled ones (300000-399999). To make it easy to get an overview of existing control points, a layer in the base map is maintained. Figure 1.3 shows the base map with this layer, where the horizontal control points are in the range 1-299999.

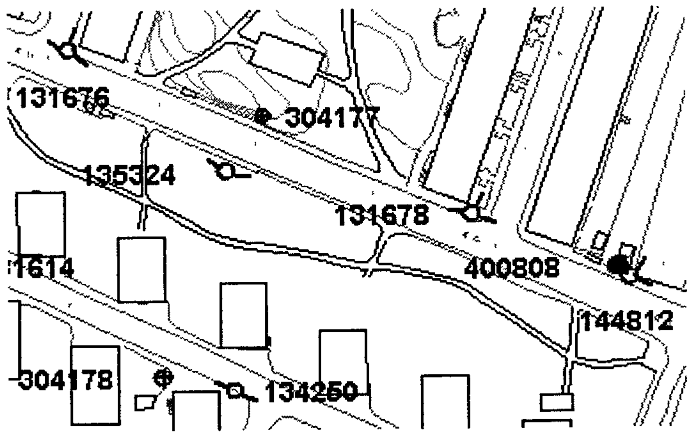


Figure 1.3: Map showing existing control points. Scale approximately 1:2000.

The number of horizontal control points within the city is approximately 28 000. The oldest were laid out in the years 1907-1911 in a local reference system, which is still used. During the last decade GPS has been used for establishing a more homogenous horizontal network (Engberg, 1994), which can be used in larger projects with high demands on accuracy.

2. USER REQUIREMENTS

2.1 Building and Construction

In a city like Stockholm, the need for height information within building and construction is more evident than elsewere. A project is often very dependent on existing buildings and structures. High demands are obvious in the rather frequent construction of tunnels (for cars, underground trains, etc.) and bridges. The users benefit from the fact that the same height system is used for all kinds of projects. A new bench mark is often placed, using trigonometric levelling and given centimetre value, close to a construction site.

The use of visiualization in construction projects, e.g. highway crossings and landfills, in the form of a terrain model is constantly increasing. There is an existing sparse terrain model covering the city that can be used for orthophoto production and coverage planning for example, mobile phones, etc.

2.2 Documentation

There are many administrations, incorporated companies and other companies that work with the infrastructure of the city. Several of these have the need to document the position of objects like water pipes, sewer systems, electric cables, etc. Those objects which have been surveyed are documented together in a digital map based on the base map provided by the City Planning Administration. This arrangement using one common map -the base map- has been found to be a good economic solution.

The vertical positions of the above mentioned objects, both on the ground and under the ground, are also often documented and the need to do this has increased and is increasing. The users ask for a dense well-distributed network of bench marks, where centimetre accuracy is acceptable.

2.3 Deformation Control

Stockholm lies to a large extent on bedrock. There are also large deposits of clay and the shoreline has in some places been extended with fill. These two last areas can be troublesome, causing deformations in existing buildings. There are records of deformation measurements on the ground itself and on approximately 2000 buildings, where something like 500 are regularly controlled. Of great concern is the area of the "old town" in the city centre.

Deformations are monitored by levelling vertical steel bolts placed in the buildings' foundations. This requires accurate bench marks that are firmly placed, preferably in bedrock. The distance to the closest benchmark is also of importance.

2.4 Map Production

The height information in the base map is to a large extent, as for other maps, based on photogrammetric methods. It is today limited to contour lines and some street elevations. When there is need, geodetic methods can be used to complement height information. To meet the requirements to convert the base map to a three-dimensional model, an efficient method must be found.

2.5 Planning

Construction planning has always required height information. Today, city planners also have a wish to visualize new building projects in three dimensions by the use of computer animation.

3. FUTURE DEVELOPMENTS

3.1 Control Networks

There is a good understanding by users of heights in Stockholm to the benefits of connecting to the height system through existing benchmarks. There are wishes for a more dense network. To accomplish this, new horizontal control points surveyed today are determined also in height. This of course only if the point is suitable for height determination. Also existing horisontal control points can be surveyed to include height.

It will soon be easier for specific users to obtain coordinates/heights and descriptions of points by downloading from a web-page with a clickable map.

It is important to determine heights on points suitable for being reference points for GPS measurements. This to make it easier to use the GPS for vertical positioning.

3.2 Map Production

There are various ways to create a 3-D city model. One method is to build it from scratch, with complete coverage, using conventional aerial photogrammetry. Of course this is a very costly endeavour. Another way is to convert the base map into a three-dimensional city model. Several methods for this task have been tested. Without going into detail about the methods, they include techniques to get building heights and roof shapes based on pure estimation, stereo photogrammetry, automatic digital photogrammetry and single frame photogrammetry with a method where at least four digital aerial photographs must be accessible. The most exciting test is based on measurements from a helicopter with a laser rangefinder that scans the ground across the track (figure 3.1). The system promises decimetre accuracy for both ground level and building heights (Saab Survey Systems, 1996). The position of the helicopter is pinpointed through a combination of GPS and inertial systems.

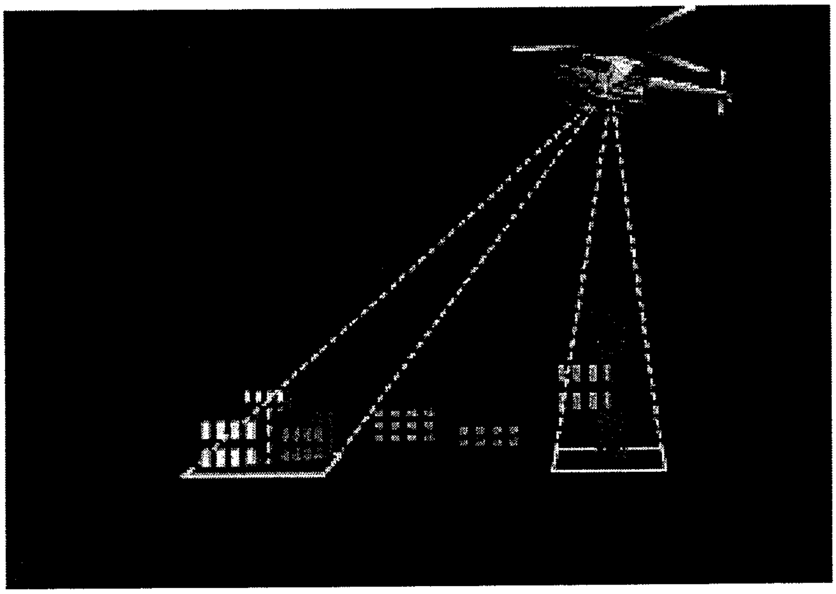


Figure 3.1: Helicopter based system with scanning laser rangefinder inprinciple.

A 3-D city model can be generated from the data (figure 3.2). The model can be further interpretated by the use of reflected data and multiple echoes and it can also be combined with the base map.

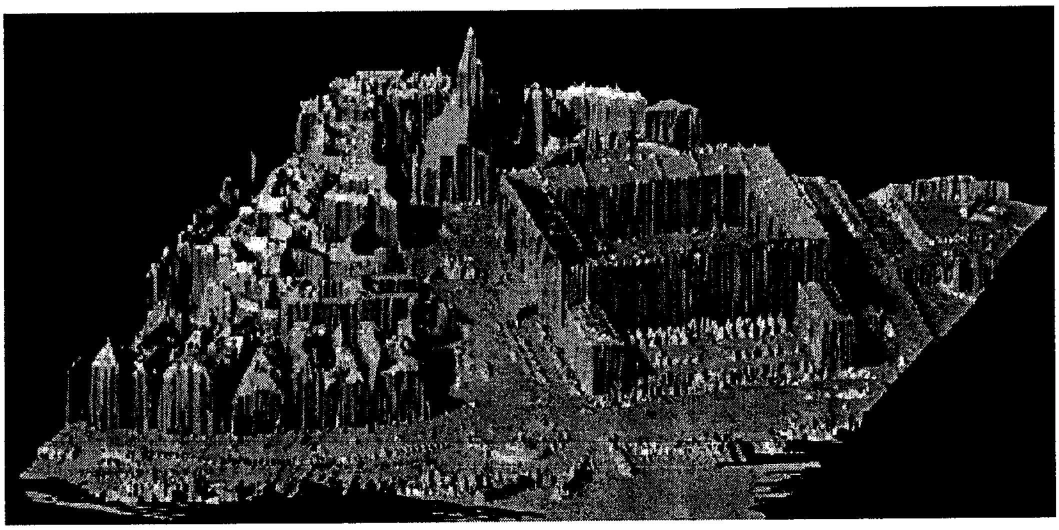


Figure 3.2: Perspective view over a part of the "old town" in Stockholm generated from laser scanning data.

Part of the above mentioned tests are done in collaboration with the Royal Institute of Technology in Stockholm.

3.3 New Techniques for Height Determination

A great benefit with GPS is the possibility for simultaneous determination of positions in three dimensions. Today reference points are used for the reference station, but with a service providing real-time data continuously, GPS can be cost effective for detailed surveying. A great disadvantage in an urban environment is the poor satellite visibility, which to some extent can be improved by the use of Glonass satellites. For height determination a geoid model is needed, such a one with nationalwide coverage is provided by the National Land Survey of Sweden. It is not possible to use heights above the ellipsoid within surveying, because a common user will have no understanding for this height.

In an urban environment, height determination can improve from methods based on terrestrial photogrammetry and so-called Mobile Mapping Systems.

REFERENCES

Engberg L E, 1994: The Establishment of an Improved Reference Network in Greater Stockholm by Densification of the National Network with GPS Technique. FIG XX Congress 1994, Melbourne, Australia.

Rosén P G, 1906: Sveriges precisionsafvägning 1886-1905 (in swedish). Stockholm.

Saab Survey Systems, 1996: Saab TopEye, the versatile airborne survey system (brochure). Saab Survey Systems, Jönköping.

Precise levellings in Finland

Jussi Kääriäinen

Finnish Geodetic Institute,

Geodeetinrinne 2, 02340 Masala

1. Introduction

Since the end of the last century, two precise levellings have been made in Finland. The third national precise levelling started in 1978 and field observations have been estimated to be completed in 2001. In this paper the short description of these three levellings and the applied measuring techniques are given.

2. The First Levelling of Finland

The First Levelling of Finland was performed by National Board of Public Roads and Waterways during 1892-1910 (Blomqvist and Renqvist 1910). It covered southern and middle Finland up to the latitude 65°N. Its network consisted 11 closed loops with circumferences between 229 and 808 km and the accuracy, calculated from the adjustment of the main network was ⇧ 1.48 mm/×km. The main network with branch lines and extensions in 1911-1913 is presented in Fig.1.

