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PROCEEDINGS OF THE 13th GENERAL MEETING OF THE NORDIC GEODETIC COMMISSION

PART 1

GÄVLE, SWEDEN
25 - 29 MAY 1998

edited by BO JONSSON

Gävle, Sweden
1999

LANTMÄTERIET

Introduction

The thirteenth General meeting of the Nordic Geodetic Commission took place at National Land Survey, Lantmäteriet in Gävle during the time period 25 - 29 May 1998 with representatives from Denmark, Finland, Norway, Sweden, Estonia, Latvia and Lithuania.

The Geodetic Commission of Estonia, Latvia and Lithuania submitted to the Nordic Geodetic Commission an application for inclusion in the work of the Nordic Geodetic Commission as an observer or as a corresponding member.

The Local Organizing Committee consisted of Bo Jonsson, chairman, Hanne Bothén, Lars E. Engberg, Lars Åke Haller, Lotti Jivall, Lantmäteriet and Jan Johansson, Onsala Space Observatory.

The participants were welcomed to Gävle and Lantmäteriet by Joakim Ollén, Director General of Lantmäteriet. Juhani Kakkuri, the chairman of the NKG Presidium, opened the General Meeting.

The program for the General Meeting comprised 7 working sessions with presentations of 59 papers:

- National and working Group reports on Monday, followed by a technical tour at Lantmäteriet and an Ice breaker Party in the evening.
- Two Working sessions on Tuesday followed by the evening lecture "The role of Geodesy in the future" by Juhani Kakkuri and a pub evening at the canteen of Lantmäteriet.
- Two Working sessions on Wednesday followed by an excursion to Falun copper mine
- Two working sessions on Tuesday followed by the traditional NKG dinner in the magnificent mirror hall of the city hall of Gävle.
- One Working session and the Closing session on Friday.

A Resolution Committee was organized, consisting of:

- Björn Geirr Harsson, Norway, chairman
- Sigvard Stampe Villadsen, Denmark
- Martin Vermeer, Finland
- Martin Ekman, Sweden

The atmosphere during the General Meeting was very warm and the NKG community is like a large family. The General Meetings are important opportunities for the young geodesists to present their findings and to create a network of contacts for their future activities in the field of Geodesy. The warm and sunny week in Gävle in the early part of the summer offered numerous opportunities to meet in the city centre of Gävle over a beer or a cup of coffee.

Gävle, May 1999
Bo Jonsson
Chairman of the Local Organizing Committee

The 13th NKG General Meeting
Gävle, Sweden

Contents

Program for the 13th General Meeting of the Nordic Geodetic Commission

Monday	**25 May**	
13.00	Welcome address	Joakim Ollén, General Director, National Land Survey of Sweden
	Opening address	Juhani Kakkuri, Chairman of the NKG Presidium
Session 1,	**National and Working Group reports** **Session chair: Juhani Kakkuri**	
13.30	National reports	
14.45	Coffee break	
15.15 - 16.30	Report from the Working Groups; Height Determination, Geodynamics, Satellite Geodesy, Permanent Geodetic Stations, Geoid Determination	
18.00	A technical tour at the National Land Survey followed by Ice breaker party	
Tuesday	**26 May**	
Session 2,	**Height Determination** **Session chair: Jean-Marie Becker**	
08.30	Location of possible gross errors by use of GPS in an open levelling line	John Sundsby
	Accuracy of GPS levelling	Matti Ollikainen
	GPS Antenna and Site Effects	Jan M. Johansson
	Automated calibrations of levelling rods in Finland	Mikko Takalo
09.50	**Coffee break**	
10.20	Results of the Baltic Sea Level 1997 GPS Campaign	Markku Poutanen

	Using mean sea surface topography for determination of height system differences across the Baltic Sea	Martin Ekman
	VREF 1996 - a new height reference system for Norway	Dag Solheim
	Implementation of a new height datum in Denmark	Sigvard Stampe Villadsen
	Towards the unification of vertical datums in the Baltic Sea region	Markku Poutanen
12.00	**Lunch**	
Session 3,	**Reference Networks** **Session chair: Per Knudsen**	
13..00	The Geodetic Reference Networks in Latvia - status report	J. Kaminskis and N. Abols
	The New Norwegian National Geodetic Network - EUREF89	Björn Geirr Harsson
	Densification of the EUREF 89 network in Finland	Hannu Koivula, Matti Ollikainen and Markku Poutanen
	The renovation of the Cadastral control-network in Denmark	Sören West-Nielsen and Sigvard Stampe Viladsen
14.20	**Coffee break**	
14.50	A New Three dimensional Reference Network in Denmark	Anna B. O. Jensen
	Realization of a new Reference Network in Greenland	Finn Bo Madsen
	Testing of a priory mean errors	Karsten Engsager
	Improving a Horizontal Datum Without Changing the Coordinates	Bo-Gunnar Reit
- 16.30	Geodetic Aspects of the Law Of the Sea - GALOS	Björn Geirr Harsson

18.00	**Evening lecture:** The role of geodesy in the future	Juhani Kakkuri
19.00	**Pub evening** at the canteen of the National Land Survey	
Wednesd	**27 May**	
Session 4,	**Geoid and Gravity** **Session chair: René Forsberg**	
08.30	The NKG -96 geoid	René Forsberg
	New gravity calibration line of the Finnish Geodetic Institute	Hannu Ruotsalainen, Jaakko Mäkinen and Jussi Kääriäinen
	Gravity and GPS measurements in Greenland	Frans Rubeck and René Forsberg
	Airborne gravimetry in Skagerrak and the Fram Strait - and possibilities of future joint Nordic aerogravity	René Forsberg et al.
09.50	**Coffee Break**	
10.20	The gravity spectrum observed by superconducting gravimeter at the Metsähovi station, Finland	Heikki Virtanen and Jussi Kääriäinen
	Geodetic contribution to a Global Geophysical Observing System (GGOS)	Hans-Peter Plag
	A project proposal: CRUSLAC	Hans-Peter Plag
Session 5,	**The future of NKG** **Session chair: Bo Jonsson**	
11.20	The future of NKG, General discussion	
12.00	**Lunch**	
13.00 - 18.00	**Excursion** to Falun copper mine	

Thursday 28 May

Session 6, Permanent GPS stations
Session chair: Jan M. Johansson

08.30	The Finnish Permanent GPS Network - Finn Net	Hannu Koivula, Matti Ollikainen and Markku Poutanen
	Quality control in networks of permanent reference stations	Gunnar Hedling
	Activities at the NKG GPS-data Analysis center 1996-98	Jan Johansson
	Problems regarding the Estimation of Tropospheric Parameters in connection with the Determination of New Points in SWEREF 93	Jonas Ågren
09.50	**Coffee Break**	
10.20	The ionospheric problem in GPS phase ambiguity resolution and some possible solutions	Lars E. Sjöberg
	Coordinate variations at the Norwegian permanent GPS sites	Oddgeir Kristiansen and Hans-Peter Plag
	The first results of the Finnish permanent GPS network	Hannu Koivula
	Monitoring of atmospheric water vapor using ground-based GPS receivers	T. Ragne Emardsson, Gunnar Elgered and Jan M. Johansson
	About GPS, Modems and the meaning of life	Martin Vermeer
12.00	**Lunch**	

Session 7, Geodynamics
session chair: Martin Vermer

13.00	Computation of land uplift from the three precise levellings in Finland	Jaakko Mäkinen and Veikko Saaranen

	Vertical secular movements in Denmark from repeated levellings and sea-level observations during the last 100 years	Klaus Scmidt
	BIFROST project: Refined vertical rate estimates for Fennoscandian postglacial rebound	Hans-Georg Scherneck et al
	On postglacial uplift rates for reducing vertical positions in geodetic reference systems	Martin Ekman
14.20	**Coffee Break**	
14.50	Marine Gravity and Sea Level Variations from Satellite Altimetry - Altimeter Data Processing 1994- 1998	Per Knudsen Ole Baltazar Andersen
	An example of the impact of winter climate on interannual sea level variations	Martin Ekman
	Tidal gravity measurements at 79^{o} North on Svalbard	Knut Röthing
	Surface Loading Models - Recent Developments	Hans-Georg. Scherneck et al
-16.30	Crustal motion in Europe determined with geodetic VLBI	Ruediger Haas
19.00	**NKG Dinner**	
Friday	**29 May**	
Session 8,	**Special Topics** **Session chair: Björn Geirr Harsson**	
08.30	Very high precision distance measurements of the FGI	Jorma Jokela
	New Satellite Laser system of Metsähovi - status report	Matti Paunonen
	GeoDisp: a PC program for visualisation of geodetic data.	Jon Olsen
	An integrated car navigation system used for establishing and updating a road data base	Lars Bockman

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The Danish National Report 1995-98
General Meeting
of
Nordic Geodetic Commission

By S. Stampe Villadsen, Kort- og Matrikelstyrelsen, Denmark

Introduction

A new organizational structure introduced in 1992 for KMS has been changed during the past 4-year period in order to facilitate a market-oriented process within the mapping geodata sector in Denmark. Furthermore, the new organisation should support the more and more intensive use of information technology in the production of maps and charts.

The new organisation was formed by January 1, 1997 with the following four units:

1. Economy, Personel and Planning.
2. Land Registration and Topography.
3. Geodesy and Cartography.
4. Market and Informatics.

The new organisation has now been in place for more than one year and so far it seems to fulfil its task. The total number of employés are 535, of which app. 50 are working in the field of geodesy. KMS is situated in a group of buildings at the adress Rentemestervej 8, Copenhagen NV.

After the first years under the new structure it is most satisfying to conclude that the formal integration of pure and applied geodesy has proved to be operational both in terms of income and scientific activities.

The contents of this national report demonstrate that the development of modern geodesy has continued during the four year period. This will be treated in more details in the following parts of the report, but already here it will noticed, that applications of modern technology has played an important role resulting in constuction of new geodetic reference systems. Furthermore an increasing focus on global aspects, geophysical, and environmental applications is foreseen in the future.

Geodetic Reference Networks

Horizontal networks.
The introduction of ETRS89 (WGS84) has been completed by introduction of GPS-observations in app. 100 points covering Denmark (REFDK). The observations are performed in old triangulations points of first and second order, and transformations parameters between ETRS89 and the old reference systems have been established.

The trigonometric network, consisting of appox. 21.400 existing points is maintained in accordence with agreements with the national defence. All stations are visited at 10 repeated intervals, and new descriptions and sketches are made. No new 2D stations are made as replacements for lost stations, but new 3D stations are introduced when necessary.

A total revision of the cadastral control network (MV-network) is finished after 12 years work at the end of 1997. The revision is done by connectecting this network to the reference network of trigonometric stations using the observations produced in connection with the cadastral changes and by introducing new measurements performed in 1-2% of the control points. The total amount of revised points is 330.000. The MV-network is used as the base for reconstruction of the digital cadastral maps now covering the whole area of Denmark.

A new database for handling geodetic observations, coordinates, station descriptions and sketches are in full service.The database are build to provide better services to users of geodetic information, including internet acces to all kind of coordinate information. In close connection with the database, most of the geodetic software used for adjustment, transformation and other programme systems necessary to handle all the data are transformed into a UNIX environment using the edp-programming language, C. The programme system includes now processing of GPS-vectors in connection with classical geodetic observations.

Vertical networks.
The third precise levelling measured by motorized geometrical levelling, mainly performed from 1986 - 1992, is now completed and adjusted. The result is a new vertical datum called DNN KMS90. The height is adjusted as ortometric heights, tidally corrected, and referenced to mean sea level at 10 tide gauge stations. Since 1992 about 20 of the nodal point in the precise levelling network are measured in a high precision GPS campaign. About 30 more of the nodal points are scheduled for observation during the coming 2 years. The idea is, that these 50 points will be monitored by GPS regularly, and in connection with the tide gauge stations support the height systems in Denmark during the next decades..

The precise levelling is now being densified in a regional network and furthermore new measurements are performed in cooperation with the municipalities in the local networks in order to support the infrastructure in local areas.The observations are collected both by motorized geometrical levelling with or without automatic registration, motorized trigonometric levelling. Since 1992 the local networks in 60 of Denmarks 275 municipalities have been re-measured.

A new geoid model based on the Nordic geoid is adjusted to the old vertical reference systems giving a possibility to determine heights with an accuracy of 1-2 cm in major parts of Denmark.

Gravity network.
During the period the national reference gravity network, based on absolute gravity measurements and numerous relative gravimeter ties, was computed and published. Most of the gravity stations have additionally been levelled.

Permanent GPS stations and dGPS services.
2 permanent GPS reference stations in Denmark has been established, and a third

-"-	Kemijärvi-Joutsijärvi	
-"-	Joutsijärvi-Maaninkavaara	
-"-	Laurila-Tornio	
-"-	Kemi-Ahjos	415
1997	Lapua-Viitasaari	
-"-	Haapajärvi-Vaala	
-"-	Tornio-Vuennonkoski	
-"-	Vuennonkoski-Aavasaksa	
-"-	Aavasaksa-Vietonen	
-"-	Rovaniemi-Sinettä	506

5. Gravimetry

5.1 Densifications

After completion of the National Gravity Net at a spacing of 5 x 5 km², densification of the net was initiated in 1980 at an average spacing of two stations per square kilometer. In the period 1994-1998 the following densification measurements with LCR-gravimeters were performed:

Year	Number of points	Measurement area or 1:20 000 map charts
1994	186+355	Bothnian Sea + charts 21 23 07 and 21 23 08
1995	816	Charts 12 42 06, 12 42 07, 12 42 08, 12 42 09, 12 42 11 and 12 42 12
1996	117+802	Bothnian Sea + charts 12 41 06, 12 41 09, 12 41 12, 12 42 10 and 12 42 11
1997	928	Alavieska, Kalajoki, Merijärvi and Oulainen

In addition, at Bothnian Sea total amount of 3 200 gravity points were measured in 1996 by Håkon Mossby ship with sea gravimeter.

5.2 Absolute gravity

The absolute gravimeter JILAg5 of the Finnish Geodetic Institute was mostly used for international gravity measurements. The following measurements have been made:

Year	Country
1994	Finland, Belgium, Spain, South Africa, Antarctica, Lithuania, Portugal (Azores), France and Germany
1995	Finland, Belgium, Germany, Latvia, Poland and Estonia
1996	Finland
1997	Finland, France, Germany, Portugal (Azores), Spain and Island

6. Geodynamics

6.1 Horizontal deformation

Horizontal crustal deformation studies were continued through analysing the data from the permanent GPS stations. Analyses performed showed that, in general, the intraplate deformation is going on as revealed by earlier analyses of the repeated terrestrial triangulations the maximum horizontal compression being in the NW-SE direction.

6.2 Earth Tides

In August 1994 the superconducting gravimeter GWR20 was installed in the Metsähovi observatory for the Earth Tides observations, and from that time on, regular recordings have been performed.

The recordings of the Earth Tides were continued at the Lohja site with very long water tube tiltmeters in 1994-1998.

7. Published works

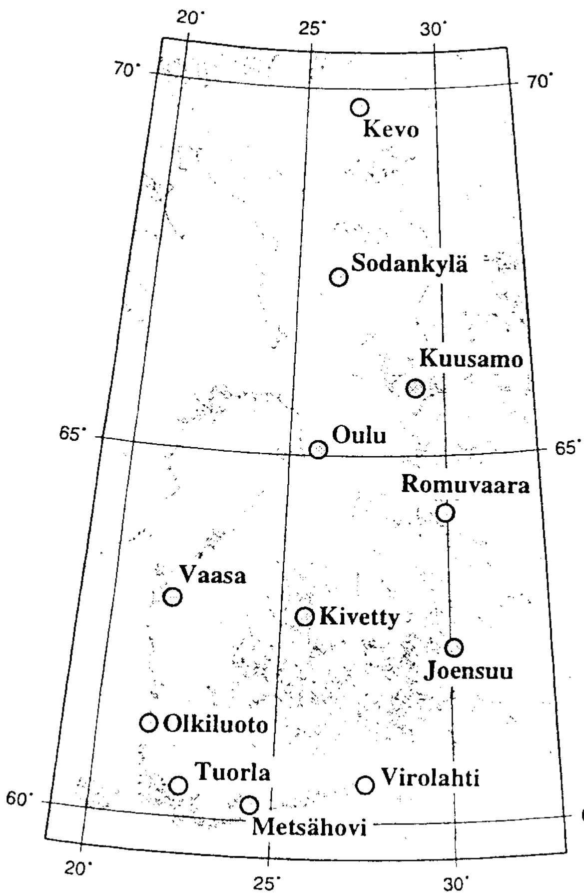
The following works were published in the series of the Finnish Geodetic Institute.

7.1 Publications

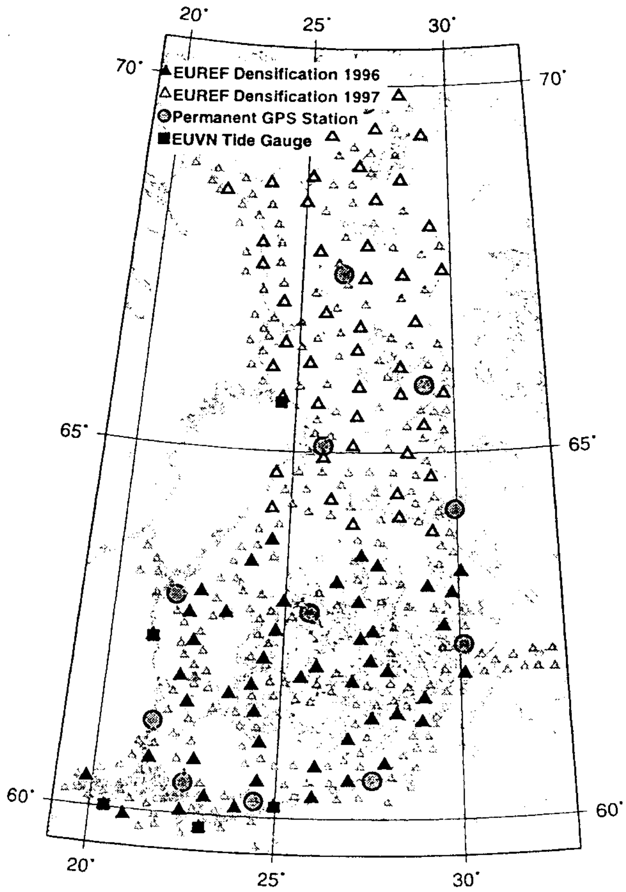
a) Konttinen, R. (1994a). Observation Results. Geodimeter Observations in 1971-72, 1974-80, and 1984-85. Publ. Finn. Geod. Inst. 117.
b) Konttinen, R. (1994b). Observation Results. Angle Measurements in 1964-65, 1971, 1984, and 1986-87. Publ. Finn. Geod. Inst. 118.
c) Jokela, J. (1994). The 1993 adjustment of the Finnish First-Order Triangulation. Publ. Finn. Geod. Inst. 119.
d) Poutanen, M. (1995). Interference Measurements of the Taoyuan Standard Baseline. Publ. Finn. Geod. Inst. 120.
e) Jokela, J. (1996). Interference measurements of the Chang Yang Standard Baseline in 1994. Publ. Finn. Geod. Inst. 121.
f) Jaakkola, O. (1996). Quality and automatic generalization of land cover data. Publ. Finn. Geod. Inst. 122.
g) Ollikainen, M. (1997). Determination of Orthometric Heights using GPS Levelling. Publ. Finn. Geod. Inst. 123.
h) Kilpeläinen, T. (1997). Multiple Representation and Generalization of Geo-Databases for Topographic Maps. Publ. Finn. Geod. Inst. 124.
i) Kääriäinen, J. and Mäkinen, J. (1997). The 1979-1996 Gravity Survey and results of the Gravity Survey in Finland 1945-1996. Publ. Finn. Geod. Inst. 125.

7.2 Reports

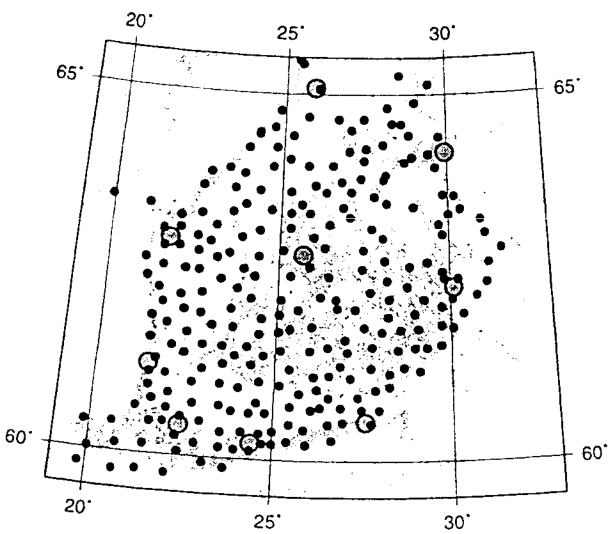
a) Vermeer, M. (1994). A fast delivery GPS gravimetric geoid for Estonia. Rep. Finn. Geod. Inst. 94.1.



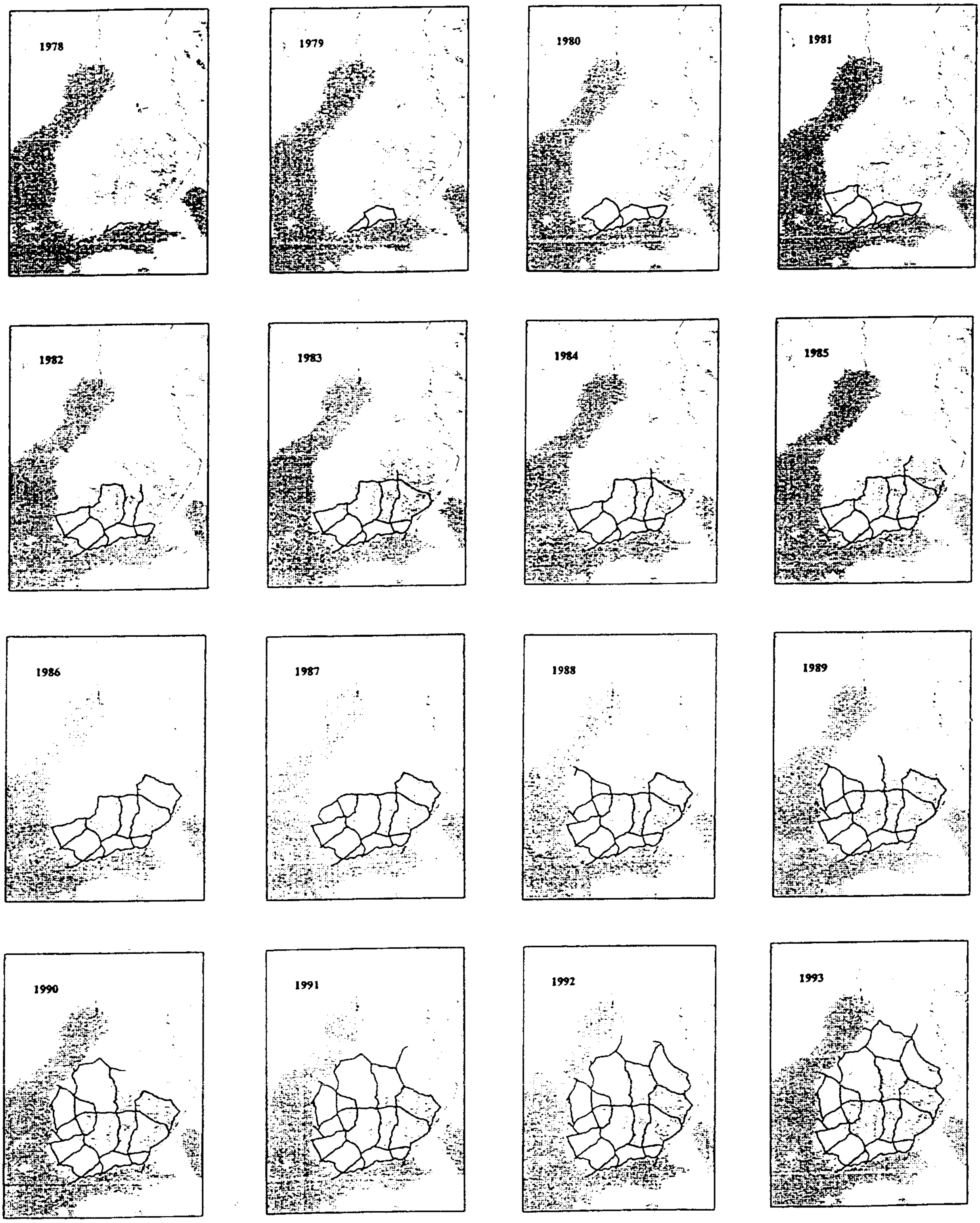
Finnish permanent GPS network.

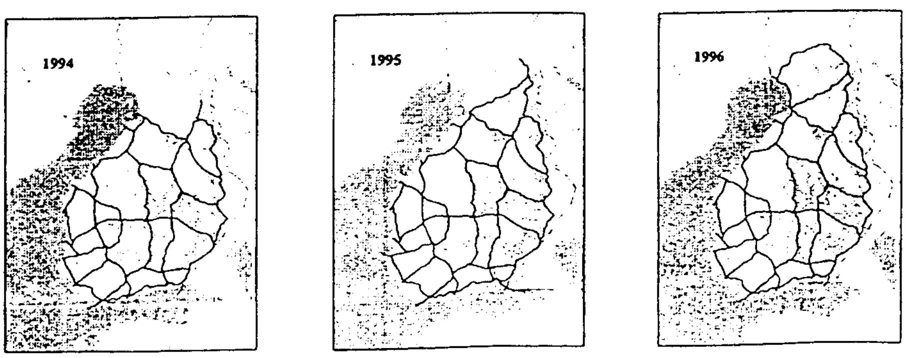


EUREF densification points.



Measurement plan for the user points, to be measured in 1998.





SUOMEN KOLMAS TARKKAVAAITUS

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MITTAUKSEN EDISTYMINEN VUOSINA 1978 - 1996

GEODETIC OPERATIONS OF THE NATIONAL LAND SURVEY OF FINLAND DURING 1994 - 1997

by Marko Ollikainen

The Control Surveys Unit of the Geographic Data Centre of the National Land Survey (NLS) and Oulu Survey Office aims to establish both horizontal and vertical control networks covering the whole country. These control networks should be dense enough for all kind of mapping purposes. The horizontal control network is based on the 1st order triangulation network measured by Finnish Geodetic Institute (FGI). The vertical control network is based on the precise levellings made by FGI.

The density of control point networks is good in the southern part of Finland. However, in Lappland and in the eastern part of Finland more control points are needed. The use of GPS in aerial photography has decreased the need for ground control points. This will reduce the amount of measured points in the future. In the time span of 1994 - 1997 this development can't be yet seen because of some other projects.

In the near future NLS is undergoing a process of reorganization. The new organization will come into effect at the beginning of 1999. The intention is to have District Survey Offices which are big enough to perform the basic duties of the NLS appropriately within their own area of operation. In this process District Survey Offices are combined into bigger units, there will be 13 of them left out of the existing 21 after the reorganization. The existing seven national operational units will be reorganizated, too. The Control Surveys Unit will become a part of Helsinki Survey Office in this process. However, tasks and area of operation of the unit will remain as the same.

The need for a uniform geographic information system in Europe and wider application of satellite measurement technology calls for increasing use of uniform European coordinate system in general mapping services. During the planning period 1998 - 2001 preparations will be made for unifying the national horizontal and vertical networks in line with EU targets. As a part of this process a workgroup has been established in order to investigate advantages and disadvantages of the reform of the national coordinate system. The workgroup is to finish its work by the end of this year.

Horizontal Control

Amounts of stations which have been measured during 1994 - 1997 by using the static relative GPS method can be seen in table 1.

Table 1. GPS stations measured by NLS during 1994 - 1997

Year	Control Surveys Unit	Oulu Survey Office	Total
1994	607	328	935
1995	532	326	858
1996	579	371	950
1997	597	410	1007
Total	2315	1435	3750

The number and types of GPS receivers used in the field work are presented in table 2. During 1994 - 1997 there has been 10 full time employees and 7 part time employees (in the field season) involved in horizontal control.

Control Surveys Unit maintains a database for horizontal and vertical control points. At this moment there are about 22500 horizontal control points stored in that database.

Table 2. GPS receivers used by NLS in 1994 - 1997.

Year	Control Surveys Unit	Oulu Survey Office
1994	6 Ashtech XII	6 Ashtech L-XII
1995	6 Ashtech XII	6 Ashtech L-XII
1996	2 Ashtech Z-12, 4 Ashtech XII	6 Ashtech L-XII
1997	6 Ashtech Z-12	4 Ashtech Z-12, 2 Ashtech XII

Vertical Control

The amount of levelled benchmarks and the total lenghts of the levelling lines are presented in table 3. Most of the levellings are performed by Control Surveys Unit. The Oulu Survey Office has levelled approximately 200 III order benchmarks (approx. 400 km) yearly. Alltogether 3 Wild NA3000 digital levels and GPCL3 invar bar staff pairs are used in this work. Three full time and six part time employees have taken part in the levelling work during 1994 - 1997. In the summers of 1996 and 1997 two levelling groups were used instead of the former three groups.

Table 3. The levellings completed by NLS during 1994 - 1997

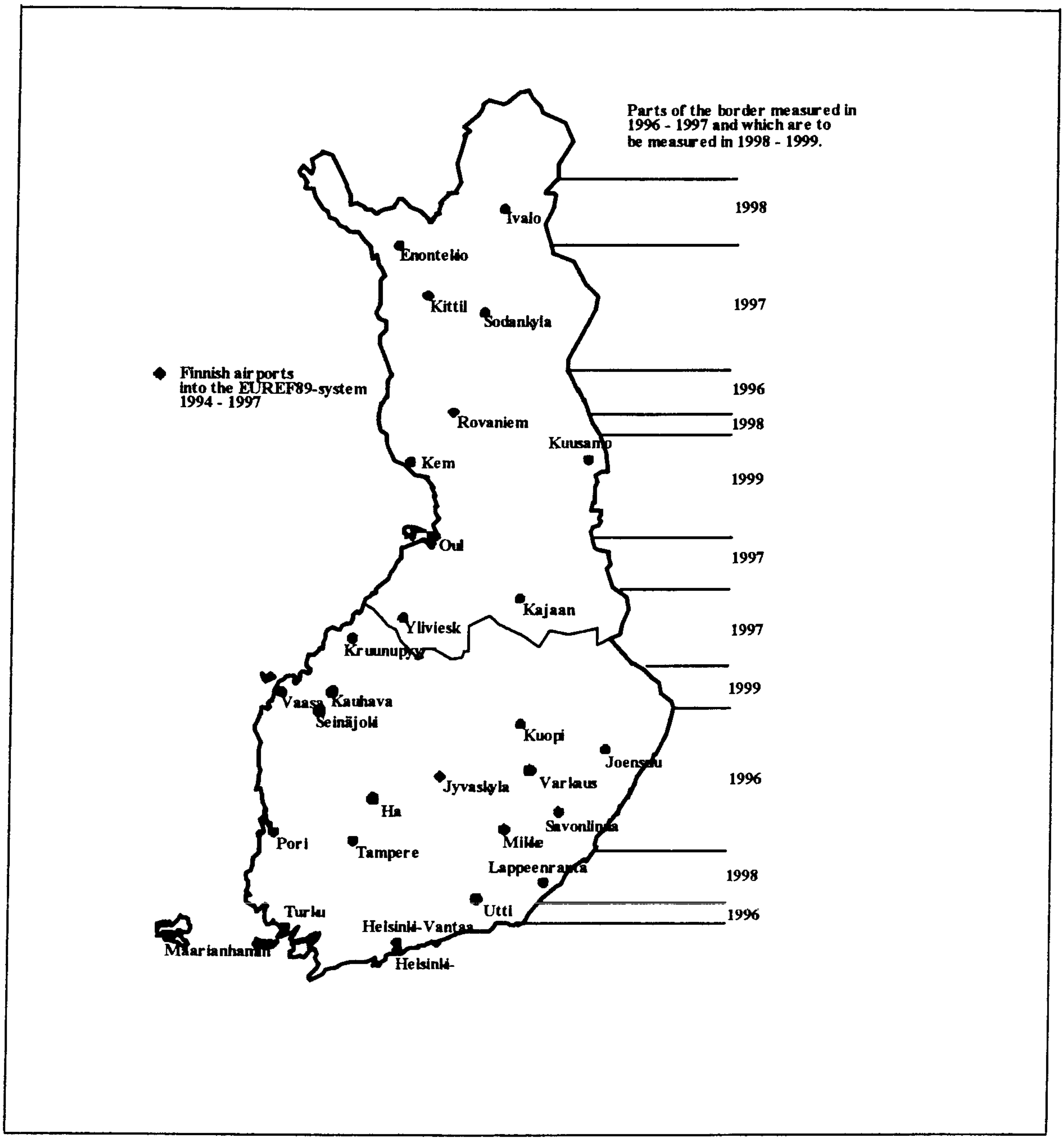
Year	Lenght of the lines in km				Number of benchmarks			
	Ib	II	III order	Total	Ib	II	III order	Total
1994	190	-	1539	1729	143	-	981	1124
1995	342	24	1304	1670	280	17	835	1132
1996	-	42	1084	1126	-	30	745	775
1997	-	67	1168	1235	-	56	739	795
Total	532	133	5095	5760	423	103	3300	3826

Ib and II order levellings are double-run levellings and III order levellings are single-run levellings. Ib and II order levellings were done in order to densify precise levelling network. The further densification (III order) is concentrated into the regions of 1:5000 scale base map. Nowadays the double-run levellings are mainly done to relevel old, partly destroyed lines.

There are about 49000 vertical control points stored in the database.