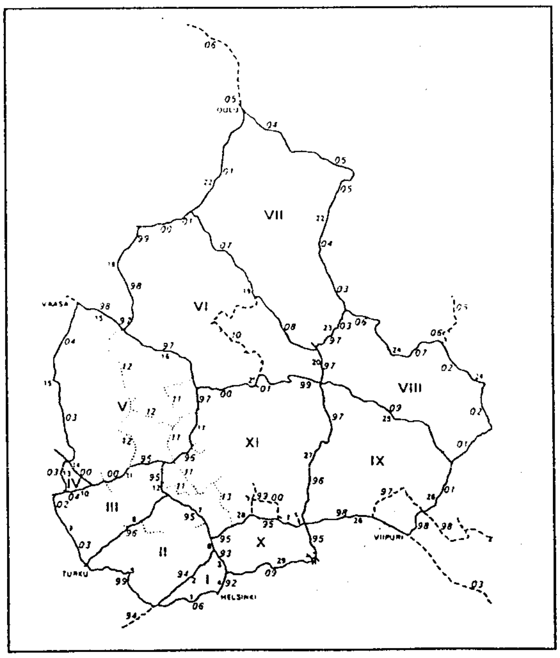


Fig.1. The network of the First Levelling, levelling years, number of the loops and of the lines. Solid lines present main levelling lines, dashed lines depict branches and dotted lines extensions in 1911-1913 (Kääriäinen 1966).

In the treatment of the observations of the First Levelling, the scale correction of the rods was taken into consideration and the orthometric correction resulting from the divergency of the theoretical equipotential surfaces was added.

The measuring technique was foot levelling. The sight lengths were equal to the both rods and the sight length did not exceed 75 m. In the plans of the work it was, however, mentioned the maximum sight be 50 m.

Two instruments were used in the First Levelling. The first was made by Breithaupt & Sohn and the second, purchased in 1901 was from the firma A.Berthélemy. The wooden rods were 3 m in length and graduated to 4 mm:s interval by black and white figure. There were graduation on both sides of the rod so that the sum of the readings was all the time constant to avoid gross errors both in observations and in writing. The rods were compared to the standard meters, maximum seasonal deviations were found to be 0.32 mm/m mainly due to the relative humidity.

Rod supports were metal pins, pounded into the ground. On the side of the pin there was a fork on which the rod was placed. These supports were used on the land roads and probably on the railway lines too.

The mean distance between successive bench marks was about 2 km. The elevation difference was measured forth and back. In forth measurements the back rod was first observed (both sides). When the bench mark interval was measured back the observations were made with readings on the front rod first.

3. The Second Levelling of Finland

The Second Levelling of Finland was performed by the Finnish Geodetic Institute in 1935-1955 (Kääriäinen 1966). Due to the Fennoscandian land uplift, detected already from sea level records and geological evidences the repeated high quality national levelling became scientifically interested. Another reason for the remeasurement of the network was the large amount of devastated bench mark of the First Levelling as a consequence of the enhanced construction works. Finally, the extension of the network to the northern part of the country was necessary for both scientific and economic reasons.

The Second Levelling was entrusted to the Finnish Geodetic Institute. In 1932 some experiments were made in order to find the most suitable instruments and working hours. The field work started in 1935. The number of expeditions varied from one to four observing in every year, except the war time, and in 1955 the main part of network, depicted in Fig. 2 was measured. This network consisted 18 closed loops, 12 lines to the mareographs of the Marine Research Institute and two joints to Sweden. The accuracy, obtained from the adjustment was ⇧ 0.64mm/×km.

The measuring technique was continuously the traditional foot levelling. In the beginning, in years 1935-1937 two Zeiss III instruments were used. In 1937 two new instruments, model Zeiss NiA were purchased and also used, later still more so that since 1938 this instrument was used exclusively.

The maximum distance from the instrument to the rod was 55 m in good weather conditions. A new correction was applied on the observations, namely refraction. The method was developed by Kukkamäki (Kukkamäki 1938) and corrections based on the measured vertical temperature gradient at each instrument station. The rods were of wooden frame with invar band having two scales with

5 mm:s graduation, offset with regard to each other.The rod read-ings at each instrument station were made in succession back-forward-forward-back.

Concerning the rod supports on the roads, heavy iron plates, the crapaudines, were used in the beginning but were soon rejected due to the unstability, especially on clay roads and wedge like steel pins were approved instead. To avoid the deformation of the spherical upper end by hammering, the special device, "pounding hat" was used.

On railway lines the pins were not anymore used but a special device was attached on the foot of the rail with an eccentric disk. During the passing of a train the device remains on the rail. It moves noticeably together with the rail as the train passes but returns, according to many tests and experiments to its original position once the train has gone.

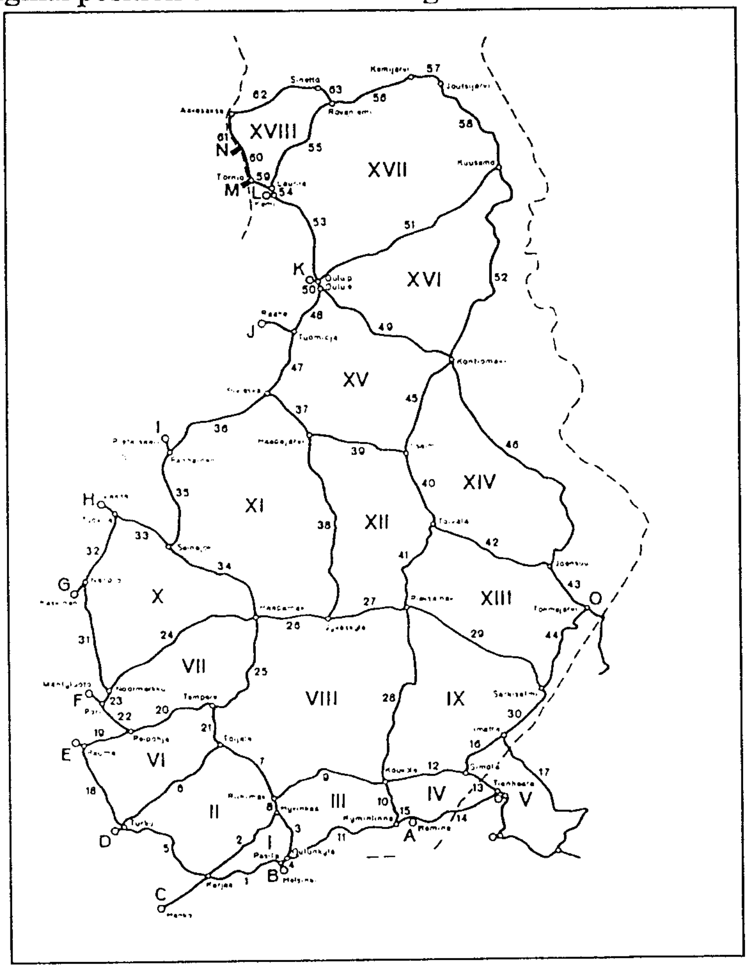


Fig.2. The network of the Second Levelling, measured in 1935-1955, with numbers of the loops and of the lines. (Kääriäinen 1966).

Observations were corrected for the scales of the rods and for the refraction as well. Measured height differences were converted to geopotential units using either observed or interpolated gravity values. The influence of the Earth Tides was already discussed (Kukkamäki 1949) but no corrections were made at the time.

The main network was completed in 1955 and the net was adjusted and for the first time the land uplift was taken into account resulting the national height system N60.

The levelling was continuing, however, more far in Lapland. These measurements were carried in two periods, first during 1953-1962 and later 1971-1975 when also one line from Kemi to Karigasniemi was remeasured for land uplift determination. The network for Lapland is depicted in Fig.3 consisting three loops and five branches (Takalo and Mäkinen 1983).

Most of the measuring instruments and methods were of the same type as those in the fundamental part of Second Levelling. The main instrument was ZeissNiA but some test measurements were done also with Ni1 level from Opton Feintechnik, Oberkochen. In remeasurement along the line Kemi-Karigasniemi instruments Zeiss NiA, Zeiss Ni002 and MOM Ni-A31 from Magyar Optikai Muvek, Hungary were used. In this measurement bicycle and handcar levellings were applied for first time (see Chapter 4).

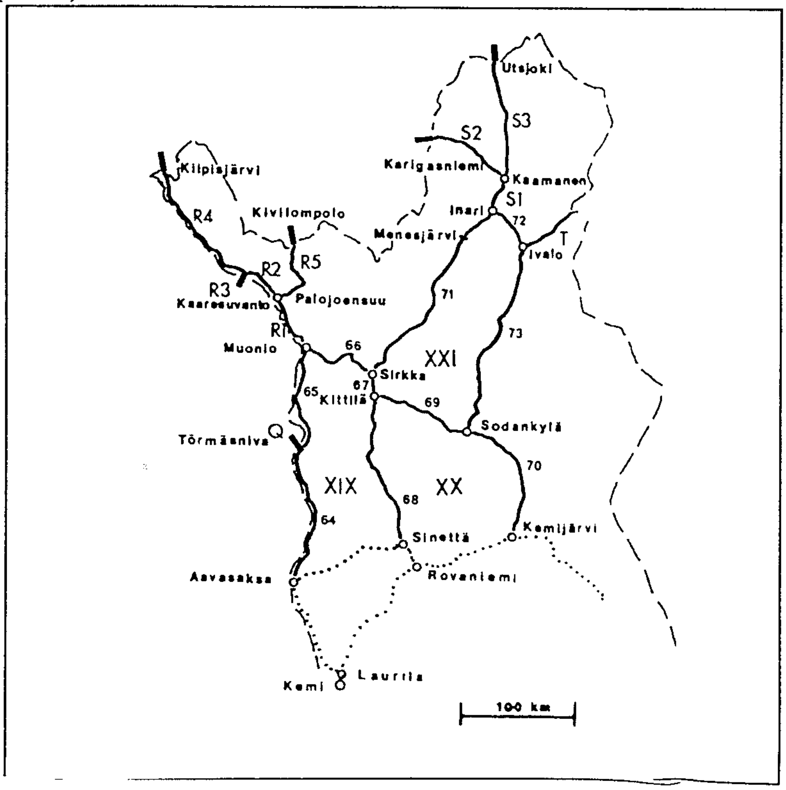


Fig.3. The network of the Levelling of Lapland, measured in 1953-1975, with the numbers of the loops and branch lines (Takalo and Mäkinen 1983).

In 1962-1975 the Second Levelling of Finland was extended also to mainland Aland and further to island Märket on the borderline between Finland and Sweden. This measurement connected also the mareograph in Degerby to the levelling network. Special techniques for watercrossings were applied using either two ZeissNiA simultaneously or two double instrument of Zeiss Ni2. To determine the refraction correction, extremely important in this kind of asymmetric measurements, simultaneous thermal vertical gradient measurements were made all along the sight. Some hydrostatic measurements using communicating vessels were also done (Kakkuri and Kääriäinen 1977).

The levelling over rocky islands was carried out using crapaudines as the rod supports.

4. The Third Levelling of Finland

The field work of the Third Precise Levelling began in 1978. The aim was to measure all the lines and benchmarks including to the Second Levelling of Finland. In the course of the work some additional lines, however, were added to the network as shown in Fig.4

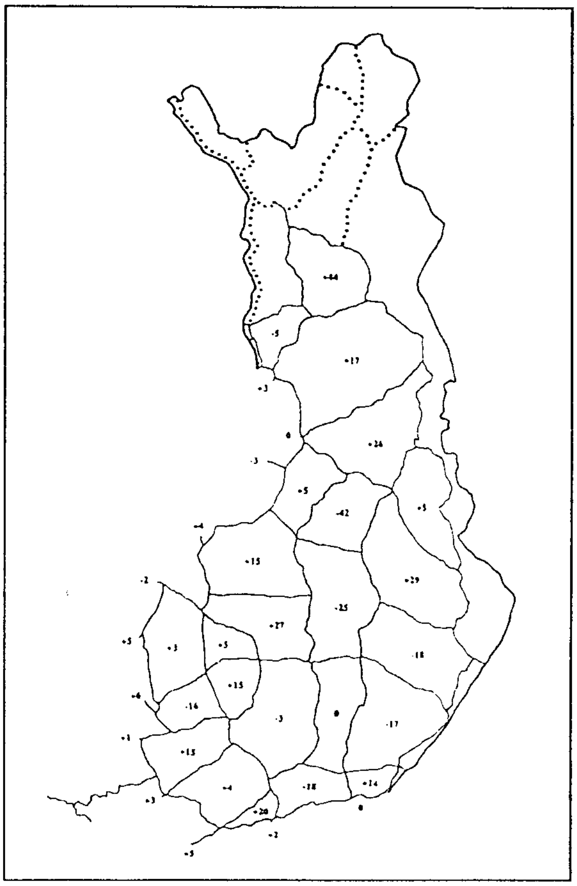


Fig.4. The network of the Third Levelling of Finland. Solid lines represent measurements 1978-1998, dotted lines plan for 1999-2001. Numbers are misclosures in mm.

Many things were changed in the dawn of the Third Levelling. In the following the principal modifications are only briefly mentioned, for details reader is referred to Takalo (1978).

The technique applied was not anymore the conventional foot levelling. To enhance the measuring speed in order to reduce the systematic measuring errors and to increase the productivity the bicycle and handcar techniques were developed for the land roads and for the railways, respectively.. The increased speed of moving was a good argument for the application of automatic levelling instruments for further increase of measuring speed. New automatic levelling instruments Ni002 from Zeiss Jena were approved. On the land roads the rod supports were not anymore wedge-like pins but mainly steel pins. Each pin had a length of 40 cm, diameter 1,4 cm and weight 450 g. The upper end was spherical and the pin was driven into the ground with a special device. Various types of rod supports were used in handcar levelling, like rail screws, pandrols or rail spikes. Small steel balls were also used, set on the upper surface of the rail. For this a small pit had to be made on the rail. The drawback was, that often this small ball rolled away and was difficult to find again.

The distances were not anymore determined by steel tapes but a rollmeter, attached directly to the handcar or slightly modified, to the bicycle.

With these modifications the work was started in 1978. The average levelling speed settled to 1.9 km/h in bicycle levelling and 2,5 km/h in handcar levelling. Formerly, in the Second Levelling the corresponding speeds were 1.13 km/h on the roads and 1.37 km/h on railways.

It appeared, however, that in the first five loops the closing errors were slightly larger than expected and four of them had the same sign. This was leading to the instrumental tests where so-called parallax error and also the possible sensitivity to the magnetic field were studied (Lehmuskoski 1982, Kukkamäki and Lehmuskoski 1984). The results could not, however, explain the modest accuracy of the levelling but was one reason to return to the methods of the Second Levelling. Second reason was the senseless closing errors in 1983 which was also the last one with automatic instruments. In 1985 the observations were carried out with old faithful instruments Zeiss NiA and since 1986 manual spirit level Wild N3 instruments are used exclusively. There are three expeditions in the field work measuring every year altogether 445 km double run (average 1994-1998). The expedition average of the recent levelling speed is 1.4 km/h.

The levelling refraction is continuously calculated on the basis of the measured vertical thermal gradient between 2.5 and 0.5 m elevation at every instrument station.The observed height differences are also corrected for tides.

In the Third Levelling the scale of the rods are determined in the vertical rod comparator, constructed by Takalo (Takalo 1997). The comparator is fully automated and the rod scales can be investigated line by line rather easily. The temperature and humidity can be regulated in the comparator laboratory, thus the thermal extension coefficient of the rods also can be determined. The reader is referred to the paper of Takalo in this proceedings.

Since 1988 the data have been collected using Husky Hunter data collectors. The data are stored on diskettes, printed daily on paper and later on bound to an "observation notebook".

The field work of the Third Levelling will be completed at the end of 2001 and a new height system will be created. For that a new epoch, possibly a new reference level and also the choice between orthometric and normal heights will be discussed.

5.References

Blomqvist, E. and H. Renqvist (1910): Das Präcisionsnivellement Finlands 1892-1910. Fennia Nr. 2. Helsinki.

Kakkuri, J. and J. Kääriäinen (1977): The Second Levelling of Finland for the Aland archipelago. Publ.Finn.Geod.Inst. No.82. Helsinki.

Kukkamäki,T.J. (1938): Über die nivellitische Refraktion. Veröff.Finn.Geod.Inst. Nr.25. Helsinki.

Kukkamäki, T.J. (1949): Tidal correction of the levelling. In: Publication dedicated to Ilmari Bonsdorff on the occasion of his 70th anniversary. Publ.Finn.Geod.Inst No. 36 Helsinki.

Kukkamäki, T.J. and P. Lehmuskoski (1984): Influence of the earth magnetic field on Zeiss Ni 002 levels. Rep.Finn.Geod.Inst. 84:1. Helsinki.

Kääriäinen, E. (1966): The second levelling of Finland in 1935-1955. Publ.Finn.Geod.

Inst. No.61 Helsinki.

Lehmuskoski, P. (1982): Systematic error resulting from asymmetric handling of a Zeiss Ni 002 automatic levelling instrument. Rep.Finn.Geod.Inst. 82:3. Helsinki.

Takalo, M. (1978): Measuring method for the third levelling of Finland. Rep.Finn.Geod.Inst. 78:3. Helsinki.

Takalo, M. and J. Mäkinen (1983): The Second Levelling of Finland for Lapland. Publ.Finn. Geod.Inst. No.99. Helsinki.

Takalo, M. (1997): Automated calibration of precise levelling rods in Finland. Rep.Finn. Geod.Inst. 98:5. Kirkkonummi.

Experience of technical co-operation in Zambia and Mozambique

Sören Lunqvist

Swedesurvey

Swedesurvey is a state-owned company, which markets co-ordinates and provides services in land administration and surveying throughout the world, often in the form of institutional co-operation. We have been active on the international market since 1980 and our aim has been to provide services based on the requirements of the customer.

We have programmes of co-operation with cadastre and mapping organisations in many countries. The primary goal of our activities in this respect is institutional building through development co-operation and transfer of knowledge and technology. We also provide production services within our range of activities.

We mainly draw our resources from the National Land Survey of Sweden and have access to their full range of equipment and modern technology, and the competence of approximately 2,000 qualified professional and technical staff employed by the National Land Survey of Sweden. We provide consultants such as lawyers, cadastral experts, surveyors, geodesists, photogrammetrists, cartographers, valuation experts, computer programmers and administrators who are specialists with first hand practical experience and many of whom have overseas experience too. We also have close contacts with other private companies and governmental departments, we are partners in international consortia and have joint venture agreements with overseas companies.

Our fields of business include:

- **Cadastral and Land Information Systems**

Swedesurvey provides consultancy in land administration, land information systems, cadastre, development of legislation, land use planning, land reform, land tenure analysis, land consolidation, real property formation, land registration, land valuation and cadastral surveys.

Sweden has a state-of-the-art national cadastral system, developed by the National Land Survey, which has provided a model for many other countries to create their own national cadastral systems. When helping to design a computerised system we suggest methods of data acquisition and data conversion and ways of simplifying a cadastre.

- **Surveying, Mapping and Geographic Information Systems**

We cover a wide spectrum of surveying and geo-information services which include aerial photography, geodetic surveying, network design and analysis, ground control surveys, digital orthophoto and line mapping, mapping based on satellite imagery, international border surveys, development of GIS and development and maintenance of geographic data bases from official mapping for public use for a variety of purposes.

- **Management, Organisation and Financing**

We assist governments in more than 30 countries in the implementation and management of mapping, surveying and cadastral systems. Based on experience from Sweden we aim to create systems which are self-financing and generate income on a long-term basis.

- **Training**

Every year, since 1987, Swedesurvey has held an advanced international training course in the Development and Management of Cadastral and Land Information Systems. In 1995 we introduced another annual course in Strategic Planning and Leadership for Mapping and Land Information Organisations. Apart from these programmes we offer courses, both bilateral and international, in Sweden and overseas. Training can also be in the form of seminars, study visits, practical assignments and on-the-job training.

Swedesurvey currently provides services in:

Europe	Armenia, Belarus, Bosnia, Denmark, Estonia, Georgia, Greece, Latvia, Lithuania, Moldova, Poland, Russia, and Ukraine
Asia	Bhutan, China, the Philippines, Uzbekistan and Vietnam
Africa	Botswana, Ethiopia, Eritrea, Madagascar, Mauritius, Mozambique, Namibia, South Africa, Tanzania, Zambia and Zimbabwe
Latin America	Costa Rica and Guatemala
The Middle East	Kuwait and Saudi Arabia

How it all began

Fifteen years ago Swedesurvey was little more than a logotype managed by a small group of entrepreneurs with a vision. It was steered by a board in the National Land Survey of Sweden and encouraged by a Ministry that welcomed the engagement of the Public Sector in the export of Swedish competence.

During the 1970s a number of the Land Survey's staff had worked overseas with international organisations such as UNDP, The World Bank and Sweden's Sida. Would it not be more rational and effective to combine the experience and competence of Swedish specialists with the very considerable material resources of the Land Survey to assist sister organisations in the Third World by establishing institutional co-operation with them? Financing would primarily be from aid organisations. Geographically the focus would be on eastern and central Africa.

From the start we were convinced that working in new environments with different cultures would also contribute to the development of the Land Survey of Sweden. Since 1980 Swedesurvey has worked closely with the Ethiopian Mapping Agency and has contributed to creating one of the best survey organisations in Africa. Soon after followed a similar agreement with the Zambian Survey Department.

The next ten years were a period of expansion. For the most part we succeeded in developing a network of contacts first in Africa: Ethiopia, Libya, Sudan, Zambia, Botswana, Mozambique, Zimbabwe and Lesotho, and thereafter in Asia: Singapore, Brunei, Malaysia, Indonesia, Bhutan, China and the Philippines.