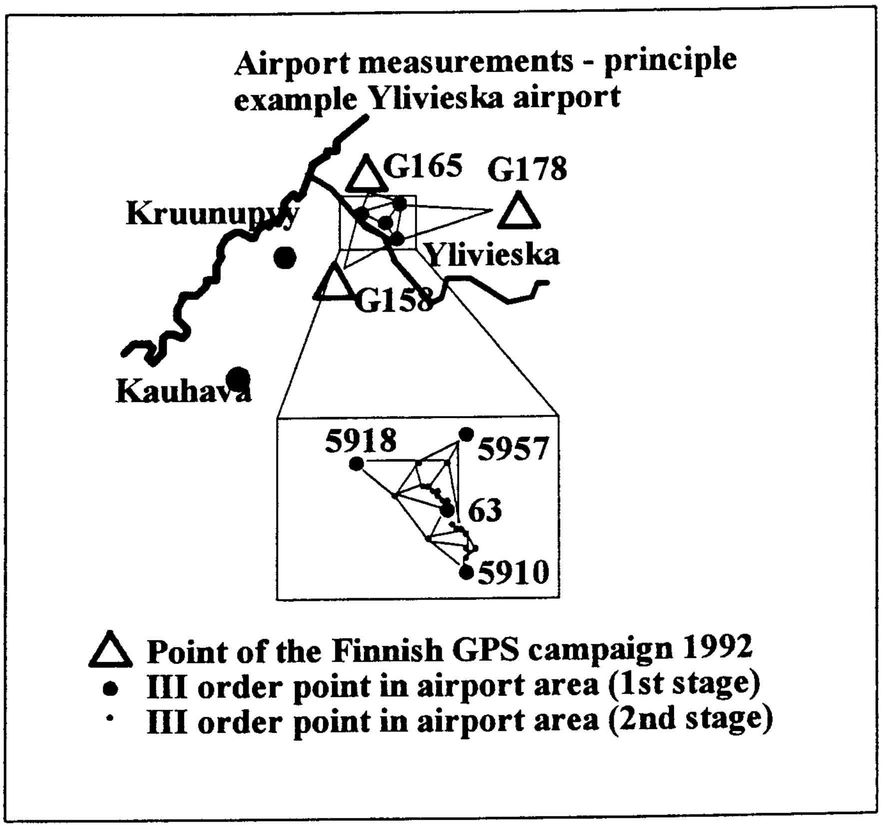
Geodetic measurements in airfields

During 1994 - 1997 Control Surveys Unit measured and transformed Finnish airfields into the EUREF-89 system. In 1994 10 airfields, in 1995 7 airfields, in 1996 10 airfields and in 1997 2 airfields were measured to the Civil Aviation Administration of Finland. Those airfields are presented in picture 1. The work included measurement of III order control network around every airfield area, measurement of IV order control points and instrument approach facility and obstacle survey.



Picture 1. Airfields measured in 1994 - 1997 and measurements in the boundary between Finland and Russia in 1996 - 1999.

The work was completed in the following way. As a EUREF-89 reference the coordinates of the stations measured in the Finnish GPS Campaign 1992 were used [Ollikainen Matti, 1994, Proceedinds of the 12th General Meeting of the Nordic Geodetic Commission]. Adjustements and measurements were done in two parts. First, III order control network was measured using static relative GPS method. At least one but usually three EUREF-89 stations were used as fixed point in the network adjustment where 3 - 4 stations were measured around the airfield and adjusted (1st stage). After that the rest of the control network was measured using those 3 - 4 stations as fixed points (2nd stage) An example is presented in picture 2.



Picture 2. An example of airfield measurement.

Other control points (IV order) were measured using kinematic method in open airfield area and static method in wooded and difficultly accessible areas. Surveys of instruments, thresholds, buildings, runway paintings, obstacles etc. were done afterwards by another group. In those surveys electronic tachymeter was used. Approximately 200 survey points were measured in every airport.

Table 4. Stations measured in airports during 1994 - 1997.

Year	Airports	Static GPS	Kinematic GPS	Total (GPS)	Survey points
1994	10	206	160	366	1852
1995	7	200	131	331	1469
1996	10	385	107	492	2242
1997	2	69	18	87	540
Total	29	860	416	1276	6103

Geodetic operations in the boundary between Finland and Russia in 1996 - 1997

In 1996 NLS decide to determine more precise coordinatetes to the boundary marks of the boundary between Finland and Russia. The boundary marks where the border line makes an angle were to be maesured. The job was started in 1996 and it will be finished in 1999. The job was again divided: Control Surveys Unit takes care of the southern part and the Oulu Survey Office takes care of the northern part of the border.

The border measurements are concentrated into areas where other operations of NLS are done at the same time. The measurements completed yearly can be seen in picture 1. Comparing newly measured coordinates to the old ones up to 30 m differences could be noticed. The reason for the worst differences can be found from the old boundary survey from 1940's.

Norwegian Mapping Authority
Geodetic Institute

GEODETIC ACTIVITY IN NORWAY 1994 - 1998

1. Introduction

Statens kartverk - Norwegian Mapping Authority is in 1998 celebrating its 225 years anniversary. Norway's Border Survey was founded in 1773 and has since developed to Statens kartverk, the oldest civilian agency of technology in Norway.

Statens kartverk is organised in a classical way according to its main objectives: Topographical mapping in the Land Mapping Division, hydrograpic charting in the Hydrographic Service, geodetic reference in the Geodetic Institute and an international electronic chart service in the Electronic Chart Centre.

The Geodetic Institute was reorganised in 1997 and consists now of three sections: Global reference headed by dr. Hans-Peter Plag. This section covers the analyses of global data from VLBI and GPS in particular. The observation and analyses of geophysical data and in addition larger projects not covered by the other sections. The section for infrastructure and reference data is headed by Rune I. Hanssen. The operation of the fundamental station in Ny Ålesund and other permanent continuos operating stations such as SATREF is covered by this section as well as all computer and network installations. The section is also responsible for the development and operation of the real time dGPS and wide area GPS service.

Mr. Magne Garberg is the head of the section for National Networks. This section covers both horizontal and vertical networks.

2. EUREF in Norway

2.1 The New Norwegian National Geodetic Network - EUREF89

The establishment of a new national geodetic network was given priority in Part 3 of the Norwegian National Mapping Plan. This Plan was officially promulgated in Government White Paper Number 1984:4 under the sub-heading "Geodesy". Completing this task therefore became one of the objectives assigned to the Geodetic Institute, as a division of the Norwegian Mapping Authority, "Statens kartverk". However, in view of the dramatic changes taking place in satellite geodesy during the 1980's, it was difficult to select the best time to address this task.

The Norwegian Mapping Authority decided to establish EUREF89 as official national geodetic datum from 1 January 1993. The objective then was that this datum would replace both of the existing datums, NGO1948 which had been used for technical and large scale mapping series, and ED50 which had been used for topographic and geographical mapping.

Geodetic Institute agreed plans for major GPS activities in the late summer of 1994. The whole work of establishing, measuring and adjustment of the New Norwegian Geodetic Network, called "Stamnett" was carried out during the next three years. A final adjustment for the 930 Stamnett stations distributed throughout the whole country was dated 30 May 1997.

A Stamnett station is a marked point in the New Norwegian National Geodetic Network with coordinates in the geodetic datum EUREF89. Stamnett stations exist in two types, "3D Stations" and so-called "Main Stations".

The costs of the whole project was very near what was estimated in the 1994-budget, and the quality of the final coordinates was better than expected.

A more comprehensive presentation of the New Norwegian National Geodetic Network will be given in a later session of this meeting.

2.2 Densified national network

Based on the 930 stations determine in the New Norwegian National Geodetic Network the Densified National Network in EUREF89 is establish with 3-5 km between GPS observed points in developed areas. Remote stations inunhabited regions and other areas not covered by development plans which are not covered by new points in the Densified Network. The Densified Network covers 75 municipalities with 112 other municipalitied under preparation as per May 1998. There are 435 municipalities in Norway and the remaining will be covered on request. The Densified Network requires active participation from the municipalities or otherwise financial contribution on a cooperative basis.

2.3 UTM as map projection

In connection with the introduction of EUREF89 as a new national geodetic datum in Norway, UTM has been chosen as the new national map projection. That means maps in all official map series will be produced in UTM. Due to the fact that the UTM zones are narrowing in direction of the poles, we are not utilizing the advantage of the scale factor 0.9996 in the northern areas, if we are not operating with broader zones up north. As a consequence of this, we are working for expanding the double zones in Svalbard to be used further south. That means between 84° and 72°N zone 34 do not exist. Zone 33 has zone 35 as neighbour zone in that area. We work for zone 34 to be dropped further south to 64°N.

2.4 Transformation programs

GPS receivers are about to be introduced among professional surveyors as well as among ordinary users and navigators. Geodetic Institute has felt a strong need for transformation between EUREF89 coordinates and coordinates in the national geodetic datum NGO1948 and ED50. A small group of four persons is formed to take care of the transformation program to be used nation wide. The transformation program is called WSKTRANS and works on two levels. On the national level the formula is based upon approximately 500 of the old first and secondary trigonometric stations where we have coordinates in both geodetic datums, EUREF89 and NGO1948. The accuracy is guarantied within 1m. On the county level all the Landsnett points are used in what is called the county-formula, since there the density of points with coordinates in both datums is much higher. The accuracy is in average about 2cm (1 σ).

2.5 The Norwegian Civil Aviation Administration and EUREF89.

In 1996 The Norwegian Civil Aviation Administration (NCAA) started a national project together with Geodetic Institute where the goal was to present all the Norwegian airport coordinates and all the NCAA installations in coordinates adjusted in EUREF89 as geodetic datum. All together 55 airports and some 20 radiotransmitters covering the main land and the arctic islands where included. The two private surveying companies Fjellanger Widerøe and Kartkonsulentene got the contract in a competition between the five most important Norwegian surveying companies. Geodetic Institute worked out the surveying specifications and did all the control work. The experiences showed very clear the importance of the control. The project was finished in September 1997 when the goal was achieved. Then all horizontal coordinates were given as geodetic coordinates, and all heights given in reference to the NKG geoid89.

2.6 European Vertical GPS Network - EUVN.

Geodetic Institute participated in the planning of the EUVN97 GPS campaign that was realized 21 - 29 May 1997. The campaign was the first one to include both West and East Europe in a common geodetic project. The goal was to get a common European heightsystem. In all 31 countries participated in the GPS campaign with approximately 180 station. Roughly 1/3 of the stations were UELN station, 1/3 tide gauges and 1/3 EUREF stations. Norway participated with 12 stations in the campaign.

4. Space geodesy

4.1 Fundamental station Ny Ålesund

The King of Norway carried out the inauguration of the geodetic observatory in Ny Ålesund on Aug. 16, 1995. The new 20 m VLBI antenna had then been on trial for several months. Together with the permanent GPS station in the IGS-network, permanent PRARE and DORIS as well as the Russian system SYRIUS-A, absolute and tidal gravity plus a permanent tide gauge the Ny Ålesund site is one of the best collocated fundamental stations on the northern hemisphere.

4.2 Permanent geodetic stations

The SATREF network with 11 permanent GPS sites continued to operate with an emphasis made for navigation. dGPS data was transmitted on the maritime beacons operated by the Coast Directorate and on RDS from Norwegian Telecom radio broadcasting program P2. A dGPS RDS receiver was manufactured by the company Seatex in Trondheim. The American company DCI however won the competition about RDS broadcasting in the other Nordic countries and it is therefore not a unified nordic system for RDS broadcasting of dGPS.

4.3 EUREKA project SWIFT

A EUREKA R&D project was started in January 1994 with French Telecom, Terracom in Sweden, Norwegian Telecom and Statens kartverk as partner for the development and testing of a FM subcarrier for data transmission. The results including an international standard proposal for the broadcasting part presented to ITU-R also included practical demonstrations of data broadcasting at the Montreux festival in June 1995 followed by similar demonstrations in France, Sweden and Norway during the autumn of 1995.

4.4 R&D project DECIPOS

An R&D project phase one study of development of a real time service giving decimetre accuracy was started by NMA in cooperation with Kværner Marine Automation in 1997. There are contact with a similar project CICERON in Sweden in order to coordinate the activities and preferably aim for an international standard.

4.5 High precision positioning

NMA has carried out a number of high precision positioning in several countries in Europe, Africa and the Far East. The main purpose has been precise WGS84 coordinates in base navigation stations for offshore activities as well as core points (3D type) for new national geodetic networks.

4.6 Professional staff

It has been very difficult to get qualified personnel to carry out the required analyses of data from the permanent geodetic infrastructure. The demand from private industry is very high but NMA hope to see some improvement in 1998.

5. Other geodetic activity

5.1 National standards in geodesy

Geodetic Institute has been responsible for two geodetic standards in the reporting period; one about official heightsystem and reference levels, 1995 and one about registers and numbering of trigonometric stations and bench mark, 1996. The institute is also working with a standard related to geodetic datum, transformation and map projections. In this subject we are also participating in the work of the ISO standard "Geographic information by coordinates", ISO/TC 211.
A standard concerning GPS is recently under planning in Geodetic Institute.

5.2 Delimitation lines in the open sea between nations

Geodetic Institute has worked for the Foreign Ministry as technical expertise under the negotiations between Norway, Denmark (Greenland) and Iceland in the area between Jan Mayen, Greenland and Iceland. In the autumn of 1997 the three parts came to agreement about the last 20 km^2. So now there exists no more doubt about the delimitation lines around Jan Mayen. We have also been active in the GALOS work (GALOS = Geodetic Aspects of the UN Law Of the Sea). GALOS is a sub committee under IAG.

5.3 Calibration baseline

In Norway, 13 local baselines or nets are established to check the instrumental errors of electro-optic distance meters. These lines are regularly re-measured, using the precision distance meter Mekometer ME5000, to monitor the length stability. During the years 1994-1997, 5 of the lines have been re-measured.

5.4 Tracking Integrated Positioning System

Geodetic Division is operating a vehicle equipped with a system for establishing and updating a national road data base.
The system is based on a combination of differential GPS and Inertia Navigation System.
A presentation of the system will be given during the meeting.

5.5 The NEONOR Project

Neotectonic crustal deformations have been reported at a large number of locations in Norway (both on local and regional scales). The NEONOR Project represents a national effort by several national research and mapping institutions to investigate these phenomena through a multidisciplinary approach. Both the industry and the Norwegian Research Council contribute with major financial support. The main objectives of the NEONOR Project are to systematically collect data, and to provide answers to the questions: 1) How can recent crustal deformation be characterized in time and space? 2) What processes cause the neotectonic crustal deformations? 3) What are the implications for migration and occurrence of fluids (especially hydrocarbons and groundwater) in bedrock? 4) What are the implications for geohazard related to constructing sensitive installations like pipelines, gas-terminals and hydropower-plants.

Geodetic Division contributed in 1997 to the project with GPS geodetic measurements in two areas whose active faults have been observed. Reconnaissance and site selection was carried out as well as field observation campaigns. Calculations are now going on. The measurements will be repeated in two years and results presented in a final report.

5.6 Courses and information about geodesy

Together with Norges Karttekniske Forbund, which this year has changed name to Geoforum, Geodetic Institute has arranged the yearly "Geodesidagene" in Norway. In 1997 the arrangement had 10 years celebration. Normally there are 70 to 90 participants. In relation to the New National Geodetic Network, Geodetic Institute has run many courses about geodesy, datum and transformation in the reporting period.

5.7 Part-time professor in Space geodesy

Dr. Bjørn Ragnvald Pettersen from Statens kartverk was in 1997 appointed professor II in space geodesy at the Agriculture University - NLH. The position is a 20% part time engagement which we hope will contribute to the higher education e.g. graduation and candidate studies in geodesy in Norway.

Geodetic Activities at the National Land Survey of Sweden 1994 - 1998

Presented by Bo Jonsson

On January 1st, 1996, the National Land Survey got a new structure. The new organization is founded on the concepts of "orderers" and "performers". For geodesy this means only small changes as compared with the earlier organization.

1. Satellite positioning (GPS)

Since 1989 GPS is routinely used for establishment of control networks. Guidelines for GPS positioning, "Handbok till mätningskungörelsen - Geodesi, GPS", were published in the first edition 1993; it has since been revised and the second edition was published in 1996.

During 1994/1995 GPS methods for cadastral surveying and detail measurements were developed, and a number of surveyors were trained for GPS measurements. GPS equipment was purchased to some regional offices in the land survey organisation.

For production work we are using the manufacturer standard processing software, like PRISM and PNAV from Ashtech, Geotracer from Spectra Precision, Ski from Leica etc., together with the adjustment program GeoLab. For high precision positioning and scientific applications we are using the PC version of the Bernese processing software.

During winter 1998 initial tests with single frequency combined GPS/GLONASS receivers have been carried out.

The National Land Survey has participated in a number of Nordic and global GPS campaigns, of which the most important ones are:

- NORDREF 94. The purpose of the campaign was to improve the transformation formula between the Swedish GPS reference system SWEREF 93 (EUREF 89) and the Swedish traditional reference systems RT 90 and RH 70/RN 92. The campaign solution included all IGS sites in Europe, some EUREF sites in Denmark and Norway, all SWEPOS stations and 18 new sites in Sweden.

- EUVN 97. The purpose of the campaign is among other things to contribute to a unification of European height systems with an accuracy of some centimeters. Four Swedish tide gauges were included.

- BSL 97, Baltic Sea Level campaign.

The National Land Survey has arranged a number of seminars and courses in order to inform geodesists, surveyors and other GPS users in Sweden about the development of the GPS technique, and to report on practical experiences from GPS measurements. For further stimulation of exchange of GPS information between GPS users the National Land Survey is operating an electronic BBS and, in the near future, a WEB-site. We have also accepted to be the national point of contact of Sweden in the subcommittee "International Information" of the "Civil GPS Service Interface Committee". The National Land Survey also participates in the on-going discussions on European activities in the navigation/positioning field.

2. Network of permanent reference stations for GPS (SWEPOS)

The Swedish GPS reference station network SWEPOS was outlined during the end of the 1980s, and the network has been under development since 1991. An experimental SWEPOS consisting of 20 stations, designed and built by the National Land Survey together with Onsala Space Observatory was in operation in 1993. Currently 21 stations are operational in IOC mode. The purpose of SWEPOS is to

- provide single- and dual-frequency data for relative GPS measurements.
- provide DGPS corrections for broadcasting to real-time users.
- act as high precision control points for Swedish GPS users.
- provide data for geophysical research.
- monitor the integrity of the GPS system.