Sida

Sida (Swedish International Development Co-operation Agency) was established 1965 and is the government body responsible for the implementation of development co-operation with developing countries.

The goals of Swedish development co-operation are as follows:

1. Economic growth. To help increase the production of goods and services.
2. Economic and social equality. To help reduce differences between rich and poor and ensure that everyone's basic needs are met.
3. Economic and political independence. To help to ensure developing countries can make their own decisions on their economies and policies and create the conditions necessary for national self-determination.
4. Democratic development. To help to ensure that people are given greater opportunities to influence developments locally, regionally and nationally.
5. Environmental protection. To promote the sustainable use of natural resources and protection of the environment.
6. Gender equality. To promote equality between men and women.

Zambia

Zambia became independent on the 24 October 1964. The neighbouring countries are Congo-Kinshasa, Tanzania, Malawi, Mozambique, Zimbabwe, Angola, Botswana and Namibia. Zambia is a member of United Nations and the Organisation of African Unity (OAU) and is an Africa - Caribbean - Pacific (ACP) country.

Zambia is one of the most urbanised countries in Africa south of the Sahara with almost half of its population living in urban areas. The population is concentrated in the Copperbelt and along the line of railway which includes the cities of Livingstone, Lusaka and Kabwe, whereas the rural areas are sparsely populated. Zambia's total land area is 750,000 km^2. The country's population is growing rapidly (3.5% per year) and it is estimated that it will exceed 10 million by the year 2000.

The City of Lusaka has a population of more than 1.4 million and is growing at a rate of 6% per year. The city administration is unable to cope with the problems arising from this rapid growth in population and according to a recent World Bank report more than 60% of the urban population live in informal settlements.

Early days in Zambia

Co-operation between Zambia and Sweden in the field of mapping and land surveying started as early as 1975. During 1975 and 1976 SIDA supported the post of Assistant Surveyor General (Mapping) and a Swedish professional land surveyor held the position for two years. Mapping of the entire country was recognised as an essential tool for development planning during this period. The emphasis of Swedish support to Zambia was to the agricultural sector and during a very thorough agricultural sector support review it was stated that "the work of Survey department is of great importance to the development of the Zambian rural sector. Planning must be based on proper maps".

From the beginning the intention was that support to Survey Department should be undertaken as institutional co-operation between two similar organisations. The National Land Survey of Sweden (Swedesurvey) was identified as the most suitable counterpart.

In 1979 a consultancy team carried out a feasibility study. Initially, support was to be directed towards rehabilitation of the technical status of the Survey Department. One specific task in the beginning was to support the mapping programme in the Western part of Zambia. Other components included in the aid were consultancies regarding methods and organisation, the introduction of higher education in land surveying, support with purchasing etc.

The overall aim was to develop the services of the Survey Department to such an extent that they should become self-reliant in technical matters and the production of the national mapping programme. During the first phase there were some specific projects which were supported.

- Precise levelling in northern Zambia, Luapula Province
- Precise Doppler Satellite Control in Western Province
- Aerial Triangulation and Adjustment in Western Zambia
- Mapping in Western Zambia from Satellite Imagery
- Mapping of North-Western and Copperbelt Provinces

In later phases other subjects included were:

- Mapping for small scale farmers
- Mapping of land under traditional tenure.

Experience

In general, experience has shown that, despite a period which was plagued by economic recession, it was possible to maintain the services of Survey Department, and in some areas even improve them.

During the institutional co-operation it has been obvious that the Zambian counterpart has shown a lot of interest in and attached great importance to the development project. Implementation of the activities has been carried out in close co-operation between officers from Survey Department and officers from Swedesurvey based on the idea of transfer of know-how through institutional co-operation.

In the beginning co-operation consisted mostly of field projects which lasted a long time i. e. several months, in which co-operation between individuals was essential. During such projects a great deal of importance was attached to the planning of the projects as well as the implementation and follow-up. The work was done under the leadership of the Zambian officer supported by the expert and this led to the development of managerial skills.

Over a long period of time of close co-operation between officers the influence of the experienced expert on the local officer has been significant though not easily tangible. The local officer has gained in-sight by observing another type of leadership and management.

Most of the projects under this co-operation have been carried out as combined production and training projects. This arrangement has sometimes caused discussion about performance because training has limited achievements. Although it has not been a major concern, it is important to realise the extent of the result in production and training respectively.

A major problem, which has hampered the development of a technical department like Survey Department, is weak management with few candidates available for the senior management posts. The post as head of the Survey Department, the Surveyor-General, has been held by an expatriate since independence because of the lack of trained local staff. This means that the department has always given the impression of being influenced by foreigners. It also meant that the development projects were thought to be owned, run and implemented by the foreigners. The first indigenous Surveyor General was appointed in 1994, thirty years after independence.

Lands Department

The development of the Zambia Land Information System started in 1984 as a joint project between Swedesurvey and Ministry of Lands, funded by Swedish aid from Sida. Three important registers within the Ministry were to be computerised. They were the:

- PROPERTY REGISTER
- LEASE PAYMENT REGISTER
- LANDS REGISTER

The main objectives of computerisation were to:

- make access to information about land easy
- provide better services
- improve collection of ground rent
- cater for the expected increase in land transactions
- reduce duplication in storage of information
- make it possible to compile information and produce reports that were either impossible or very cumbersome to produce in a manual system.

Property Register

It was decided that the property number should be the main search key for the whole system. Standardised property numbers were created for the purpose. Computerisation started with data collection of the records, kept by Survey Department,of all numbered properties in Zambia.

Lease Payment Register

The project continued with the Lease Payment Register. Zambia has a leasehold system. Information on annual ground rent was stored on cards sorted by account number. The computerised register became more accessible through searches by property number.

Land Register

The last step in the project was the Land Register. All properties with title deeds are registered in the Land Register in Lands & Deeds Registry. The main search key in the computerised Land Register is property number + entry number. A name file in the system makes it possible to search using the name of a title holder.

One system - several data files

Although the data collected from the three main manual registers is stored in separate data files, a common search key makes it possible to combine the information.

Further development

Computerisation of the main registers was completed in 1990 and the system has been in operation since, that is, the maintenance and upkeep of the information stored is carried out continuously from about 30 workstations in various offices.

The system has been further developed to simplify the work in the following areas:

- Offer registration
- Lease numbering in Folio Section
- Registration in the Land Register
- Ground rent collection
- Filing Registry's records of movement of files.

Ground Rent Collection

There has been a tremendous improvement in the collection of ground rent since the system was introduced.

A bill monitoring system makes it possible to print out annual ground rent bills and monitor payments of them. Computerisation has also made it possible to introduce different ground rent fees depending on zoning, location and size of the property.

Capacity building

Capacity building within the Ministry of Lands has been an important part of the development of the Zambia Land Information System.

The system was initially developed, according to the requirements of the Ministry, by staff from Swedesurvey. Staff from within the Ministry carried out data collection. Users at all levels were trained.

A Computer Section has been formed within Lands Department and much of the further development of the system since 1990, an essential tool for development planning, has been done by the staff within the section.

Security

The system administrator is the only user who has full access to all resources in the system. The main reason for this is to protect the system from unauthorised use. Resources are protected on the following levels:

- System level - The LOGON process restricts access to the system to proper users whose IDs and passwords are defined by the system administrator.
- File level - Users can access only those files belonging to the file protection class or classes to which they have access rights.

- Access level - Each file protection class specifies different access privileges, so that some program files can be modified, some can only be read and executed, and some can only be executed.

Users are also restricted from logging on at all workstations. Backup of the system is done regularly on a backup drive to prevent loss of data.

Experience

The idea of using modern technology like computerisation for land information in a third world country in the early 80's was very new and strange, raised many doubts and attracted a lot of questions.

At the on-set of the computerisation process in the Lands Department the staff had very little technical knowledge and there was no modern equipment.

During the training phase for the data collection process it was obvious that the staff found their technical training very easy. After some initial problems, the most time consuming of which was the correction of all the errors in the registers which were detected as a result of computerisation, good progress was made.

The major problem was the management of the system. The traditional management system in the Lands Department was not used to handling such an effective and powerful tool like a computerised land information system. Different parties within the Ministry claimed ownership of the system.

Another aspect of great concern to the co-operation partner Swedesurvey was that the system was often referred to as "the Swedish system" although a lot of effort was made to develop the system according to the customer, Lands Department. This emphasises the importance and the difficulty of establishing ownership of the project from the beginning.

Mozambique

Mozambique became independent 25 June 1975. The neighbouring countries are Zambia, Malawi, Tanzania, Swaziland, South Africa and Zimbabwe. Mozambique is a member of United Nations and the Organisation of African Unity (OAU) and is an Africa - Caribbean - Pacific (ACP) country.

Mozambique's total area is 800,000 km^2 and the population is approx. 15.6 million. The official language is Portuguese. The country is divided into 10 provinces.

All land in Mozambique belongs to the State. A new land law was enacted in 1997 which has provision for individuals and companies to apply for occupancy right to land for 50 years. DINAGECA is assigned the responsibility to prepare such occupancy rights, demarcate the area on the ground and register the rights nation-wide except for the bigger towns. DINAGECA also prepares and approves the title regarding the right. The occupancy right can be inherited but not sold commercially. Any development on the land can be sold commercially.

Land which has been occupied and farmed in good faith for ten years or more belongs to the occupant. Land which has been occupied by inhabitants of a village and used for the needs of the villagers, belongs to the villagers with traditional right and there is no requirement to claim the occupancy right to the land in order to obtain a title. Such land can be demarcated and registered by DINAGECA at the request of the villagers.

DINAGECA

The national cadastre and mapping authority of Mozambique is called DINAGECA, DIRECÇÃO NACIONAL DE GEOGRAFIA E CADASTRO. It has its headquarters in the capital, Maputo. There is a provincial office in each provincial capital. There is a staff of 140 at the headquarters and 110 in the provincial offices.

The provincial offices, SPGC, are mainly working with real estate formation and registration.

Institutional Co-operation: How it all started

A Sida supported study in 1988 identified DINAGECA's need for training, modern surveying and mapping equipment, upgrading of provincial offices, and improved mapping and registration capabilities for land application and titles. It was proposed that emphasis be placed on rural land, specifically family sector land holdings and that a pilot project be implemented to develop methodologies for systematically issuing titles to all land parcels in a geographic location.

The programme had the following components:

1. Specialised training and education in surveying, mapping, management, English etc
2. Support for the Machava Training School in curriculum development, teacher training, materials and equipment
3. Upgrading of the central office of DINAGECA, and one or more provincial offices through training and provision of modern surveying and mapping technology, computers, and vehicles for field work
4. Development of a pilot project for cadastral surveys and titling of land to test various methodologies and to develop a national strategy for titling and land registration
5. Development of a national land information system
6. Introduction of new methods for topographic and orthophoto mapping
7. Provision of consultancies and equipment to support these activities.

DINAGECA, the National Directorate for Mapping and Cadastre, has received technical support since 1991 funded by Sida and implemented as an institutional co-operation project with Swedesurvey. The aim has been to rehabilitate and upgrade the services of Dinageca, to start developing a national land information system and to strengthen the capacity to carry out cadastral activities and topographic mapping. Long-term and short-term consultancies for the purpose of training, transfer of know-how and advisory services have been carried out involving some 15 consultants recruited from the National Land Survey of Sweden.

The introduction of modern methods and equipment has been a vital part of the co-operation, coupled with training of staff members in their use. Knowledge of computer technology for supplying information about land has also been improved considerably.

In the strategic plan for the development of DINAGECA during the next five years one important issue is the need for autonomy for Dinageca in order to improve its funding base and the conditions for staff members as well as improve the supervision of its provincial and district offices.

Experience

DINAGECA had very few qualified professionals employed at the beginning of the development cooperation in 1991. The few who were available were very much overloaded with work and had little time for planning and development. They mainly had to attend to daily problems of all kinds and try to solve them with the scarce resources available.

After the fall of the socialistic system various development projects were presented and different donor organizations made proposals for cooperation. All this also took a lot of the time of the few senior officers in the management.

At the same time a large number of officers were abroad for post-graduate training. The training was implemented in Russia, which was leftover from the Soviet era, where many officers were placed. A large number of officers were also trained in Portugal but in a number of cases they remained there for a very long time, up to ten years, which made their return doubtful. Later on a number of officers were trained in the U K.

In 1991 very few officers were able to speak English well enough to make use of all the training opportunities abroad. It was then that all members of staff recognised the importance of knowing English in order to seek further education abroad. The interest in learning at that time was enormous, everybody was training and studying.

Although staff were familiar with modern field surveying equipment to some extent as a result of training, in 1991 but there was no equipment available. During development cooperation equipment was provided and the field officers were trained in its use. They learnt very easily and quickly. It included total stations with automatic data logging and a full set of GPS equipment with post-processing features.

In 1991 there was only one PC available, an IBM PC AT. It had some technical software and a French DOS. From 1995 very rapid development has been implemented with regard to general and specific computerization in DINAGECA. A group of software and hardware technicians were trained and they developed advanced skills surprisingly quickly. A local area network was established and built up using their own resources and some external support. The skill of the technicians reached very high levels in a very short time. The network comprises some 40 workstations plus peripherals.

In the Sida supported development cooperation a land application handling system has been developed in a customized way by DINAGECA using local resources and with supervision and advice from external resources. The system is spread to half of the provincial offices. The work has been successful and the system will be distributed to all provincial offices and also to the bigger towns.

There has been a dramatic improvement in the number of professionals in the department and in the management. Under the development cooperation programme there has been some very successful training abroad which has resulted in promotions to senior posts in the department. But the number of senior officers is still insufficient in order to develop the department in parity with the increasing requests for its services. This will become very clear when the requests from the general public for more services according to the new land law come into effect. Other development cooperation projects draw, even furthermore, on the management resources in the department. As a result despite an increased number of senior professional officers the increased level of activities means that they are still under great pressure.

The 1998 national levelling report from Denmark

Sigvard Stampe Villadsen

National Survey and Cadastre, Denmark

Motorized levelling started in Denmark as a production tool back in 1979/1980. A production team with three cars were build up after Swedish model, and used in the detailed levelling lines as a testing field before the third precise levelling should start. Another team were build up in 1981. Every year smaller, needed changes were made in the cars and the equipment. Changes were made for three major reasons. Increase of production, elimination of error sources, and finally to make the cars a better place to stay for all the persons involved who had to use the cars as there working place 6 - 8 months every year.

At first data collection was done with simple multiplication machines in the cars, and data were stored in the main computer afterwards, simply by retyping all the numbers. In 1982/1983 a data collection program was made in HP-basic, and the same program and the important data structure was used with only smaller modifications in all the years until 1993. In 1993 the program were structured in the PC world, still keeping the same data structure.

In the years from 1982 to 1993 Denmark performed a third precise levelling, of which 85% of the levelling was done i the periode from 1986 - 1991. Except minor parts of the levelling lines, all the levellings were done with motorized geometric levelling, with invar staffs, Zeiss Ni002 compensator levelling instruments and HP85 used as data collection. The levelling was carried out fulfilling the NKGguidelines from March 14. 1984 including use of the "Pantalon rouge" method. The years 1982 1984 and 1985 were used for testing the methods and equipment, and only few lines were measured in these years. In 1982 special interest and care was taken about possible magnetic influence of the compensator instruments, so the north - south going lines were in this year measured with spirit levelling instruments. The magnetic influence on the Ni002 were tested every year in a Helmholz coil, and showed two important things. A very low value, between zero and 3/100 mm, and that the value for every specific instrument was constant over the years. The testing showed that there were no danger in using the Ni002 as main instruments in the levelling.

The main part of the third precise levelling was done from 1986 to 1992 with two motorized teams, running about 5 months every year. In 1993 and 1994 three minor supplementing lines was measured. Figure 1 shows the whole third precise levelling. In total the third precise levelling consist of approximately The precise levelling lines consist of about 7.700 benchmarks including about 1.100 precise fundamental benchmarks in a 3600 km levelling line.

The speed in the precise levelling is about 15 km pr. day, and the accuracy with the Ni002 and manual readings is about 0.6 mm*√km.

Parallel with the precise levelling, levellings were still carried out in the detailed levelling lines. A third levelling team were build up in 1985/1986, and this team were working full time in the detailed levelling lines, Also the precise levelling teams worked about 2 months every year in the detailed levelling lines, all three teams together covering the open land in parts of Mid-Jutland. Though the production in the detailed levelling lines were much to little to make much difference.

The most essential problem in the motorized geometric levelling with Ni002 is the production rate versus the precision. The precision in the levelling is in fact too good for detailed levelling. In only few places there are a need for so heigh precision, and there are only very few possibilities of getting lover precision and higher production. This fact was one of the major reasons for the Danish interest in the Swedish experiences with motorized trigonometric levelling. We adopted the general idea, but the main item in Denmark were not keeping the precision in the mountain area, but an increase of production. We were ready to accept a decrease in precision to a RMS about half a centimetre pr kilometre, if we could get a higher production. As it can be seen from the paper of Klaus Schmidt we succeeded.

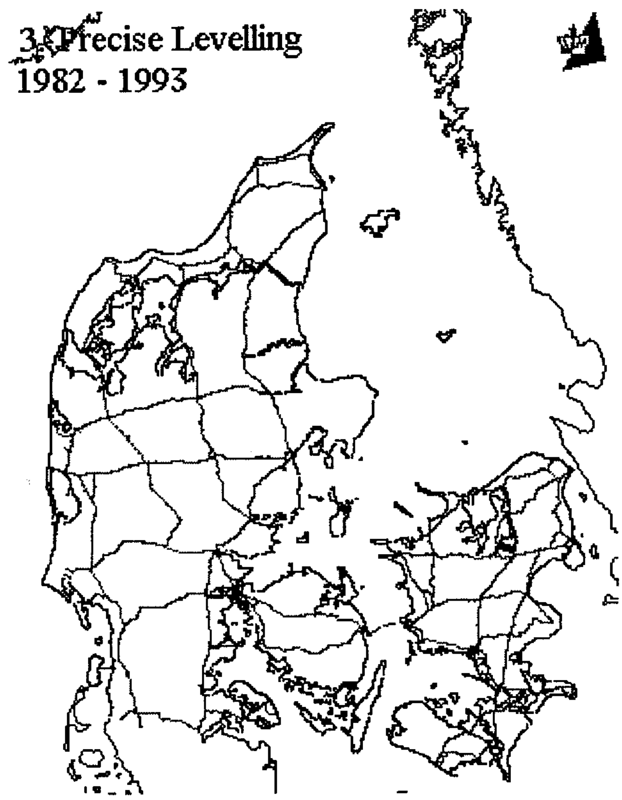


Figure 1. 3.Precise levelling in Denmark 1982 - 1993

Three years ago, we started investigating the use of automatic levelling equipment in the motorized levelling. The first attempt were made using Wild Na3003, but we had to drop the idea. The production were too often stopped or slowed down in bright wether conditions, where the contrast between the sky and the staff were to little for the instrument reading. The instrument were changed to a Zeiss Dini11, equipped with revolving eyepiece. Special bar code invar staffs was made ready for motorized use, supplied with an extra 0.5 m iron bar at the zero end, aiming to have the middle of the bar code at the same height as the instrument.

In the spring this equipment was brought into production with success, and one of the motorized teams has used it in the whole field season of 1998.

Until now the automatic levelling equipment has only been used in detailed levelling with other specifications than the levelling with the Ni002, so is not possible to make a direct compare of the two geometric methods. Daily production is around 20 - 25 km with a RMS around 1.2 mm*√km.

It seems that the instrument is not as stable as the old analog reading instruments. Instrument errors occurs more often than with the old instrument, but the benefit of the instrumentation are until now bigger than the disadvantages. The benefit comes from production rates, and most important, less stressed people involved in the levelling.