During 1994 the Swedish company Teracom, which broadcasts the Swedish radio and television programs, started a commercial DGPS service, called EPOS, for the distribution of real-time corrections from twelve SWEPOS-stations. The EPOS service covers the whole of Sweden and is using the RDS channel on the FM-Radio network as distribution channel.

In 1995 a cooperation group, consisting of representatives from the National Railway Administration, the National Road Administration, the Civil Aviation Administration, the National Maritime Administration, the State Railways, the Telecom, and the Swedish Defence, was established. The tasks of the group are to approve the final design of an operational SWEPOS and to contribute to the financing.

It was decided that SWEPOS should be developed in two phases. In the first phase SWEPOS should be operational for real-time applications on the meter level and for postprocessing applications on the centimeter level; this was completed in early 1998. The goal for the next phase is that SWEPOS shall be operational for real-time positioning with a few centimeter accuracy in the beginning of the next century.

All the SWEPOS stations are connected to a central node, or control centre, using TCP/IP connections. 1 Hz raw observation data and RTCM correction data (DGPS and RTK) are sent via the communication channels to the control centre.

Communication with other types of sensors on the reference stations, like meteorology sensors and battery loaders, are also using these communication channels. The control centre thus has access to all the observations in real time and provide both real-time data to distributors and observation data for postprocessing direct to the end user.

To investigate the conditions for a national service for real-time positioning on the decimeter/centimeter level, National Land Survey, Onsala Space Observatory and Teracom have decided to cooperate in a project called CICERON. The plans are to study the conditions for such a service during 1997 - 1998 with respect to the modelling of atmospheric effects and multipath errors, predicted orbit information, the DARC channel on the FM-Radio network and the SWEPOS network.

The compatibility of SWEPOS data with available GPS equipment on the market has been studied both for postprocessing and real-time applications in two diploma works. In another diploma work the accuracy and the length of the observation time have been studied for different methods and equipment when determining new points relative to the SWEPOS stations.

Combined GPS/GLONASS equipment will be installed on some SWEPOS stations for a joint Swedish participation in the IGEX experiment during autumn 1998.

Data from the SWEPOS stations Onsala and Kiruna are included in the IGS network, and in the permanent EUREF network data from the SWEPOS stations Onsala, Mårtsbo, Visby, Vilhelmina and Kiruna are included.

3. Reference systems and transformations

Since 1995 a project involving GPS measurements on triangulation stations (RIX 95) has been in operation. This is supported by the same group of authorities as SWEPOS, with addition of the Association of Local Authorities (but without the Civil Aviation Administration). The principal aims are to establish transformation formulae between the national reference systems SWEREF 93 / RT 90 and local coordinate systems, and to establish new points easily accessible for local GPS measurements. The project is to go on for 10 years; each year about 400 triangulation stations and 550 new points are measured.

The SWEPOS network makes it possible to determine the position anywhere in Sweden in the SWEREF 93 system with cm accuracy, today by postprocessing and tomorrow maybe in real time. In the light of this, and the increased use of GPS for all kinds of purposes in society, a project has recently been initiated which aims at producing guidelines for the future handling of reference networks and reference systems.

Some new methods for making coordinate transformations in three dimensions have been developed. They also include a method for transformations between three-dimensional systems and horizontal systems, using a modified transverse Mercator projection.

It should also be mentioned that a Swedish geoid height system connected to SWEREF 93 and RH 70, and based on the Nordic geoid NKG 96, is in progess.

4. Levelling

The third high precision levelling is continuing, using the motorized levelling technique. Three teames are measuring during the field season, producing between 2000 and 1800 km per season. The relevelling percentage has been high, 10 %, for some years, but was last year reduced to 8 %.

Until the whole network is completed (probably in 2003), all heights are computed in the height system of the second high precision levelling, RH 70. At present, heights in the new network are available for the part of Sweden south of the line Luleå - Gällivare - Tärnaby; this part covers 4/5 of the country.

Each year about 2000 bench-marks in the southern half of Sweden are visited to see how many of them that have been destroyed. It turns out that more than 10 % of them have disappeared; these are usually replaced by new ones. In general, nearly 1 % of the bench-marks disappear every year.

5. Gravimetry

In a Nordic cooperation ship gravity measurements have been performed in the Baltic Sea during 1996. The measurements covered mainly the Baltic proper east of Sweden and around Gotland; a smaller part in the Gulf of Bothnia not already covered by ice measurements was also included. The aim of these ship measurements was to help filling the large gravity data gap over the Baltic proper in order to improve the Nordic geoid.

No measurements have been made in the first order gravity network. In the second order network (5 km x 5 km) 900 points have been measured. In addition, 3000 points measured by the Swedish Geological Survey have been included in the data base. Over 90 % of the country is now covered by gravity measurements; what remains are mainly small areas in different parts of northern Sweden.

6. Geodynamics and marine geodesy

In cooperation with Onsala Space Observatory continuous GPS observations are performed, since 1993, on the permanent reference stations in the SWEPOS network. The purpose is to monitor crustal motion, especially postglacial rebound, in three dimensions, i.e. both vertical uplift and horizontal strain.

A series of studies have been made on the postglacial rebound of Fennoscandia using long-term mareograph data and gravity data. This has led to, among other things, a set of absolute uplift rates possible to use for the reduction of GPS-measured heights of the permanent reference stations.

A determination of the mean sea surface topography of the Baltic Sea and its transition area to the North Sea has been completed, in cooperation with the Finnish Geodetic Institute. The results are presented in a consistent Nordic height system, NH 60, designed for this purpose; they agree very well with recent oceanographic models.

* * *

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13th General Meeting of the Nordic Geodetic Commission
Gävle, Sweden, May 25-29,1998

Report of the Geodetic Commission of Estonia, Latvia and Lithuania

A. ELLMANN [1]

INTRODUCTION

The commission between the three Baltic countries was formed in 1990 after well-known political changes and it was named the Geodetic Commission of Estonia, Latvia and Lithuania. Currently the Geodetic Commission of Estonia, Latvia and Lithuania consists of 12 persons - four representatives from each country. According to the statutes, the National Associations of Surveyors nominate canditates, whereas in order to support the development of geodetic science and practice in the Baltic countries, also Directors General of national agencies responsible for the performance of geodetic works are members of the Geodetic Commission.

The main objectives of the Commission are to coordinate the national geodetic and cartographic activities and to develop common methods and standards, to develop recommendations for the creation of regional geodetic and cartographic information banks, to coordinate scientific studies, to organise scientific conferences and seminars, to train researchers in specific fields, to enhance foreign relations, particularly with the former member countries of the Baltic Geodetic Commission, etc.

INTERNATIONAL COOPERATION

Up to date the Geodetic Commission of Estonia, Latvia and Lithuania has participated in the realisation of several international cooperation projects.

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 cooperation 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 Budapest in 1993.

Relying on the approved values of coordinates, all Baltic states have started to implement the new European Reference Frame.
After the publication of EUREF.BAL'92 results, the Geodetic Commission of Estonia, Latvia and Lithuania elaborated a recommendation for the national agencies of three countries responsible for geodetic and cartographic activities for adopting the coordinate

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system ETRS89 as a national coordinate system. In Latvia and Lithuania the use of coordinate system based on ETRS89 is regulated by a Government Decree, in Estonia the new coordinate system has been widely used since 1993 and in near future the use of the system should be approved by the Government.

As the ongoing land reform in the Baltic countries requires extensive land surveying, a further 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 in the past years. During the initial stages of the GPS networks establishment, the Nordic countries in particular have provided us with measuring instruments and know-how transfer.
A good example of that is the Nordic-Baltic Geodetic Seminar "Coordinate Systems, GPS and the Geoid" which was funded by NorFA (Nordic Academy for Advanced Study) and organised by the Finnish Geodetic Institute (FGI), and which helped the specialists from the Baltic countries to extend their horizon and to implement the acquired knowledge in their further work.

In 1995, FGI performed gravimetric absolute measurements in the Baltic countries determining absolute values of gravity on three stations in each country, 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.

In the past years the Resolution No.2 about the establishment of geodynamic reference (GPS) stations in the Baltic states, adopted in the 12th General Meeting of the Nordic Geodetic Commission in Ullensvang in 1994, has been realised. Thanks to the equipment and technology provided by the Onsala Space Observation and FGI, permanent GPS stations are currently operating in all three countries, and data from these stations is forwarded to the Nordic countries.
The data from permanent GPS stations have been used also in EUVN 97 and BSL 97 GPS campaigns.

Financed by the Danish Government, Kort&Matrikelstyrelsen (KMS) has, through the Geodetic Commission of Estonia, Latvia and Lithuania, initiated a sector programme that is mainly oriented to the improvement of vertical networks of Estonia, Latvia and Lithuania. It includes connecting GPS networks with national levelling and gravimetric networks, improvement of the geoid model of the Baltic region by additional GPS, levelling and gravimetric observations, data processing and creation of basis for geodetic data bases.
Within the framework of the sector programme, Estonia, Latvia and Lithuania will obtain modern geodetic instruments, hardware and software. The most important benefit is however that the participants in the programme are provided with on-job-training under supervision of the leading Nordic specialists, or with training in Denmark. Up to date several training programmes have already been completed, and currently field work is being carried out. Conclusions about the results of the sector programme will be drawn in autumn 1998, but already now it can be said that the cooperation has been very successful and all the objectives will be fulfilled.

CURRENT SITUATION AND PLANS FOR FUTURE

According to the statutes, the post of the Chairman of the Geodetic Commission of Estonia, Latvia and Lithuania is subject to rotation every third year. In the years 1990-1993, the Geodetic Commission was chaired by Dr. J. Balodis from Latvia, from 1994 to 1996 by Dr. V. Tulevicius from Lithuania, and in 1997 up to 1999 A. Ellmann from Estonia was elected the Commission's Chairman, whereby the seat of the Geodetic Commission is now located in Tallinn.

The Baltic cooperation has been rapidly increasing during the past seven years in every field. Also the authorities that are responsible for geodetic and mapping activities have been in good collaboration, but at the same time the need for an even more close cooperation has become obvious.

One possibility to achieve this end is to participate in the work of the Baltic Council of Ministers. On 9 May 1998 a Committee of Senior Officials was formed at the Baltic Council of Ministers, one of the responsibilities of the Committee being the coordination of activities in the field of geodesy and mapping.

One representative from each country belongs to the Committee of Senior Officials: from Estonia Director General of the National Land Board Mr. K. Kangur, from Latvia Director General of the State Land Service Mr. G. Grube, from Lithuania Director of the National Service of Geodesy and Cartography Dr. Z. Kumetaitis, who at the same time are the members of the Geodetic Commission of Estonia, Latvia and Lithuania. In order to solve the tasks, it is obvious that the Committee of Senior Officials has to form several commissions and work groups. It is highly probable that the Geodetic Commission of Estonia, Latvia and Lithuania will continue its activities as an official commission of the Committee of Senior Officials at Baltic Council of Ministers.
Of course it is a positive development which improves the prestige of the Geodetic Commission of Estonia, Latvia and Lithuania and probably only thereafter it will be possible to solve the problems of financing the Commission.
The main objectives are a more close cooperation between the specialists of the three countries, coordination of common projects, organisation of conferences on regular basis, and informing of specialists and the public. A regular and efficient financing of the Geodetic Commission of Estonia, Latvia and Lithuania will help to enhance even relations with other countries, first and foremost with the Nordic Geodetic Commission.

The meeting of the Geodetic Commission of Estonia, Latvia and Lithuania held in April 1998 discussed among other things the current state and continuation of work in a new situation. The international cooperation projects realised in the field of geodesy, mapping and GIS in the past years have indicated that cooperation and an intensive information exchange between specialists of different regions, and even of the whole continent, is necessary.
According to the resolution of the meeting, the Chairman of the Geodetic Commission was authorised to inform the Nordic Geodetic Commission at the 13th General Meeting in Gävle on 25-29 May 1998 about the activities of the Geodetic Commission of Estonia, Latvia and Lithuania.

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Finland (SF): Three FL-teams have continued the levelling work in the precise levelling network and they produced about 1640 km double run levelling in the central & northern parts of Finland with the classical technique using spirit levels. The levelling work is planned to be finished in year 2001 or 2002. Parts of some loops have been remeasured and some loops have been cut off by cross lines due to big loop misclosures. Deviations have been detected when new measurements are compared to old ones.
A new horizontal-vertical comparator has been set up in the new FGI building in Masala.
The NLS (Lantmäteristyrelsen) has during the period carried out levelling in the second and
Third order network.

Norway (N): The decreasing trend from the last period seems now to be stabile around a minimum production. During the period one team produced 680 km double run distributed all over the country. In addition to that the NLS from Sweden produced 70 km double run near the border to Sweden. The levelling technique has changed during the period, from motorised levelling using the NI002 to foot levelling using the digital instrument DiNi10 from Zeiss. In 1994 a 10 year plan was presented, aiming to "secure a new Norwegian Height System by a common Scandinavian adjustment around year 2002 to 2005", but the plan was never implemented as presented. A new revised plan has been announced several times.

Sweden (S): The production has decreased during the period, due to shorter seasons, since the measurements are now located in the far north of the country. Three teams produced 7110 km double run levelling in the network using the motorised technique and Zeiss Jena NI002. Some problems with high relevelling percentage turned out to be due to the instruments. After a thorough check of the instruments in Germany the relevelling decreased again. The bench mark maps and descriptions are now produced and stored in a fully digital production line. Due to the great deterioration of bench marks (up to 1% per year), the updating of the network has been going on during the period. About 2000 BM/year were updated and about 300 km double run levelling per year were performed in order to connect the replacement BM.

CONTROL LEVELLING OF TIDE GAUGES

Control levelling of TG(tide gauges) and TGBM(=tide gauge bench marks) have been done in all countries during the period. Discussions on this matter showed that this is done in separate ways in different countries. The results from an investigation in all Nordic countries, in order to set up guide lines for this kind of work, showed that the time interval between control levellings of the TGBM varied between one and fifteen years in a most irregular time interval. The control levelling of the TG to the TGBM turned out to be the responsibility of other authorities in some countries, but the connection of the TGBM to the national levelling network was the responsibility of respectively national survey authorities: KMS, FGI, SK and LMV. The control levelling were always performed as precise levelling, using the ordinary precise levelling equipment.

OTHER ACTIVITIES

Besides the questions strictly concerning the production of the precise levelling networks, items on analyses and calculation of the networks were often discussed, since problems concerning this area are now coming closer. The titles of the following technical reports are examples of what has been discussed:
-New data systems replacing old ones (DK)
-Establishment of a common Nordic calculation Ad-Hoc group (S)
-Analyses of repeated levellings (SF)
-Error analyses in precise levelling (S)
-Determination of mean sea level (DK)
-Calculation of the third precise levelling in Denmark (DK)
-Analyses for detection of systematic errors in the third precise levelling in Sweden (S)
-Land uplift calculations in Norway (N)
-Definition of height systems (S)
-Land uplift determination (S)

Other items discussed were:

- Densification of the precise levelling network with MTL (DK)
- Readjustment of the Nordic block of UELN-73 (SF)
- EUVN97 campaign (N)
- Calibration of invar staffs (SF)

NEW INSTRUMENTATION AND TECHNIQUES

The first digital level NA2000 from Leica has been modified and improved in several steps. The latest version is the NA3003 series. Digital levels are available from many manufacturers today. Zeiss developed maybe the most interesting one DiNi10, followed by DiNi11. This instrument was tested both in S and SF. Results were presented from both tests. Other digital levels are also available, but they seem to be more or less copies of the NA3003. However, the digital levels have come to stay.

GPS can not yet replace classical precise levelling, although improvements have been done during the period. Many problems still have to be solved, which is shown in some of the technical reports presented. ”Limitations in height determination with GPS” deals with antenna problems, the near environment field and the multipath phenomenon. The conclusion was that even cm-accuracy requires great care. ”Geoid undulations in GPS height measurements” showed on the problems in getting an enough accurate geoid model to provide good heights.
Other items presented:
- Tests of EDM Height Traversing over long distances (SF)
- ISO standards about surveying instruments (S)

1997 a similar marine project was carried out in the North Sea in a Danish-Norwegian cooperation, financially supported by the danish research foundation GEOSONAR project, again filling in important data voids, and again using R/V Haakon Mossby.

A major effort in the past 4-year period has been the computation and release of the new Nordic geoid model NKG-96. Compared to the earlier model NKG-89 many improvements have been made:
1) Much new gravity data has been incorporated - especially from eastern Europe,
2) Data voids in the Baltic Sea has been filled with "draped" satellite altimetry gravity,
3) Higher resolution digital terrain models have been used (e.g., 100 m resolution in Norway), and
4) New spherical FFT methods have been used for a more accurate computation.
The details of the computations can be found elsewhere in this volume (Forsberg, Kaminskis and Solheim: "The NKG96 geoid").

The NKG-96 geoid is made available to users as a grid file and an associated DOS interpolation grid program. The quality ranges from a few cm in areas with good gravity coverage to some 10 cm in less well-covered areas. The overall level of the geoid has been fitted to coincide with the european levelling system UELN and ITRF93 (through an empricial additive constant). However, due to differences in the levelling systems of the Nordic countries, the geoid will show national bias discrepancies up to some 20 cm, which subsequently should be fitted into a national geoid by comparison to precise GPS levelling. This has e.g. been done in Denmark, yielding a geoid accurate to 1-2 cm when used in combination with heights in the danish levelling system and GPS coordinates tied to the national realization of EUREF (REFDK).