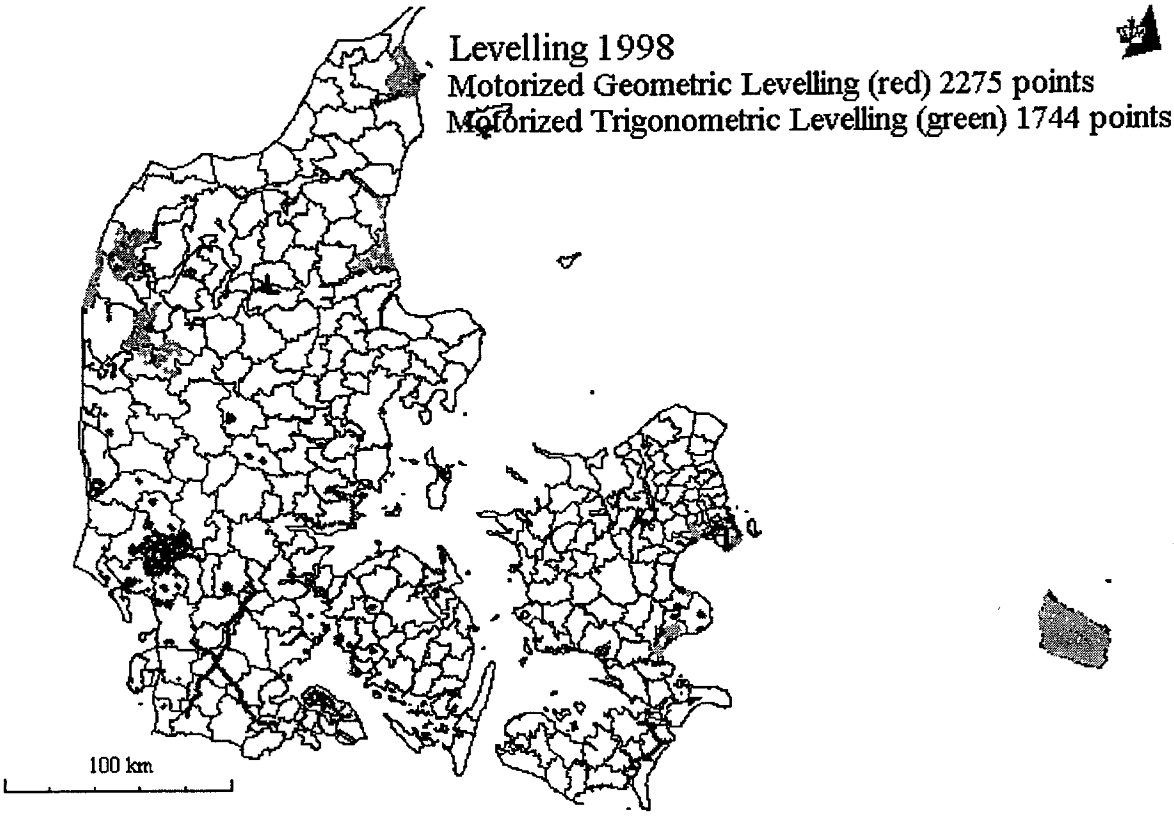


Figure 2: 1998 Levelling

In 1998 levelling has been carried out as seen in figure 2. In total around 4100 benchmarks has been levelled. The levelling is carried out in close cooperation with the users of the levelling points, who is also paying a good part of the total levelling cost. Users are mostly counties, municipalities, coast protection authorities, harbour owners and metrological office.

The increase in production from 1996 to 1998 is showed above

No. of Points levelled	1996	1997	1998
Geometric levelling	1667	1690	2275
Trigonometric levelling	1765	2359	1744
Indeks %	100	118	123

Indeks is calculated by the fact that the production in the trigonometric levelling is twice as productive as the geometric levelling

The Norwegian levelling activities

Björn Engen

Problems and expectations

Plans for height determination in Norway have been presented according to the requirements for a joint Nordic solution in cooperation with Sweden and Finland. In Southern Norway the connection to Sweden would strengthen both networks and in Northern Norway levelling lines from both Sweden and Finland could be connected to tide gauges in Norway.

The national plan presented in 1981 proposed 820 km of levelling per year in 15 years, while a plan from 1994 proposed 633 km per year in 9 years. The average result since 1981 however, is 350 km per year.

A new plan presented in 1998 proposes 550 km per year to 2003. There seems to be political support for the same total amount to be completed in year 2005. Further reductions are however indicated by the Government for the next years so the real result remains to be seen.

Norway has invested considerably in new space geodetic infrastructure. The question has been raised if such investments should be given priority before height determination. Most of the space geodetic projects in Norway are introduced and "sold" in competition with a lot of other both geodetic and mapping proposals. The selected projects therefore have funds earmarked for that particular activity and cannot be reallocated to other activities.

New techniques in levelling and an enthusiastic staff has improved the production considerably over the last few years. The Geodetic Institute will continue trying to find funds for an increased activity but will in that case need to purchase levelling services from the other Nordic countries.

The Swedish geodetic networks today and in the future

By Lars E Engberg, National Land Survey of Sweden

Background

In Sweden, the responsibility for geodetic control networks is divided between local authorities and the national authority. The main cause for this is mostly different aims. The responsibility for the national authority was to establish ground control for official mapping in small scales and the local authorities had to establish control networks for urban developments.

Some historical facts about local networks

The first control networks for municipalities were established in the beginning of this century. Most of them were in a very weak way connected to the national network prevailing at that time in Sweden. Since then control networks have been established in almost every urban area.

Nowadays we have 289 local authorities and almost every municipality has their own control network. Today, in some areas, there is more than one network because a forming of two or more into one municipality has taken place.

The situation is similar in character for both horizontal and vertical control networks. So we can, for excellent reasons, assume that we have at least 500 to 600 local control networks.

Some historical facts about national networks

Networks for vertical control

The first national precise levelling network was established for hundred years ago. Most of the benchmarks were situated along railways. The result of this levelling was The national height system 1900 (RH 00). This system or system derived from it is still in use.

The second precise levelling, started fifty years later and resulted in The national height system 1970 (RH 70). This system is the system of today and is used for national mapping.

Networks for horizontal control

In modern time, that means this century, we have carried out two national triangulations in Sweden.

One started in 1903 and was ended in 1951. The horizontal control system is called The national triangulation network 1938 (RT 38). This system is in some ways still in use because many local networks are derived from it.

The most modern triangulation took place between 1967 and 1982 and the horizontal control system is called The national triangulation network 1990 (RT 90). Associated to RT 90 are twelve regional subsystems, all of which were created for technical use.

Network for three dimensional control

For three-dimensional control we have a network of 21 permanent reference stations. The related tree dimensional system is SWEREF 93, the Swedish densification of the European reference frame EUREF 89.

Several networks to deal with

For historical reasons we have several geodetic control networks to deal with.

National

- ***two height systems***
- ***two plane systems***
- ***one 3-d system***

Local

- ***≈300 height systems***
- ***≈300 plane systems***

This will be a problem when all users will go into the "digital" world and everything will be "positioned by co-ordinates".

NLS responsibility

National Land Survey (NLS) is the national geodetic authority but has no power against municipalities and other authorities. NLS cannot dictate anything in this field just propose or give advise.

If we shall succeed to convince all users that common system a good thing we have to look in the future and try to describe the efficiency of new technique.

The future

We are testing GPS/GLONASS-technique, which can give us more useful satellites and better coverage for this positioning-technique. Another area is "national RTK-service", which maybe will cut down costs for some types of measurements also for local users. A pre-requisite condition for this is our reference network SWEPOS™. We may have to densify this network to get appropriate accuracy for the positions.

Network connections

RIX 95 is a national project, which aims at creating high quality connections between local, national and global reference frames.

With these connections we can study different transformation formulas and determine appropriate parameter-values. This project will also give us a mapping of RT 90 onto SWEREF 93 and that give us the opportunity to study local deformations of RT 90. This knowledge will be useful when we have to decide upon a new horizontal co-ordinate-system.

New situation

New techniques and new methods give rise to new type of control networks. Will the use of GPS-techniques imply that we have no need for dense control networks? How shall we handle heights in new three-dimensional reference frames?

These are questions that we have to deal with and there are no simple answers.

Up to now our control networks have been what we can characterise as "passive" networks. In the future we may have what we can call "active" networks. If we have passive networks the user normally doesn't pay for using them but if we have active networks the user will have to pay for them in some way. This situation is completely new an leads us beside technical problems also into financial problems.

The height problem or can we get accurate heights above the geoid from GPS-measurements? The situation in Sweden is lucky because within five years we will have a new height system. Our main problem in this area is to harmonise the new height system, the model of the geoid and the tree-dimensional reference frame.

Different solutions

The financial problem, which was mentioned above, may lead us into different solutions for urban and rural areas.

In urban areas, exclusive the inner cities, we can introduce RTK-techniques with permanent reference stations. In rural areas on the other hand it will be too expensive to establish permanent reference stations.

A possible solution for the future geodetic networks is some kind of active networks for urban areas, passive networks in the inner cities and in rural areas but of cause with different density.

In the future NLS has to learn more about end-user need

In the past the end-user never used the national network direct, there was always some densification network in between.

Nowadays, end-users will use this active network in building construction, road construction and railway construction. One example is machine-guidance on construction sites.

Therefore NLS has to learn more about end-user need otherwise we shall not be able to build the suitable geodetic networks in the future.

The past and the future of levelling networks in Hungary

György Busics

College for Surveying and Land Management

1-3. Pirosalma street, P.O.B. 52. 8002 Székesfehérvár, Hungary

Fax: +36 22 327697 E-mail: bgy@cslm.hu

Summary

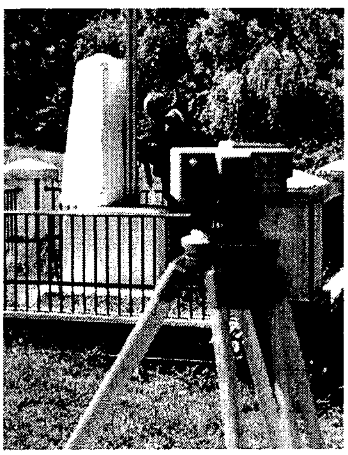
Because of its historical situation, Hungary has started to establish levelling networks four times up to now. The first network was built on the territory of the Austro-Hungarian Monarchy from 1873 until 1914. The second network was finished between the two world wars, but it was not adjusted and used at all. The current national levelling network – which is used on the half part of the country now, – was developed between 1948 and 1964. From the 1970s we started to develop the so-called Unified National Vertical Control Network (UNVCN), that is suitable also for crustal movement analysis, but the work has not been completed yet because of financial reasons. Only the first order network is ready for the whole territory of the country, the second and third order ones (which are necessary for practice) are finished for only the half of the country.

Using the strict regulations of precise levelling, and thanks to the appropriate adaptation of the new technology, Hungarian experts produced very good results.

Between 1995 and 1997 a new GPS network with 10 km long baselines was established. A question occurred, weather GPS technique can help to increase the density of the levelling network or not. There are research works concentrating on that how precisely the heights of the third order points can be determined with the knowledge of the GPS ellipsoidal heights and the geoid model. According to the initiatives the third order levelling network will be developed with GPS technique. The paper gives a report on the current status and introduces special Hungarian solutions.

The past: Levelling networks before the world war II

National levelling network I



The first nation-wide levelling network was established from 1872 in the member states of the Austro-Hungarian Monarchy. The campaign was organised by the Military Geographical Institute of Vienna. It meant that the work was managed and done by army officers, thus it is also called as “military levelling”. Seven fundamental benchmark points were built up in rocky surface, protected by obelisks.

Fig. 1. Digital level at the ancient Nadap point

One of these benchmarks can be found in the Velencei Mountains, close to the settlement Nadap. The height of this point was deduced in 1888 comparing to the middle water level of the Adriatic Sea. This point still exists, this is the reference point of every levelling network of Hungary. The stabilisation of the points was solved 3-4 kilometres far along the lines with bore-hole boards on the wall of the buildings. Other endpoints of the 2 km long (on average) sections were marked only with paintings on the horizontal surface of different constructions.

The measuring was quite long in every station, it lasted for 20-25 minutes. The applied levelling instruments can not be considered as precise ones in present-day sense. Furthermore, the relative inaccuracy of levelling can be explained with the fact that the wooden levelling staffs were not calibrated and sometimes many years passed between the to and back levellings.

Summary table of the four national precise levelling networks

	1st network	2nd network	3rd network	4th network
Time of establishment	1872-1914	1921-1939	1948-1964	1968- ...
Standard error	5 mm/√km	0.46 mm/√km	0.79 mm/√km	0.49 mm/√km
No. of 1st o. polygons	69	36	33	11
Length of 1st o. lines	18210 km	6285 km	6143 km	3900 km
Fundamental mark	1 (Nadap)	1	1+ 8	1+8+32
Benchmark type, marking	Bore-hole board	Rivet, stone, knob	Rivet, stone, knob	CMP point, Rivet, stone, knob
Lines running along...	Railways, roads	Railways, roads, embankments	Railways, roads	Only roads
Section length	1500-2500 m	1500 m	1200 m	1000 m (1500 m)
Instrument	Starke-Kammerer level	Oltay-Süss type level	Wild N3 bubble level	MOM NiA3 w. compensator
Levelling staffs	1 wooden staff + hanging staff	2 wooden staff	Double scale invar staff	Double scale invar staff
Sight length	60-80 m	Max. 50 m	Max. 40 m	Max. 35 m
Base plane	Iron board	Wooden peg+nail	Wooden peg+nail	Iron peg
Measuring time (observing+moving)	20-25 min	6-8 min	4-5 min	3.5-4 min

National levelling network II

After the World War I, new states were formed in this region and by the disorganisation of the Monarchy also the common land surveying organisation was closed down. The original sketches, notes and field books remained in Vienna. Among new borders, the establishment of the new levelling network of the country started in 1921. For these observations a new precise level was created according to the plans of Károly Oltay, professor of the Technical University Budapest, in the Süss Works. The 3 m long levelling staffs were made of wood again but they were calibrated twice a day. They were graduated in half cm divisions. The level and staff distance was maximised in 50 m, and the length of section in 1200 m.

Cast-iron fixing bolts marked the points placed in the buildings. On horizontal surfaces (e.g. on bridges) cast-iron knobs were placed. In the lack of suitable structures levelling stones or pillars with knobs were used.

The number of first order polygons was 36, within them second order lines were created.

The standard errors show that the network was one of the best at that time. Unfortunately, when the network became ready the World War II broke out. During the war 60% of the points were destroyed., thus it was not possible to use the network for practical purposes.

Present levelling networks

Nowadays we have two levelling networks, and consequently two height reference systems at the same time in Hungary. In the following a short description is given about their development and characteristics.

National levelling network III

A network used for rebuilding and technical practice was developed between 1948 and 1964. The aim was to establish at least one benchmark in every settlement. This aim was realised, thus approx. 23500 points were established, which means 1 point/4 km^2 density on average.

According to the hierarchic structure the national fundamental network consists of first-, second- and third order levelling lines and benchmarks. The first order network was created between 1948 and 1956, it has 33 polygons. The second order lines were developed between 1950 and 1958, the third order ones were measured between 1950 and 1964.

Eight so-called rocky benchmarks were established in mountain rocks. It means that each benchmark has 3 knobs in an underground cavity with two covers. All the other points are marked with rivet, knob or knob in stone, as it was before.

The measuring equipment changed significantly in comparison with the previous ones, WILD N3 precise level and invar staffs were used according to the rules of precise levelling.

During the establishment of the network, in 1960 it was decreed that in the Eastern European socialist countries Baltic sea level had to be used instead of Adriatic sea level. It meant that the height of Nadap point (and every point in Adriatic system) was decreased with 0.6747 meter. Naturally the conversion caused a lot of inconveniences.

National levelling network IV

From the middle of 1960s international geodetic organisations paid special attention to repeated levellings, from which the determination of vertical components of crustal movements was hoped. In Hungary a levelling network was planned and established as well, because the points of the existing network, exactly the marking were not suitable for this purpose.

New, underground marking methods were elaborated which exempt the movements of the points from the movements of the surface (soil water level change, frost effect), thus the points really represent the movements of the earth crust. These points, which are established for the investigation of crustal movements, are called CMP points (namely: CMP-Crustal Movement Point).

In the course of observing and marking the CMP points it became known that a great number of points were destroyed which was the result of constructions, re-building of roads etc. Only a part of these points could be replaced.

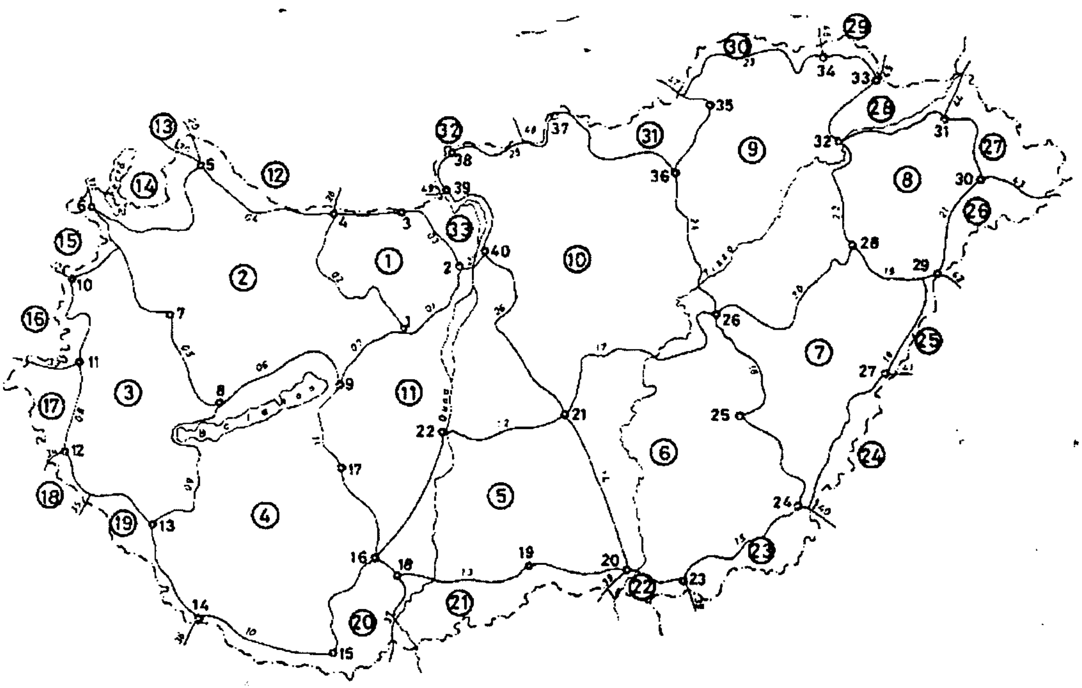


Figure 2. The new Hungarian 1^{st} order Levelling Network

Thus the idea of developing a new national network arose, which practically could be based on the crustal movement network. In that time the new Hungarian horizontal reference system (HD72) was introduced, too. In the frame of the modernisation programme of the geodetic basis of the country the concept of height reference system, namely the Unified National Vertical Control Network (UNVCN) was developed, as well.

The UNVCN is divided into first-, second- and third order networks, it also aims to ensure the point density of 1 point/4 km^2 on average. The first order network is the same as the crustal movement network, which was observed between 1973 and 1978. Within each first order polygon the development of second- and third order networks are usually made in one working procedure. Since 1980 this work is continuous, but unfortunately it goes ahead very slowly because of financial reasons; up to 1998 it was completed only on the 50% of the country.

The first order network of UNVCN consists of 11 polygons, these polygons are formed from 27 lines. The polygons are connected to the neighbouring countries with 22 side-lines. The total length of the first order lines is 3900 km, 90% of them is similar with some lines of the previous network, and only 10% is newly established.

From scientific point of view the most valuable parts of the UNVCN are the crustal movement points which can be fundamental benchmarks or CMPs.

Altogether the UNVCN has 40 fundamental benchmarks, 15 of them are placed on rock and 25 of them has special deep groundwork. The CMP points are situated 6 km far from each other on average along the first order lines, and 12 km far along the second order lines. Today their number is more than 800.

Experiences about the establishment of the UNVCN

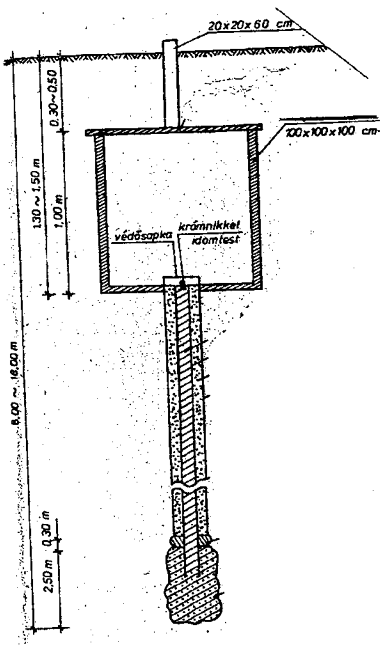


Fig. 3. Special fundamental benchmark

One of the specialities of the UNVCN is the use of crustal movement points, the special marking method. Separate standard design was developed for benchmarks placed on non-rocky surface and for CMP points.

In case of those benchmarks which are placed on non-rocky surface two 10-18 m long iron tubes mark the points, and between the two iron tubes ballast is stuffed in. It is presumable that the movement of the point is free from the surrounding soil mechanical influences if we use this marking method. For safety reasons more than one points of this kind were established on each site, two of them 40-60 km far from each other; these are called as twin points. Around the twin points further guard points were established. On the basis of observations it was proven that there are no movements in the relative situation between the two points, but surface points showed 2-4 mm subsidence as well.

Standard design was developed also for marking the CMP points (there are more than 800 of them). The benchmark itself is under the ground with 1 meter. It was a question to decide the most expedient diameter and depth of the concrete pile which includes the benchmark. In order to determine it 5 experimental CMP points of different size were established along two circles of 10 m radius, one of them was on sandy soil, the other one on clay. The movements of these points were measured for three years. From the results it was determined that the relative movements of the point groups to one another were negligible. It followed from the foregoing that the shortest and thinnest (4 m long and 20 cm diameter) concrete pile was selected for marking the CMP points. At the place of this experiment, at 1.7 m deep stones a spring-autumn "breathing" movement was found with the amplitude of 5 mm.

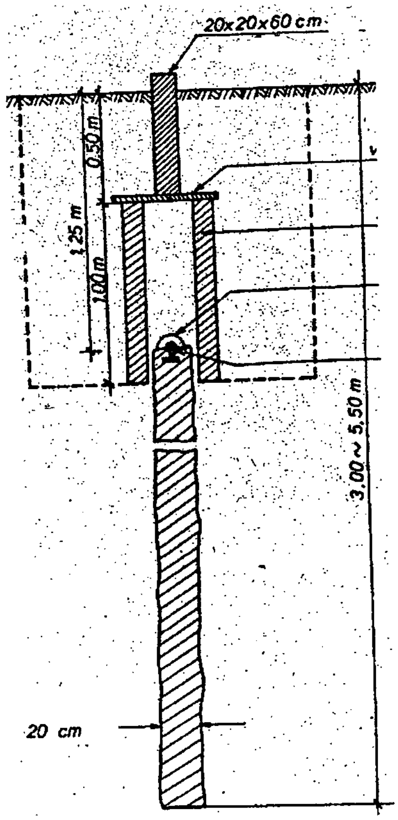


Fig. 4. CMP benchmark

Several experimental measurements were executed for investigating the technology of levelling and the method of observation, and the final rules were formed according to the results. Some of the rules used in Hungary and experiences are given in the following. The maximum instrument-staff distance was decreased to 35 m. After measuring three sections of a closed polygon of 2.2 km circumference several times the standard error was 0.37 mm at 35 m level-staff distance and 0.52 mm at 40 m level-staff distance. According to the experiences 25-31 m instrument-staff distance is the most advantageous.