The geoid computation work has now reached a mature state, and efforts of the near futute should put more weight on comparisons with GPS-levelling, and the adjustment of the geoid to the vertical networks of the region. Due to the inherent high quality of the geoid model, this effort put great constraints on the allowable data entering such a process: GPS points should preferably be levelled, and GPS sessions must be sufficiently long and well tied to fundamental GPS reference points. Also land uplift must be taken into account since GPS and levelling refers to different epochs.

However, gravity data collection and validation activities should still continue, and there is need a.o. for improved gravity data in eastern Europe (existing data have some uncertainty in transformations to western gravity system), a need for more gravity data over the Baltic Sea, as well as more detailed data coverage of some land regions and coast-near waters. For the Baltic Sea a joint airborne gravity project might be the most efficient way to collect the necessary data, and it is proposed to continue the cooperation from the marine surveys to establish such an airborne gravity project.

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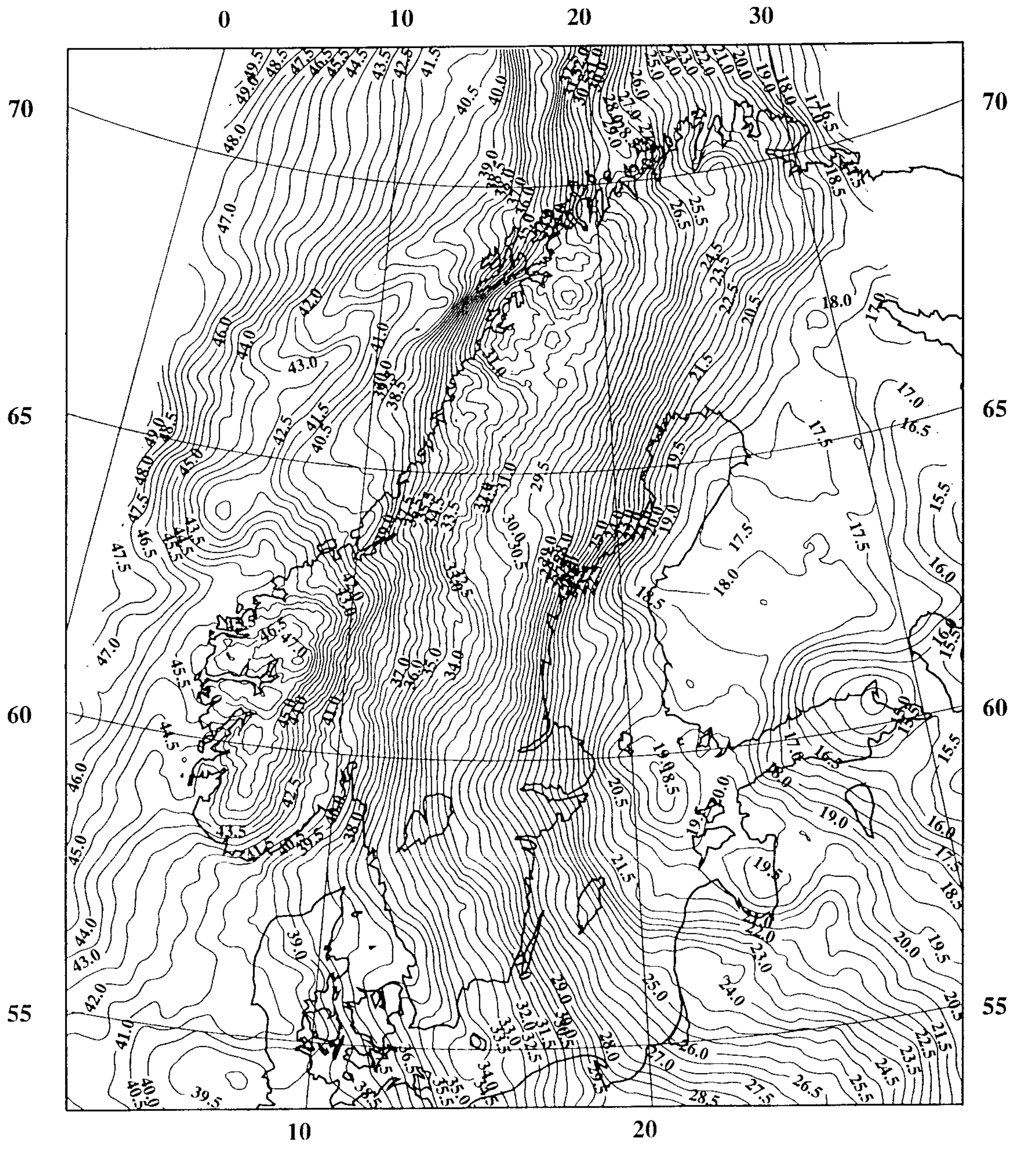

NKG96 geoid, 0.5 m contour interval.

Report of the NKG WG for Geodynamics Period 1995 – 1998

from the chair

M. Vermeer

May 22, 1998

1 Activities of the Working Group

The WG for Geodynamics is the oldest of the existing working groups of the Nordic Geodetic Commission; an established tradition existed when I became its chairman at the Ullensvang Meeting. I have tried to strive for continuity, mainly by continuing the tradition of assembling yearly, every time in a different Nordic country, and having these meetings together with the WG for Height Determination.

Another tradition is the participation of Prof. Gerhard JENTSCH, who has a long involvement in geodynamic projects involving the Nordic countries. Unfortunately due to lack of time, he has been able recently to participate less than he himself would have perhaps wanted.

1.1 Members

The members (national representatives) of the WG have been:

Denmark: Søren Gregersen and Klaus Schmidt

Finland: Martin Vermeer and Jaakko Mäkinen

Norway: Knut Røthing and Erik Roland

Sweden: Martin Ekman and Lars-Åke Haller

Iceland: Unknown. Info sent to Gudmundur Palmason and Águst Gudmundsson

1.2 Meetings

During the report period, yearly meetings were organized, which were coordinated with the Working Group for Height Determination, chaired by Jean-Marie BECKER. These meetings were:

- Gothenburg, 29 March 1995 (geodynamics afternoon)
- Masala, 29-30 April 1996 (Geodynamics 30 April)
- Hönefoss 28-29 April 1997 (Geodynamics 29 April)

The memoranda of these meetings are available.

1.3 Scientific content

Some of the subjects that have been discussed at the various WG meetings:

- Absolute gravimetry in Fennoscandia and Baltic states
- Superconducting gravimeter studies
- Tidal gravimetry, modelling of tidal deformation
- Tilt measurements and crustal deformation modelling
- Local crustal movements as detected by precise levelling
- Study of postglacial land uplift by various techniques, such as precise levelling, precise gravimetry and GPS
- Sea level change and variations

1.4 Other events

A letter received from S. GREGERSEN, representing (as I understand) the views of the Danish delegation, discussed the sensibleness of having these meetings once a year, when the subjects being discussed are also being discussed at several other international meetings. The only purpose is to meet face-to-face, whereas the business could be done more effectively by email.

We should indeed consider the future of the WG for Geodynamics, as well as the whole WG system. It is true that much of the discussions taking place in these groups now also takes place e.g. at the yearly EGS meetings; when the WG was founded, this was (1) not the case, and (2) travelling to central Europe was considerably more difficult and expensive than now.

Also within the Nordic Geodetic Commission circuit, there is a perhaps excessive overlap: Permanent Station, GPS and geodynamics all three have something to do with permanent GPS monitoring of crustal motions, and so does actually height determination.

We should formulate the Working Groups of the NKG is such a way, that

3. Task from the NKG Presidium: Preparation of a proposal for uniform handling of ITRF Epochs in computation of fundamental networks

In 1995 the Working Group on Satellite Geodesy got the following task from the NKG presidium: Preparation of a proposal for uniform handling of ITRF epoch in computation of fundamental networks.

At the Working Group meeting in Gävle 1995 a "state of the art" discussion was carried out and a subgroup consisting of one person from each country was appointed to come up with a proposal:

Sweden: Jan Johansson (chairman)
Finland: Markku Poutanen
Norway: Oddgeir Kristiansen
Denmark: Bo Madsen

The matter has been processed by the sub group in collaboration with the EUREF group.

The sub group proposes to make use of products from the IGS and EUREF networks in computation of fundamental networks. Although, the best solution would be to use a global set of stations and not impose any reference frame in the computation this is often too time consuming and may also lead to large problems both in data retrieval and processing. Given the quality of the IGS products almost the same high quality results can be obtained by the use of IGS orbits and EOP files. By including at least three stations from the IGS and EUREF networks in a free-network solution the connection to the ITRF-EUREF realisation and different epochs are maintained. ITRF and EUREF supplies information on the transformation between different epochs of the ITRF.

Due to the complexity of this problem and on-going international work, the sub-group proposes to continue this activity for an extended period. Recent recommendation from the IERS on implementation of new geophysical models and the availability of longer time-series from the IGS/EUREF stations in the region will better describe present-day crustal deformation in Fennoscandia. Especially, crustal motion associated with post-glacial rebound has previously not been properly represented within the ITRF solutions. A few more years of analysis of data from IGS/EUREF and National permanent GPS stations will give improved information which will be reflected in future ITRF solutions. The future ITRF will include local and regional crustal deformation such as those associated with post-glacial rebound.

4. Co-ordination of GPS campaigns

The Working Group has co-ordinated the following Nordic GPS campaigns and the Nordic part of the EUVN 97-campaign:

- DOSE 94. GPS measurements on a number of tide gauges stations in Finland, Norway and Sweden as a part on the on-going geodynamic studies at Onsala Space Observatory

- NORDREF 94. GPS measurements on some EUREF stations in the Nordic countries and 18 Swedish stations in order to improve the transformation formula between the Swedish reference systems SWEREF 93 and RT 90/RH 70/RN 92.

- DOSE 96. The same purpose as DOSE 94.

- BSL 97. GPS measurements on a number of tide gauges around the Baltic sea during the same period as the EUVN 97 campaign

- The Nordic part of EUVN 97. The purpose of the EUVN 97 campaign is among other things to contribute to unification of European height systems with an accuracy of some centimetres.

5. The future of the Working Group

The opinion of the Working Group on Satellite Geodesy is that there is need for a Working Group on satellite related matters during the next four years period, since there is still a rapid development in the satellite position technique and the use of this technique in geodetic applications.

During the time period 1994-1998 the meetings of the Working Groups on Satellite Geodesy and Permanent Geodetic Stations have been co-ordinated and the meetings of the two groups have had the same participants.

Proposal: It seems natural to combine the two Working Groups on Satellite Geodesy and Permanent Geodetic Stations to a "new" Working Group which covers both satellite geodesy and permanent geodetic stations. The matters concerning reference systems can be included in this new Working Group or in an other convenient Working Group.

On the behalf of the Working Group on Satellite Geodesy
Bo Jonsson

4. Task from the NKG Presidium: to discuss issues of data exchange

In 1995 the Working Group on Permanent Geodetic Stations got the task from the NKG presidium to discuss issues of data exchange between the NKG countries.
The working group has discussed different aspects of this matter at all three meetings. Decisions have been made regarding how requests of data from the different Nordic GPS stations should be handled. Agencies asking for data should always turn the organization responsible for the network in their own country independent of station-data requested. The national operational center will see to that data from other Nordic countries will be transferred to the user. For scientific use data will be free of charge. The decision whether the project is scientific, or not, are taken by the national operative center. Other projects requesting data will be charged according to the national price list.
The working group has decided to make data from some of the stations available to the EUREF permanent GPS network. Data from these stations are sent to an archive hosted at the Onsala Space Observatory. The data are accessed by several EUREF computing centers. The NKG-EUREF analysis center process all the NKG stations.

5. The EUREF Permanent GPS Network

One of the main tasks of the working group has been associated with the establishment of the EUREF permanent GPS network in 1996. As a response to a call for participation from the IGS and EUREF community the working group prepared a proposal. Each country decided on which stations should become members of the EUREF network. Decision were made to make these data available through the establishment of a data archive at Onsala. Furthermore, the working group prepared a proposal to become an EUREF analysis center. This proposal was accepted by EUREF/IGS and the activity started in September 1996. GPS-data from about 25 stations are processed daily. Most of these stations are located in the Nordic countries, the Baltic countries, Iceland, and Greenland. The daily solutions are combined to form a weekly solution which is sent to the EUREF combination center in Berne Switzerland. Solutions from all 10 EUREF processing centers are combined to a final EUREF solution. In the final stage, this European solution is combined with resutls from other regions to form a global solution. This is done by the IGS and utilized by the IERS in their computation of the ITRF and in several other geophysical projects.

6. Projects supported by the permanent GPS networks

As reported by the working group on Satellite Geodesy several large Nordic and international GPS campaigns has been based on the permanent GPS network in our region.

- Support to the IGS and EUREF for satellite orbit determination, establishment of the global and European reference frame, and monitoring of station coordinates.

- Nordic reference frame. The permanent GPS networks are used to establish and maintain the reference frame in our region. The GPS-data from the permanent networks are frequently analyzed by the IGS, EUREF, or by national agencies.

- Baltic Sea Level Project. GPS measurements on a number of tide gauges around the Baltic sea. In the 1997 campaign the permanent GPS stations were available (and used) as a backbone in the data processing.

- The Nordic part of EUVN 97. The purpose of the EUVN 97 campaign is among other things to contribute to unification of European height systems with an accuracy of some centimeters. Like in the case of the BSL-97 campaign several of the permanent GPS stations were included in the campaign.

- DOSE and BIFROST projects: measurements on a number of permanent GPS stations in the area of the NKG. Daily data processing of about 50 stations are used as a part on the on-going studies on geodynamics and sea-level change.

7. Future use of data from permanent geodetic stations

Several, if not all, projects and services list above will most likely continue over the next decade. There is a large number of different topics needed to be discussed in the very near future. Starting in fall of 1998, the International GLONASS experiment (IGEX-98) pilot project is carried out. A number of IGS stations (and other stations) will be equipped with GLONASS receivers and carry out observations in support of both GLONASS and GPS. Most likely will some of our Nordic permanent stations be equipped with combined GPS and GLONASS capability. We anticipate similar discussions concerning GLONASS as in the case of GPS.
Regarding both national as well as international use of data from permanent GPS stations we can expect a trend towards faster turn-around time. Today data are collected each 24 hours. In the near future hourly data offloads will be tested in order to speed up the production of satellite orbit determination and possibly get better orbit predictions. Faster data turn-around time may also be important for GPS-based remote sensing of the atmosphere. The working group has established contacts to e.g. the meteorological community in order to evaluate the possibility to use GPS-based estimates of tropospheric water vapor. Water vapor may be important in both climate research and weather prediction models.

8. The future of the Working Group

In the future, Permanent Geodetic Stations will continue to play an important role in many different aspects. It is important to continue this activity within the NKG also for the next four-year period. The rapid development in global satellite navigation systems and space geodetic techniques underline the importance of an NKG working group dealing with questions related to these permanent networks.

During the time period 1994-1998 the meetings of the Working Groups on Satellite Geodesy and Permanent Geodetic Stations have been coordinated and the meetings of the two groups have had the same participants. As stated in the first paragraph the main work of our group has been concentrated to the permanent GPS stations and not so much to other techniques. Although, some information and topics related to other geodetic stations (VLBI, SLR etc) have occasionally been discussed it is expected that satellite geodetic stations will continue to be the main important issue.

Proposal: It seems natural to combine the two Working Groups on Satellite Geodesy and Permanent Geodetic Stations to a "new" Working Group which covers both satellite geodesy and permanent geodetic stations. The matters concerning reference systems can be included in this new Working Group or in an other convenient Working Group.

On the behalf of the Working Group on Permanent Geodetic Stations
Jan Johansson

LOCATION OF A POSSIBLE GROSS ERROR IN AN OPEN LEVELLING LINE BY USE OF GPS

by

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ABSTRACT

A small part of a long open levelling line in Lofoten was in 1997 measured by GPS in order to detect a possible error. A gross error in the levelling was suspected, based on data from tide gauges.