- At investigating the most appropriate order of staff reading three methods were examined: (backsight-foresight-foresight-backsight) BBFF, BFFB and BBFF-FFBB orders. On the basis of simultaneous observations the following km standard errors were derived at these orders: 0.81 mm, 0.45 mm and 0.35 mm. In practice for saving time the BFFB order is used. The calculations and field data setting were made with Sharp PTA 4000 calculator, so it possible to control the data and repeat the observations immediately.

- In Hungary the influence of refraction was not taken into consideration, because experimental observations proved that it was not perceived in general conditions, and in case of temperature gradient smaller than 0.3°C/m.

- Since the 70s in Hungary almost only MOM NiA3 (NiA31) compensator level instruments were used for precise levelling, replacing the previously used bubble Wild N3 instrument. On the basis of experiments in general conditions the 1 km standard error was 0.3 mm with NiA3, and 0.4 mm with Wild N3. From the middle of 1990s digital levels and barcode staffs came out. At the measurement of the second and third order networks Leica Na 3003 level is used as well, where in the BFFB reading order each reading is composed of the arithmetic mean of values automatically. The measuring period did not change essentially, one level station is observed for 3.5 minutes on average.
- For decreasing the vibration sensitivity of compensator levels a special tripod was developed which legs were made of metal tubes filled with antifreeze liquid. In the liquid metal eases the vibration. For easing the hand trembling the micrometer screw is springy.

- The calibration of staffs was made before and after the observation season. The temperature was measured after 4-6 level stations, today it is measured in every station with digital thermometer. The change points were marked with iron pegs, but from the 80s the pegs were replaced with Hilti nails driven into the road surface.

- It was a separate organisational task to provide that another observer had to execute the foresight and backsight measurements in a different part of the day. This was a rule in case of first order observations. In second order network the same observer can measure the fore- and backsight observations but in the opposite parts of the day. By the growth of traffic more and more difficult to ensure safe circumstances for observations.

- The error limits in mm are: in first order network $1.2\sqrt{L}$, in second order network $2.0\sqrt{L}$, in third order network $3.0\sqrt{L}$, where L means the length of the section or line or polygon in km, depending on whether we speak about observation difference, height closing error or polygon closing error. In case of crustal movement polygons the closing error was always less than $0.9\sqrt{L}$.

Ideas about the future of the levelling network

Nowadays in Hungary the UNVCN serves the national economy only in the eastern part of the country because the network is completely developed only in this region. Making the second and third order networks denser lasted for 15 years on the Great Hungarian Plain because the work has become slower from 1989. It is natural that the political changes, the privatisation necessitate to speed up the preparation of digital maps instead of continuing the basic geodetic works. However good height basic data are necessary for highway, flood prevention and other construction works, and in case of several other users' demands. It means that these requirements can be satisfied only by relying on reliable vertical control networks. The necessity of re-measuring the national network is proven by the large movement of the points in the Plain. In the course of comparing the heights of the common points of the third and fourth order national networks it became obvious that the surroundings of Szeged and Debrecen subsided 10-15 cm as a result of oil mining and taking out of water. Such frame errors endanger the solution of practical tasks, too.

In accordance with the original idea in every 20-25 years re-measurement of the crustal movement network should be executed to show the movements. This period has already passed but we have no chance to think about the second measurement because the first one has not completely finished yet and knowing the present timing we have to wait for long years for the re-measurement.

Meanwhile in the 90s the GPS technology appeared which makes possible real spatial positioning instead of separate horizontal and vertical determination. As the reference plane of GPS measurements is the WGS84 ellipsoid, and for levellings is the geoid, it means that the "traditional" heights can be derived from GPS co-ordinates only with the knowledge of the geoid image. Though gravimetric geoid maps are available but the accuracy within several cm can be reached if we have common (GPS vertical) points, too. From this point of view it is also necessary to make the UNVCN denser. At the same time the question arises whether GPS as measurement technique can replace levelling or not. GPS measurement is much quicker and more economical than levelling. Several experimental observations were done concerning GPS levelling.

Fig. 5. Trimble antenna above benchmark

The height difference of approx. 70 control points along two first order lines of UNVCN coming from levelling and from GPS+geoid determination was less than 2 cm in case of the 90% of the points.

On a working area smaller than 15 km, using repeated GPS observations of the benchmarks it was possible to reach a correspondence within 1 cm between the known and GPS heights, also without using geoid model with applying simple plane fitting. This method requires at least 3 hours for observations.

Within a second order polygon of UNVCN the height of third order points could be determined within 5 mm by using GPS and geoid. It was possible to reach this accurate result because the length of GPS vectors did not exceed 20 km and the period time was at least 4 hours

Though there are continuous professional discussions about the future of levelling networks, the following standpoints seem to be clear:

1. It is necessary to have high accuracy, precise levelling networks because also nowadays this technology can be considered as the most precise one, and this meets the natural height conceptions which is based on geoid.
2. The re-measurement of the crustal movement network is explained from scientific point of view. This work would be expedient in continental sense, thus the elaboration of joint European recommendations would be useful to encourage the decision making bodies of each country to support such expensive measurements.
3. The further observation of the Hungarian second order network should be solved together with the repeated measurement of the first order polygon. According to the present practice the first order points are considered as given ones while they were defined 25 years ago, so their movement is possible.

In accordance with the plans the third order network will be measured with GPS instead of levelling for the sake of increasing rapidity and cost efficiency. In the future these third order points may serve as not only vertical control points but also as horizontal or GPS control points if their establishment is made carefully.

FIG Commission 5
Positioning and Measurement

Chair: Prof. Jean-Marie Becker, Sweden
v. chair: Mr. Matt Higgins, Australia
Secretary: Mr. Mikael Lilje, Sweden

FIG Commission 5 (Positioning and Measurement) activities concerns the science of measurement and acquisition of accurate, precise and reliable survey data related to the position, size and shape of natural and artificial features of the earth and its environment.

The mission of the Commission is to focus on modern technologies and technical developments and, through guidelines and recommendations, to assist individual surveyors to choose and utilise those methods in their daily work. The work of the commission is concentrated to several Working Groups (WG), working in collaboration with other organisations as IAG, ISO, ISPRS and different manufacturers.

The work of the Commission and its WG is presented and promoted on an on-going basis at FIG working weeks and other relevant technical meetings and in appropriate FIG and other media.

FIG C5 is organised with one Steering Committee and five WG, and several ad-hoc groups.

Steering Committee (SC)

Members of the SC are the chair, v. chair, secretary, WG chair and WG v.chair. The SC will meet minimum once a year.

The aim is to optimise and co-ordinate the work of the commission and to avoid duplication of work.

The SC shall supervise the finalisation of the task(s) of each WG

The SC is co-ordinating and organising the distribution of information by Newsletter, WWW or other suitable media.

WG-5.1 (Standards, Quality Assurance and Calibration)

Chair: Dr. Vaclav Slaboch, Czech Republic.

The task of the WG is to establish guidelines and recommendations for the checking and determination of the field accuracy of different types of instrument (e.g. total stations, laser planes and digital levels) and for the calibration of them.

The WG shall participate in the work of FIG-Task Force on Standards and other organisations as CLGE, CEN and ISO as well as other WG in Commission 5.

WG-5.2 (Height Determination Techniques)

Chair: Prof. Michel Kasser, France.

The task of the WG is to follow the technical developments concerning height determination techniques and instrumentations (e.g. GPS, total stations, digital levels and laser planes) and to prepare recommendations for making the best use of them to achieve expected accuracy in their different applications.

The WG shall collaborate with IAG as well as other WG in Commission 5.

WG-5.3 (Kinematic and Integrated Positioning)

Chair: Dr. Naser El-Sheimy, Canada.

The task of the WG is to report on performances (possibilities and limitations) of existing systems and recommendations for making the best use of them.

The WG shall collaborate with organisations as ISPRS and IAG as well as other WG in Commission 5.

WG-5.4 (Digital Mapping)

Chair: Nicolas Paparoditis, France

The task of the WG is to follow the technical developments concerning digital mapping techniques and to study the use of digital photogrammetry and its integration with other technologies such as geodetic surveying for practical applications

The WG shall collaborate with ISPRS and IAG as well as other WG in Commission 5.

WG-5.5 (Reference Frame in Practise)

Chair: Prof. Paul A Cross, United Kingdom

The task of the WG is to help the surveyor to fulfil his daily work at local and regional level by giving him recommendations/guidelines to handle all data such as co-ordinates and transformation parameters in a correct and efficient way.

The WG shall collaborate with IAG as well as other WG in Commission 5.

Contact addresses for officers in FIG Commission 5 (1998-2002)

Jean-Marie Becker	chair	fax: +46 26 61 07 76	email: jean-marie.becker@lm.se
Matt Higgins	v. chair	fax: +61 7 3891 5168	email: matt.higgins@dnr.qld.gov.au matt.higgins@uq.net.au
Mikael Lilje	secretary	fax: +46 26 61 06 76	email: mikael.lilje@lm.se

Working Group 1

Vaclav Slaboch	chair	fax: +420 2 685 7056	email: vaclav.slaboch@cuzk.cz
Hansbert Heister	v. chair	fax: +49 89 60 04-39 04	email: heister@glabse.bauv.unibw-muenchen.de

Working Group 2

Michel Kasser	chair	fax: (+33) 2 4343 3102	email: kasser@esgt.cnam.fr
Hilmar Ingensand	v. chair	fax: (+41) 1 633 11 01	email: ingensand@geod.ethz.ch

Working Group 3

Naser El-Sheimy	chair	fax: (+1) 403 284 1980	email: elsheimy@ensu.ucalgary.ca
Orhan Altan	v. chair	fax:	email: oaltan@srv.ins.itu.edu.tr
Yang Gao	v. chair	fax: (+1) 403 284 1980	email: gao@ensu.ucalgary.ca

Working Group 4

Nicolas Paparoditis	chair	fax: (+33) 1 43 98 85 81	email: nicolas.paparoditis@ign.fr

Working Group 5

Paul Cross	chair	fax: +44 171 380 0453	email: paul.cross@ge.ucl.ac.uk
Matt Higgins	v. chair	fax: +61 7 3891 5168	email: matt.higgins@dnr.qld.gov.au matt.higgins@uq.net.au