1. INTRODUCTION

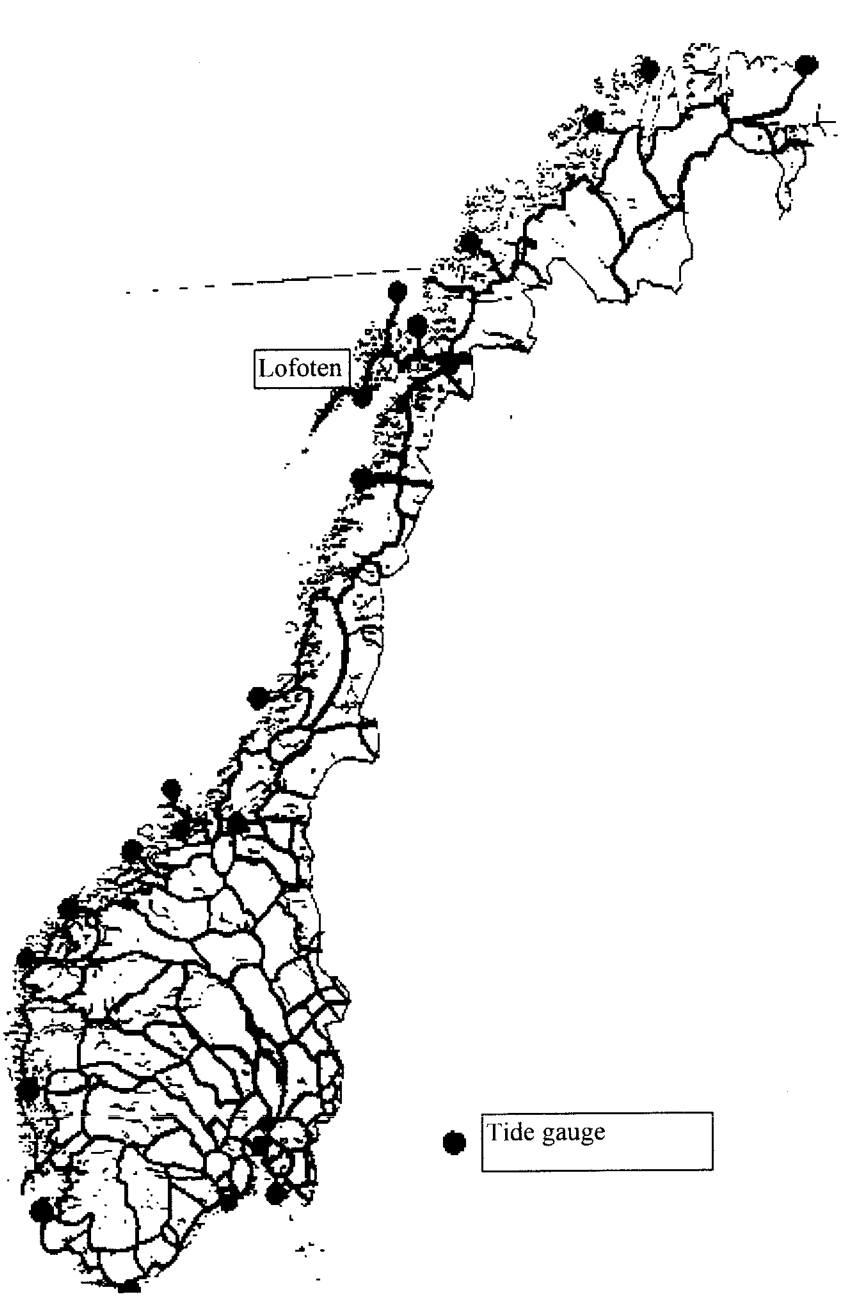

Figure 1. Levelling network of Norway 1995.

In south Norway we have mostly closed loops in the levelling network. Closed loops gives us a chance to discover errors in the levelling, like the error of the relevelling in 1990 of the line from tide gauge Heimsjø to the Swedish border.

In North Norway we have open lines, except for parts of Finnmark. Gross errors may pass unnoticed because we have no control except the control by to- and fro levelling.

Usually we have a tide gauge at the end of the open lines. So if we assume that mean sea level is the same at that gauge and at other nearby gauges we have a control against gross levelling errors. The assumption that there is no difference in annual mean sea level between tidal stations might be false because of sea surface topography.

One of the open levelling lines in North Norway, is situated in Lofoten/Vesterålen, North Norway, and is approximately 200 km long(Figure 2). We suspected an error on that line.

We have no gauge at the end of the line, but we have a tide gauge at Kabelvåg, approximately halfway between the end and the junction point. According to the precise levelling mean sea level at Kabelvåg is approximately 15-20 cm higher than at the nearby gauges. There are two possible explanations:

1. Mean sea level is exceptionally high at Kabelvåg
2. A gross error in the levelling

We found explanation 1 to be unlikely and suspected a levelling error somewhere between the tide gauge and the junction point. We decided to try to use GPS together with geoid information in order to detect the suspected error.

2 ANOMALOUS MEAN SEA LEVEL AT KABELVÅG

New digital tidal equipment were installed in the time period 1986-1991 at the tide gauge at Kabelvåg and the nearby three gauges Bodø, Narvik, Harstad and Andenes. From the same time we also have good control of the zero point of the gauges by frequent levelling from the tide gauge bench mark. From 1992 we have reliable annual mean sea level from all the gauges. Figure 3 shows the mean sea level for the gauges in the official height system Normal Null 1954.

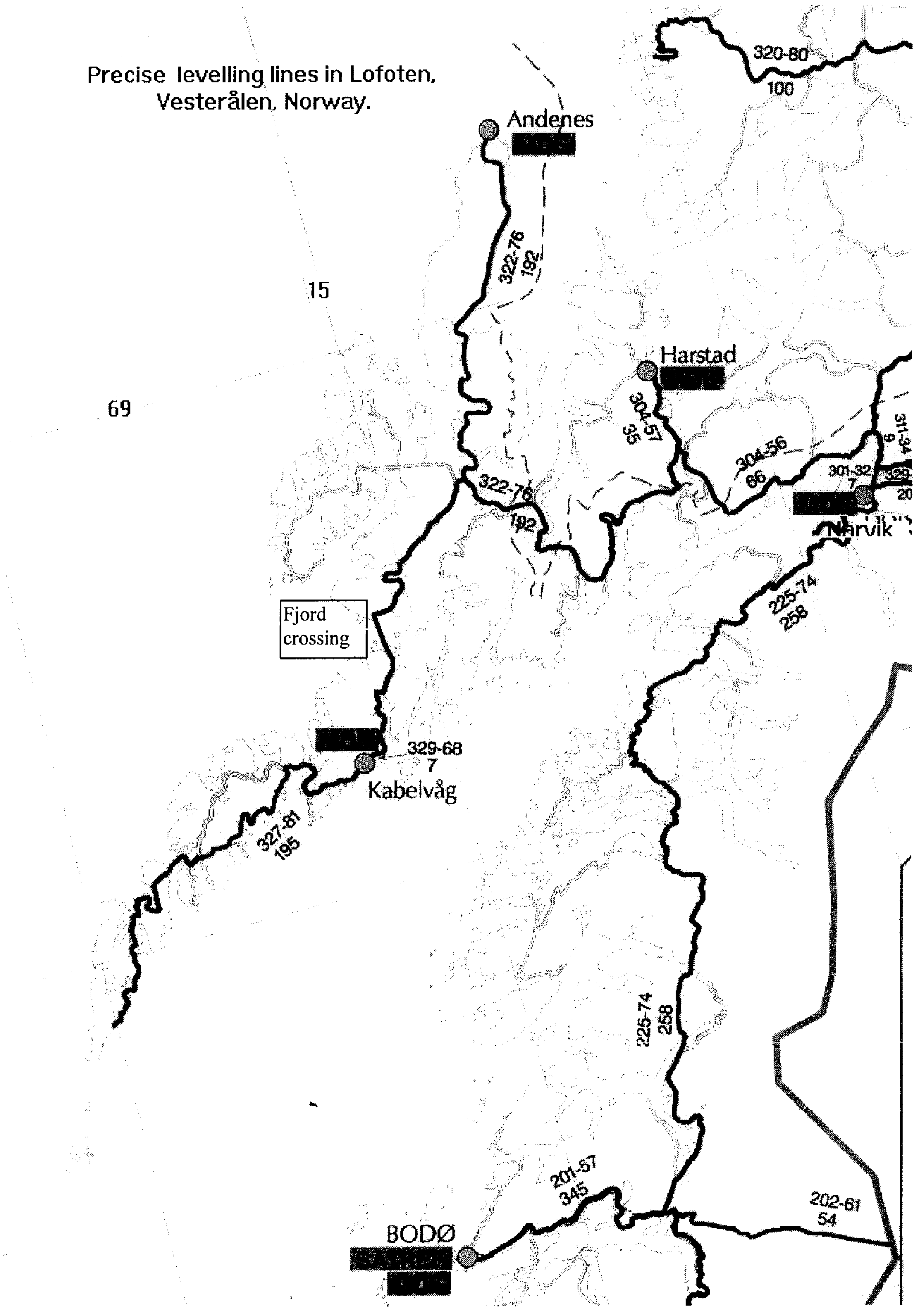

Figure 2. Precise levelling lines in Lofoten and Vesterålen, Norway. Scale ca. 1:1000000
MDS = Tide gauge

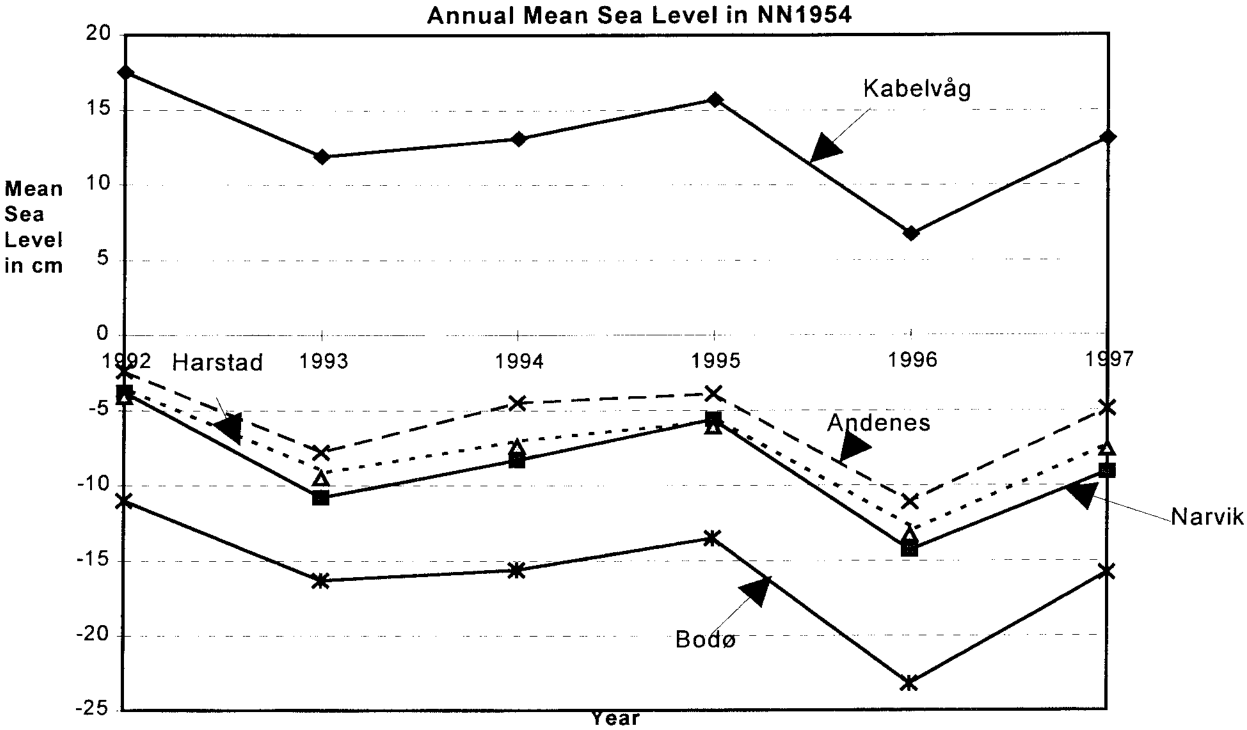

Figure 3
Annual mean sea level 1992-97 at the five tide gauges on Figure 2 in NN1954

From figure 3 we notice that Kabelvåg is anomalous with mean sea level 10-15 cm **above** the zero-level of NN1954 while at Bodø, Narvik, Harstad and Andenes the annual mean sea level is 2-23 cm **below** zero. Usually annual mean sea level is a little below the zero level of NN1954 for the tide gauges in Norway.
Can we expect a difference of 20 cm in annual mean sea level between Kabelvåg and the nearby gauges on the mainland. Certainly there are strong tidal currents around the Lofoten islands. But in my view 20 cm is too much. Professor Bjørn Gjevik, Department of Mathematics, University of Oslo, who has made studies of the Lofoten Maelstrøm[3] and [4], is not surprised that there is a difference in mean sea level between the north and south side of Lofoten. But he agrees that 15-20 cm sounds too much(pers. comm).

If we can't explain the 20 cm difference in annual mean sea level by sea surface topography, we have the possibility of an error in the levelling out to Kabelvåg.

3 ERROR IN THE PRECISE LEVELLING?

The precise levelling line to Lofoten was observed in 1982 and measures 195 km from the junction point at Stokmarknes to the end of the line. From the junction point to the tide gauge in Kabelvåg there is 80 km.
Where on this line between Stokmarknes and Kabelvåg it most probable that the error is located?("Wo steckt den Fehler?", as the Germans say). Three fjord crossings were

performed between the junction point and Kabelvåg. Figure 4 shows the crossings on a map of a scale of 1:50 000. The three fjord crossings are over Hadselfjorden 30 km north of Kabelvåg. The total distance over the fjord is 7 km. We have not only fjord crossings there, but also short foot levelling in the terrain on small islands and islets. Many sources of errors might be present here. It is more likely that the error is "buried" somewhere in the Hadselfjord than in the ordinary foot- or motorised levelling on the road. Therefore we decided to measure with GPS at levelling bench marks on the north and south side of the fjord.

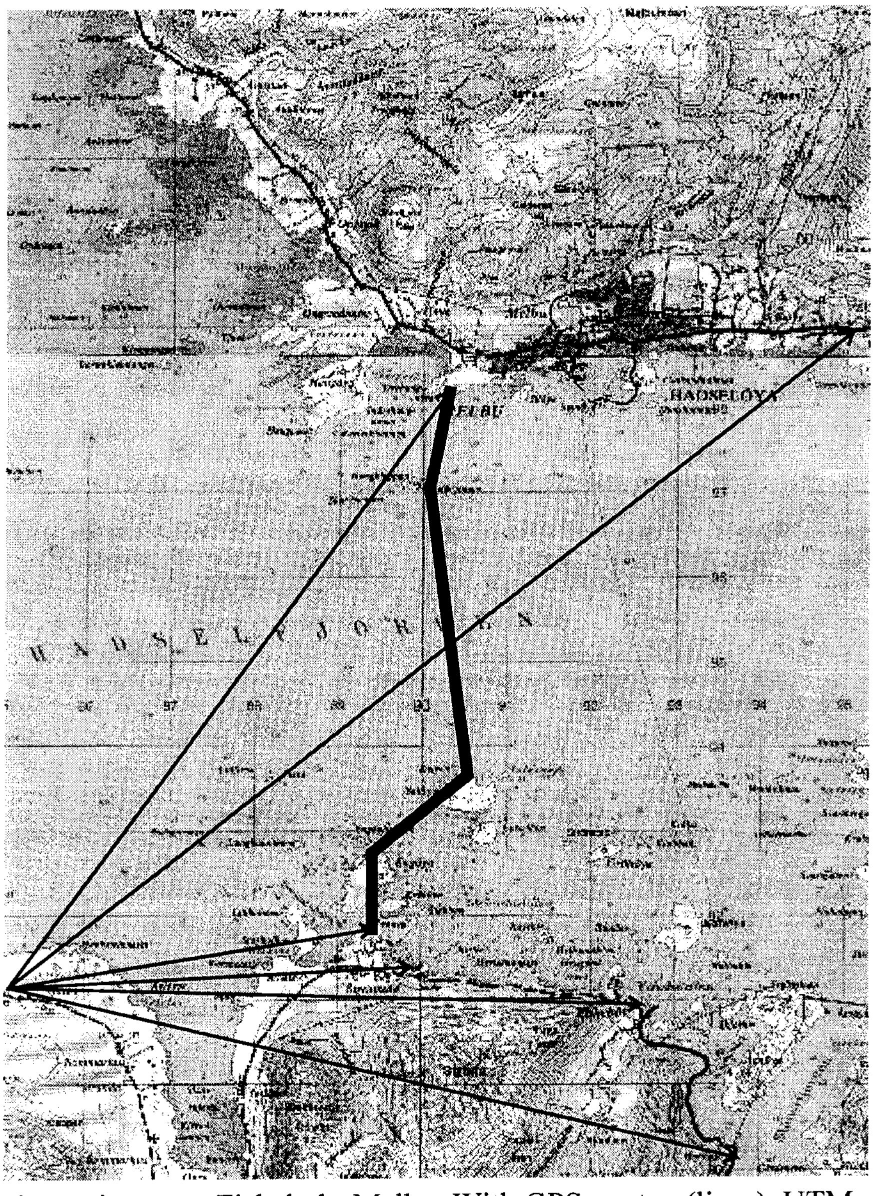

Figure 4. Fjord crossing area. Fiskebøl - Melbu. With GPS-vectors(lines). UTM-grid 1x1km.
Thick line = "fjord crossing"

4 GPS-MEASUREMENTS AND COMPUTATIONS

In 1997 the levelling group at SK bought two GPS Trimble 4600-LS Surveyor receivers for the determination of horizontal co-ordinates for the bench marks. The receiver tracks the L1 C/A-code to achieve 1 metre accuracy. In addition, for land survey applications , both L1 carrier phase and C/A-code are tracked simultaneously to provide millimetre/centimetre accuracy. In the summer of 1997 I was up north for establishing levelling bench marks and for control levelling of tide gauges. I used one day to measure at four levelling bench marks on the south side and three bench marks on the north side of the fjord. As base station we used a point in The New Norwegian National Geodetic Network- Euref89, called *Stamnettet* in Norwegian. Later today Bjørn Geirr Harsson will talk about this network[1]. Without this new network, which in this connection is considered free of errors, our control method could not have been used because our GPS-receivers are only single frequency instruments. All vectors were between 5- 9 km length, except one which was 13 km. One baseline was observed twice, in independent sessions. This is the only check on the solution. The observation time was only 30 minutes at each point. The data were transferred to a PC and computed with the Trimble software suite GPSurvey in the field.

5 GPS RESULTS

The computed co-ordinates in Euref-89 are shown in table 1.

Table 1

Bench mark number	Latitude	Longitude	Ellips. height m	Precision in ell. height in mm
North side of fjord:				
K9N42	68 30 10.52511 N	014 52 51.86727 E	57.272	5.6
K9N50	68 29 43.25851 N	014 45 44.70235 E	39.045	1.9
K9N49	68 29 43.56867 N	014 45 47.23968 E	38.130	4.6
South side of fjord:				
K9N1	68 25 53.36457 N	014 49 30.06319 E	37.662	4.9
K9N4	68 25 02.28110 N	014 50 28.40887 E	43.652	5.1
K9N64	68 26 24.23195 N	014 44 36.36072 E	37.127	4.1
K9N66	68 26 01.12213 N	014 45 44.10253 E	44.928	2.0
K9N66	68 26 01.12222 N	014 45 44.10238 E	44.909	5.5

6 "Heights in NN1954" FROM GEOID + GPS

For the transformation of heights from the ellipsoidal heights in table 1 to heights in our official height system NN1954 I have used Statens kartverk's transformation program WSKTRANS with the new height reference surface VREF 1996, which over a short distance(7 km) is equal to using the (quasi-)geoid. Later in the session my colleague Dag Solheim will talk about VREF1996[2]
The accuracy of VREF1996 is difficult to stipulate, but in South Norway at a test field a standard deviation of 2 cm has been reported(Solheim, pers. comm.).

7. COMPARISONS WITH LEVELLED HEIGHTS FROM 1982

Table 2 shows the bench mark heights derived both from the levelling in 1982(official heights in NN1954), and the heights derived from GPS/Geoid in 1997. The differences between these two heights are also given.

Table 2

Bench mark no	**GPS height m**	**Geoid height(NKG-96)**	**GPS/ Geoid 1997 in m**	**Official height in m in NN1954**	**Difference in mm (NN1954-GPS/geoid)**	**Mean diff. in mm**
North side of the fjord:						
K9N42	57.272	35.498	21.774	21.781	7	
K9N50	39.045	35.723	3.322	3.336	14	
K9N49	38.130	35.722	2.408	2.423	15	
					Mean:	12
South side of the fjord:						
K9N1	37.662	35.284	2.378	2.507	129	
N9N4	43.652	35.182	8.470	8.605	135	
K9N64	37.127	35.522	1.605	1.751	146	
K9N66	44.928	35.440	9.488	9.590	102	
K9N66	44.909	35.440	9.469	9.590	121	
					Mean:	127

GPS-observations together with the geoid(VREF1996) shows a mean difference of 11,5 cm from the official heights between the north and south side of the fjord.

8 CONCLUSIONS

It looks like we have found a gross error of approximately 12 cm in the levelling at the fjord crossings over the Hadselfjord. The fjord crossings themselves might be accurate, but the short levelling over the islets may be faulty.

In 1968 a 7 km line was levelled from Kabelvåg to Svolvær. The absolute heights were based on mean sea level at Kabelvåg for the period 1948-1966. These heights are 12 cm lower than the official heights in NN1954 from 1982 for the common benchmarks indicating the same error.

GPS together with geoid information can locate gross errors of a dm in the levelling/fjord crossing. If we "correct" the official height at Kabelvåg, we get a difference of approximately 5 - 10 cm between Kabelvåg and the nearest gauges, which are smaller differences. However there might still be other errors? Figure 5 shows the annual mean sea level after mean sea level in Kabelvåg is "corrected" with the 11,5 cm "correction" found from the observations. Figure 5 shows the annual mean sea level at Kabelvåg and the other gauges after a "correction" of Kabelvåg with 12 cm.

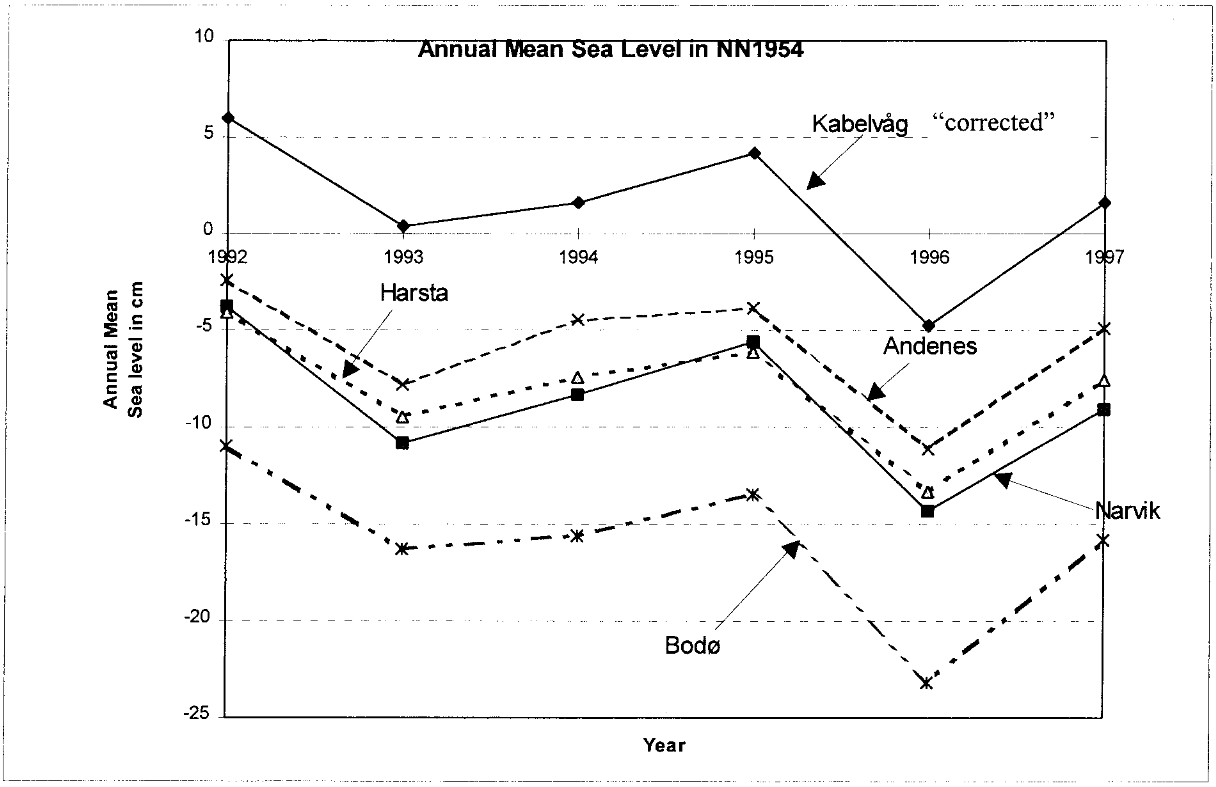

Figure 5. Annual Mean Sea Level 1992-97 at the five tide gauges after Kabelvåg is "corrected " with 11.5 cm.

Future:

In the summer of 1998 new levelling are planned at the fjord crossing area. Then we will see if my conclusion is right or wrong.

In Norway we have only one tide gauge which is not yet connected to the levelling network. Levelling to this gauge at Honningsvåg, Finnmark is planned in the course of next year. The Road Authorities now build a tunnel under the seabed to Honningsvåg. The new line out to Honningsvåg is also an open line.The experience from the levelling in Lofoten teaches us that we must use a strict quality control in the levelling of open lines. May be we should use GPS-observations together with VREF1996 as a check on possible gross errors in the levelling of the open lines in North Norway?

Proverb: Errors should never be pushed under the rug.

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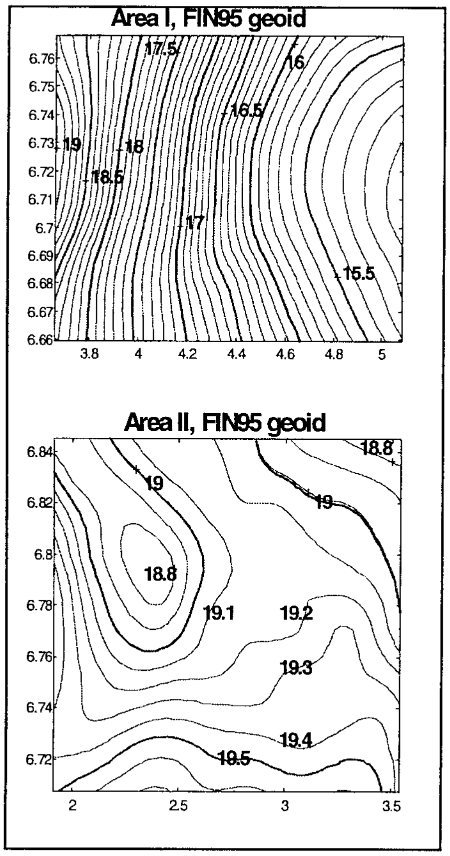

Fig. 3 Geoidal heights of the FIN95 geoid in Area I and Area II. Contour interval: 0.1 m.

1.3 GPS observations

The GPS sites observed in Areas I and II are shown in Fig. 2. The number of sites observed in Area I was 51, and the area of the network was approx. 9000 km^2 or one GPS site per 158 km^2; the average distance between stations is thus 13 km. The size of Area II was approx. 10 000 km^2, and the number of sites was 45, or one GPS site per 222 km^2; the average distance between sites was 15 km.

The GPS observations were made using 6 Ashtech Z-12™ type receivers. In Area I the observations were performed during a period of 6 days. The lengths of the observation sessions were two hours only. In Area II, the distances between observation sites were slightly greater than in the first area, which is why the observation sessions were prolonged to three hours. In Area II the observations were performed during a period of 8 days.

1.4 Solution of the GPS observations

The GPS observations were processed using two software packages: GPPS Ver. 5.0 software (ASHTECH 1993) and Bernese Ver 3.5 software (ROTHACHER *et al.* 1993). The Precise Ephemeris obtained from the University of Bern (IGS/Centre for Orbit Determination in Europe) were used with both software.

The final solution with the GPPS software was obtained using the Lc option (ionospheric-free), and the widelane solution. All possible baselines were solved in each session, and finally all baselines with fixed ambiguities were adjusted by the FILLNET program (ASHTECH 1992).

1.5 Comparison of the GPS solutions

Solutions with GPPS were undertaken to obtain preliminary coordinates for the stations. To determine the real accuracy obtainable with a commercial software package, the GPPS coordinates were compared with the Bernese solutions. The largest height difference between the GPPS and Bernese solutions is -20 mm. The RMS of the differences is ± 4 mm in Area I, and ± 5 mm in Area II.

1.6 Conversion of ellipsoidal height differences to orthometric height differences

The orthometric height differences between the benchmarks were computed according to the following formula:

$$\Delta H_{GPS} = \Delta h_{GPS} - \Delta N \qquad (1)$$

where Δh_{GPS} is the ellipsoidal height difference observed by GPS and ΔN the geoidal height difference. To determine the orthometric heights obtained with GPS levelling (H_{GPS}) in the same reference frame as the levelled orthometric heights (H_{Lev}) the height differences were added to the orthometric height of the initial benchmark, H_o:

$$H_{GPS} = H_o + \Delta H_{GPS} \qquad (2)$$

The height differences obtained with GPS were reduced for the land uplift from the epochs of the observations to the epoch of the national height system (1960.0) using the following formula:

$$dH_U = (1960 - T) \cdot \lambda \qquad (3)$$

where T is the epoch of GPS observations and λ the land uplift (mm/year). λ values were obtained from the adjustment of the consecutive Precise Levellings of the loops surrounding the research areas.

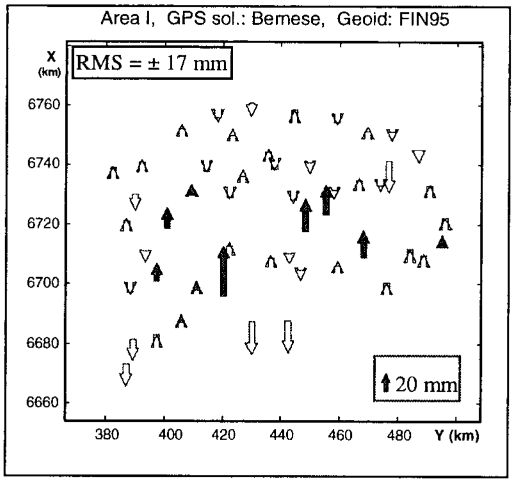

Fig. 4-1. Height difference residuals of the plane fit in Area I. The geoidal model: FIN95.

1.7 Comparison of GPS-levelled heights with spirit-levelled heights

The GPS levelling results obtained with four different geoids were compared with the spirit levelled heights. The height differences showed in all cases a clear tilt, which is why the systematic part of the differences was removed by fitting first and second order surfaces to the differences.

The following conclusions may be drawn according to the RMS of the residuals of the plane fit. The RMS values of the residuals clearly describe the errors in the geoidal heights used. In both areas the best result was achieved by using the NKG96 and the FIN95 geoidal heights. When the FIN95 geoid was used, the RMS of the residuals was ± 17 mm in Area I and ± 14 mm in Area II. The result obtained with the NKG96 geoid was even better; the RMS values were ±16 mm in Area I and ±13 mm in Area II, but the global models resulted in RMS values that varied from ±38 mm to ±83 mm. The distribution of the residuals in both areas are shown in Figs. 4-1 and 4-2. The RMS values of the residuals are shown in Fig. 5. As seen in the Fig. 5 the 2nd order polynomial fit did not reduce the residuals significantly.

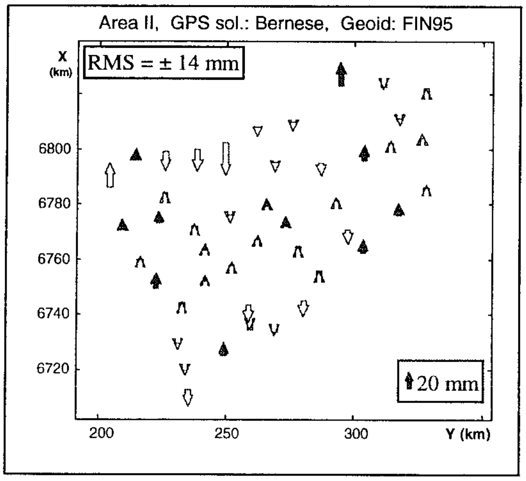

Fig. 4-2. Height difference residuals of the plane fit in Area II. Geoidal model: FIN95.

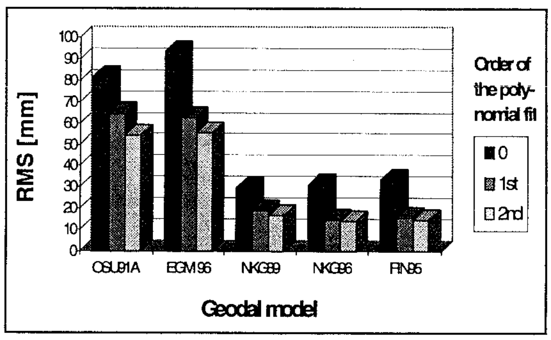

Fig 5. RMS of the height difference residuals in mm between GPS and spirit levelling in the test areas.

GPS Antenna and Site Effects

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Abstract

The improvement in precision obtained from GPS observations over recent years has revealed problems related to the local conditions at the GPS sites. In order to further improve high precision GPS positioning, orbit determination, and estimation of atmospheric parameters, investigations of site dependent effects are required. Concerns have been raised regarding the antennas and the monuments used and the long- and short-term mechanical and electromagnetic stability of the sites. Here, we review the problems associated with site-specific errors and present recommendations on how to eliminate or minimize these effects.

Introduction

During the last few years, an increasing number of permanent GPS sites have been established. The demonstrated repeatability of horizontal position estimates for regional networks is currently of the order of 2 mm and typically a factor of 3–5 greater for the vertical component. There are many advantages to continuously operating GPS networks. Stable pillars with fixed antennas eliminate errors associated with variations in the measurement of the local vector from the reference marker to the phase reference point of the antenna. For fixed pillars in a continuously operating network, the reference marker is usually a fixed, well-defined point on the antenna. In addition, denser position estimates (spatially and temporally) decrease the statistical uncertainty of the results. Continuously operating networks may also serve as a global or regional reference frame for different types of regional and local surveys. Another essential advantage is the increased ability to study and eliminate unmodeled systematic effects on daily estimates of site positions, both short- and long-term effects.

To be able to constrain the common mode of motion, sometimes in the submillimeter range, in a regional or local network a strong reference network is needed. The origin of the reference frame must be maintained with a high degree of robustness. In addition, orbits must be compatible with the reference frame. For this purpose data from the IGS network and other permanent network need to be regularly examined in detail. Site-specific errors at permanent stations may introduce errors in the determination of satellite orbit parameters and in the estimate of site positions. In the following we adress the problem of site-specific errors and present some recommendations on how to handle these errors.

Site Specific Errors

We have chosen to divide the site specific error sources into three subgroups. The first group consists of problems associated with the receiver, antenna, radome, and the signal. These are effects that will not, in general, change on a day-to-day basis. However, they might introduce biases in the solution. As long as nothing changes the effect stays the same. If something changes, such as the satellite constellation or the elevation cut-off angle, the results will be affected. The influence is especially obvious looking at the estimates of the vertical components and precipitable water vapour where bias terms can be introduced (e.g. [*Elósegui et al.*, 1995]; [*Niell et al.*, 1994]; [*Niell*, 1996]). Such a bias could seriously affect the interpretation of the GPS data.

The second group represents areas of site effects that will vary, but will only periodically affect the measurements. Precipitation, multipath, atmospheric pressure loading, and atmospheric gradients are probably the most important of these, but others may be discovered.

Finally, the third group consists of errors that might affect the long-term stability of a site such as the location of the site, ground, and the monument. Most of the material related to this specific group of site errors are rather new. These errors may seriously affect the reference frame and the geodynamical projects.

GPS Antennas

It has been found that antenna-to-antenna phase differences can introduce range biases at the several centimeter level, which may limit the precision of the measurements [*Rocken*, 1992]. Differential phase errors due to GPS antennas will not only affect the precision in GPS networks with different types of antennas, but also in networks using identical antennas if the network covers a large spatial area (baseline lengths $\gtrsim$1000 km) [*Schupler and Clark*, 1991]; [*Schupler et al.*, 1994]. Differential phase errors in regional networks (baseline lengths $\lesssim$1000 km) using identical antennas are dependent on the electromagnetic environment around each individual antenna.

The problem of antenna mixing was addressed at the IGS Analysis Center Workshop in Silver Spring, 1996. Two sets of phase calibration corrections (PCC) tables have been put together based on material presented by *Mader and MacKay* [1996], *Rothacher and Schär* [1996], and *Meertens et al.* [1996a] to be used by the IGS Analysis Centers and others in the GPS community: (1) a set of "mean" phase center offsets and (2) a set of elevation-dependent PCC and offsets relative to the Dorne Margolin T antenna.

Since the PCC values are all relative to the Dorne Margolin T antenna some effects of antenna mixing still remain. Even with the same type of antenna the variation in the apparent phase center as a function of elevation angle will influence the results on longer baselines. Therefore the task of getting absolute calibration of the antennas through, e.g., chamber measurements or simulation software may be essential for some applications even though these calibration values most likely will change when the antenna is deployed in the field.

Effects like these can of course be reduced by utilizing antennas less sensitive to scattering from external structures. One way to achieve this is to reduce the side- and back-lobe levels of the amplitude patterns by means of well designed ground-planes. For this purpose new antenna designs have been proposed (see e.g., [*Alber*, 1996]; [*Ware et al.*, 1997]; [*Jaldehag*, 1995]; and [*Clark et al.*, 1996]). Futhermore, several groups are currently developing methods to perform absolute field calibration of antennas (see e.g., [*Wübbena et al.*, 1996]) and insitu calibration of antenna/pillar systems.

Antenna-Pillar System and the Signal

Here we concentrate on the site-dependent error associated with the electromagnetic coupling between the antenna and its nearby environment (e.g., [*Tranquilla*, 1986]; [*Tranquilla and Colpitts*, 1988]). The total electromagnetic field of an antenna which radiates a signal in the presence of conducting structures may be expressed as a superposition of the transmitted field and the fields scattered (i.e., reflected and diffracted) by the structures. By reciprocity, the same is true for a receiving antenna. The significance of the scattered field depends on the degree of electromagnetic coupling between the antenna and the scatterer, that is, the distance to the scatterer and the size and reflectivity of the scatterer. Signal scattering affects both the amplitude and phase of the received GPS signal, presumably independently at each site in a network. This independence creates differential phase errors.

Scattering from structures in the vicinity of the antenna effectively changes the antenna phase pattern, and, thus, affects the precision of the carrier phase measurements of the GPS signal. In studies by *Elósegui et al.* [1995] and *Jaldehag et al.* [1996a] it was shown that estimates of the vertical component of baselines formed between sites using identical antennas were dependent on the minimum elevation angle of the data processed. Both studies found that the elevation-angle- dependent systematic effect was associated with non-identical pillar arrangements, causing differential phase errors due to scattering from structures associated with the mounting of the antenna to the pillar, and with the pillar itself. Even the most perfectly calibrated antenna the antenna phase pattern will change when attached to a pillar.

Jaldehag et al. [1996a] demonstrate that estimates of the vertical component of many baselines strongly depend on the minimum elevation angle (elevation cutoff angle) of the data analyzed. A significant part was found to be due to differential phase errors caused by scattering from structures associated with the mounting of the antenna to the pillar and with the pillar itself. As the precision and accuracy of GPS measurements improve in general, antenna phase pattern variations due to different pillars and antenna mounts could be the major error source in just a few years, if not now. Modeling of the scattering effect, or rather the complete phase response of the antenna system, including the pillar, is an important issue for future improvements of the GPS technique.

Radomes - Protective Covers

At several permanent GPS sites located in areas with periodically severe environmental conditions (snow, rain) radomes have been employed. Until recently, most radomes in use have had a conical shape.

All materials have some effect on a electromagnetic wave. Radomes appear to delay and refract the GPS-signal in a similar way as snow [*Jaldehag et al.*, 1996b]. Several groups have recently been investigating effects due to the excess signal path delay through the radome. Different radomes have been tested in anechoic chambers [*Clark et al.*, 1996]; [*Meertens et al.*, 1996b] as well as in field tests [*Meertens et al.*, 1996b]; [*Jaldehag et al.*, 1996c]. All tests show that a conical cover may cause cm-level vertical errors when the tropospheric delay parameter is estimated. The recently employed hemispheric radomes seems to show much less elevation dependence. The influence on the tropospheric wet delay estimates and subsequently, the vertical component will only be on the 1-2 mm level. We can conclude that all radomes effect the GPS signal at some level and in form of an excess signal path delay which will map into other parameters in the GPS software. The effect of the protective covers can most likely be misinterpreted as a tropospheric effect in a similar way as snow. The effect is more or less constant and may be calibrated or modeled.

Precipitation

Signal propagation delay during snow storms has been investigated by, e.g., *Tranquilla and Al-Rizzo* [1993] and *Tranquilla and Al-Rizzo* [1994] who demonstrated that due to the localized nature of many snow storms differential effects may cause systematic variations at the centimeter level in estimates of the vertical coordinate of site position. Systematic variations introduced by snow storms may, however, if short-lived (minutes to hours), be reduced to a high degree by data averaging. A potentially more serious effect of heavy snow precipitation is the accumulation of snow on the top of the GPS antenna and on its surroundings, such as on the top of the GPS pillar or, when present, on the radome covering the antenna. This accumulation may last for days, weeks, or months. *Webb et al.* [1995] reported variations on the order of 0.4 m in estimates of the vertical coordinate of site position. The variations were correlated with the accumulation of snow over the antenna. Variations at the several centimeter level in estimates of the vertical coordinate of site position strongly correlated with changes in the accumulation of snow on top of GPS antennas have also been observed by others [*Jaldehag et al.*, 1996b]; [*BIFROST project members*, 1996]; [*Meertens et al.*, 1996a]. The results indicate that the variations in the vertical coordinate of site position can be fully explained by reasonable accumulations of snow which retard the GPS signals and enhance signal scattering effects.

Horizontal Atmospheric Gradients and Air Pressure Loading Effects

In the data processing the atmosphere is normally considered to be spherically stratified. We assume that one equivalent zenith wet delay value determines the wet delay in any direction, given a certain elevation angle. More advanced models, using more parameters to describe the atmosphere, have been proposed as alternatives to this very simplified model (e.g., [*Davis et al.*, 1993]; [*MacMillan*, 1995]). Several groups are now implementing possibilities to estimate horizontal gradients in the software [*Bar-Sever and Kroger*, 1996]; [*Chen and Herring*, 1996].

The lack of pressure data available during the GPS analysis can be the reason for different errors. During the entire GPS processing we have to model many external and internal effects on the crust of the earth. One effect currently not modeled is the pressure loading. The vertical position of the GPS receiver changes due to different atmospheric pressure loading the Earth [*vanDam and Herring*, 1994]. Extreme values could affect the vertical component of the GPS estimates on the cm level. These effects are of course related more to the general presssure field in the region rather than to a specific site. To properly model this effect a grid of pressure data has to be available. Unfortunately, it is very difficult to isolate these effects from other elevation-angle-dependent effects (multipath, scattering, snow/ice, etc.). Small variations in the vertical component are also caused by these other errors. We are thus not in the position of being able to correct for horizontal atmospheric gradients and loading errors optimally. At this point, theoretical studies are needed to quantify these effects, and to understand how we can best deal with these problems.

Local Stability and Monumentation

As GPS measurements have become more precise and are more frequently acquired, the issue of monumentation and site stability has become more important. The long-term contribution to the maintainance and densification of the global reference frame could be seriously affected by instable sites. The IGS network consists of a large variety of monuments established on top of everything from solid bedrock to buildings. The long-term stability of the reference frame and products associated with it, such as the orbits, are at issue here. Much attention is currently focused towards motions of geodetic monuments. These motions have been found by some researchers to be random-walk-like (e.g. [*Johnson and Agnew*, 1995]; [*Johnson et al.*, 1996]) while others find no evidence for random walk behavior (e.g. [*Mao et al.*, 1996]; [*Davis et al.*, 1996]). An ideal GPS monument would move in response only to the tectonic motion of the Earth. However, location, ground, and the environment at ground surface can have dramtic impact on the long-term stability of a site. The implication of this type of power-law noise is serious if the data are used to estimate low-frequency characterisitics of a time series such as the slope (deformation rate). *Mao et al.* [1996] and *Davis et al.* [1996] found no tendency of a random-walk like behavior possibly because the records where not long enough to see a random walk component above the noise in the low portion of the signal. It is also quite possible that monument motion may depend critically on the monument design and the site locations.

Nevertheless these investigations will continue and are most effectively addressed using continuous GPS measurements gathering data in a large variety of local conditions and GPS satellite configurations. There are design techniques which can be employed to mitigate this unwanted influence, most of which involve anchoring the monument to several points at depth and isolating the monument from surface material. Detailed spectral analyses and examination of the long time series available for some of the global sites. Monument and local stability problems could also manifest themselves with a periodic behavior, and be correlated with atmospheric conditions and precipitation.

Conclusion

Site-specific errors cannot be separated out when data from the global IGS sites are being used to determine orbits and reference frame. To be able to constrain the common mode of motion, sometimes in the submillimeter range, in a regional or local network a strong reference network is needed. The origin of the reference frame must be maintained with a high degree of robustness. In addition, orbits must be compatible with the reference frame. For this purpose the IGS sites need to be better examined. We especially found that the problems associated with the antenna-pillar system and the signal distortions have to be addressed. The effect of the antenna and signal related errors are constant from day-to-day but are biasing products like the orbit determination, station time series, and precipitable water vapor time series. Any changes either at a station or in the GPS-data analysis strategy might change this bias and thereby affect the daily products and the reference frame. The other important issue that needs attention is the long-term stability of the sites and the monuments used in the IGS network. This is especially important bearing in mind that local and regional continuously operating GPS networks are now used to detect motion at the level of 1 mm/yr or less.

Acknowledgements

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Experiences of Automated Calibration of Levelling Rods in Finland

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Abstract

An automated vertical laser rod comparator was constructed at the laboratory of the Finnish Geodetic Institute during 1996 - 1997. The comparator uses a CCD-camera with an area sensor for determining the position of the rodmarks. The HP Laser Interferometer 5527A constitutes the length standard and a stepping motor moves the rod on a linear conveyor. The humidity, air pressure and temperature are measured by an automatic weather station. The hardware and all processes are coordinated by the PC. This comparator is capable of calibrating classical rods as well as modern bar code rods. The accuracy achieved is higher than 1 p.p.m.

Introduction

The first report of the automated vertical laser rod comparator of the Finnish Geodetic Institute (FGI) was given at the NKG meeting of the Height Determination Group in Hønefoss, Norway in spring 1997. The construction and the principle of the comparator has been described in the publication by TAKALO (1997) (Fig. 1). The goal of this comparator is

to calibrate the levelling rod in vertical position,

to calibrate particularly the rods of the FGI used in the Third Precise Levelling and

to improve the resources of the FGI as a national standard laboratory for geodetic length.

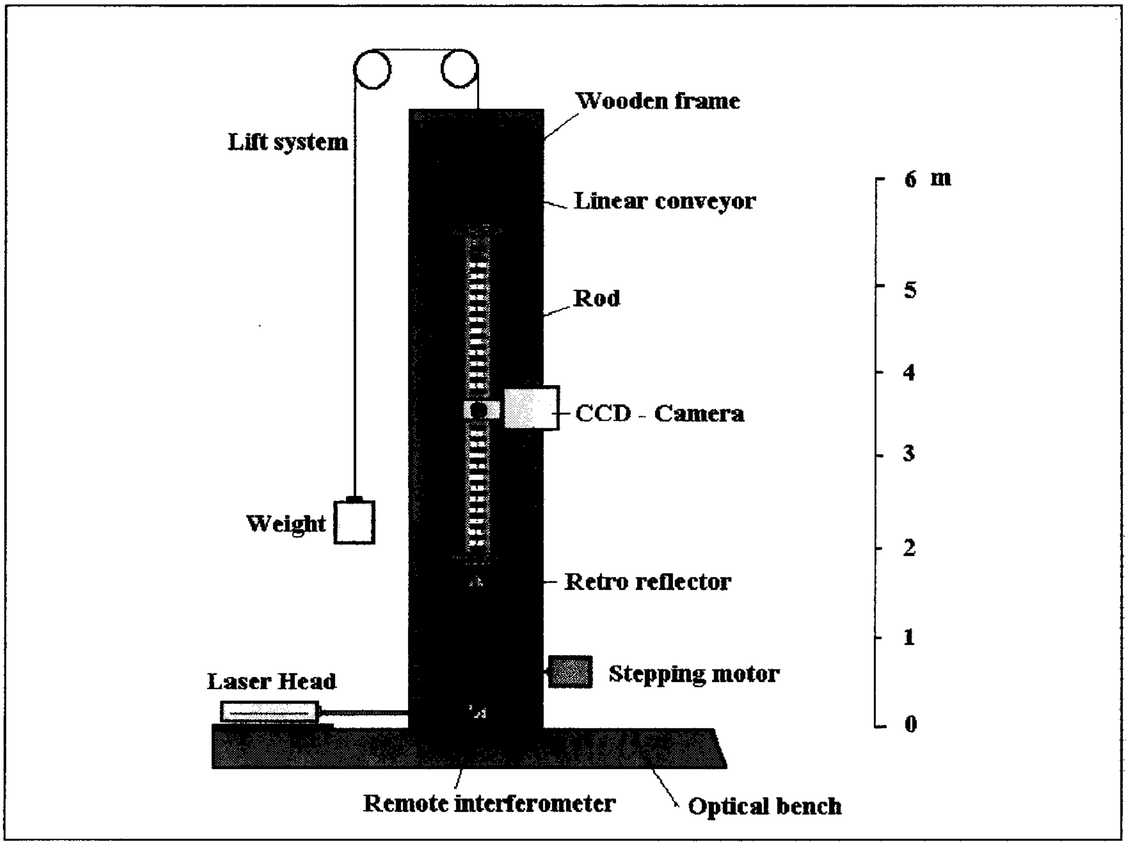

Fig. 1. The vertical laser rod comparator of the Finnish Geodetic Institute.

Since spring 1997 several tests and practical calibration measurements were carried out.

This report gives a short description of

problems of edge detection in rod calibration,

improvements of comparator construction,

malfunction of rod compensator and

calculation method to correct readings of the Wild digital level with corrections of bar codes.

Problems of edge detection

The laser rod comparator uses a laser interferometer as a length standard and a CCD-camera for determining the position of the rod line. To achieve a high accuracy in positioning, a CCD-camera with an area sensor, a PC board, image processing software and an efficient method are needed to detect edge lines (Fig. 2).

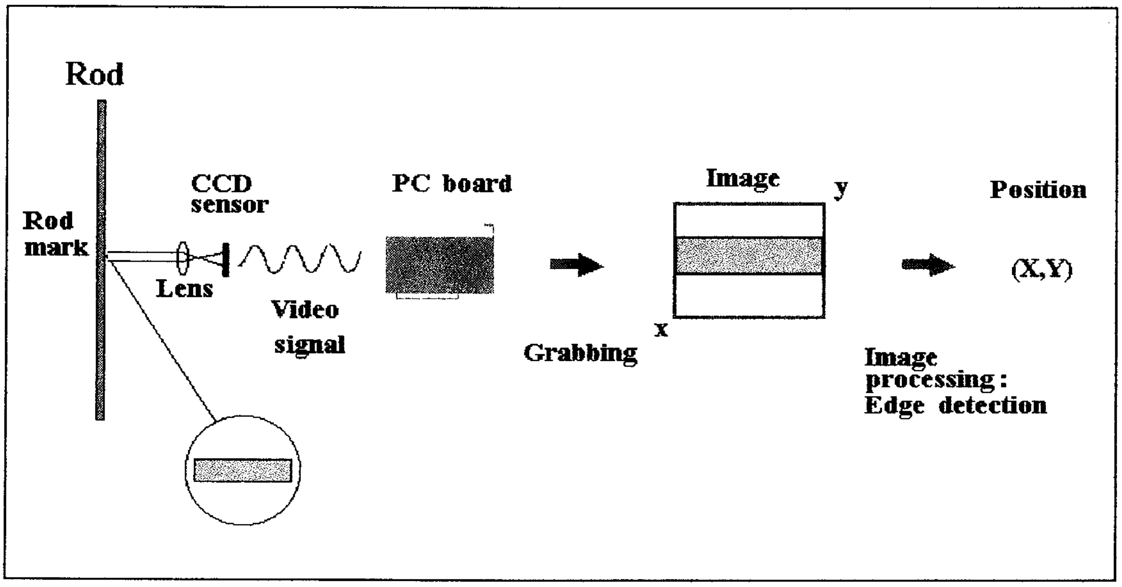

Fig. 2. Determination of the position of one rod line.

The digital image grabbed by the CCD-camera consists of pixels, each containing information on position and grey value as illustrated in Fig. 3.

At first glance the determination of the position for rod line seems to be a simple task. If we regard the image data column as a simple discrete location function G(x), the line (edge) can be detected by positioning the extreme value of the first derivate G'(x) (Fig. 4).

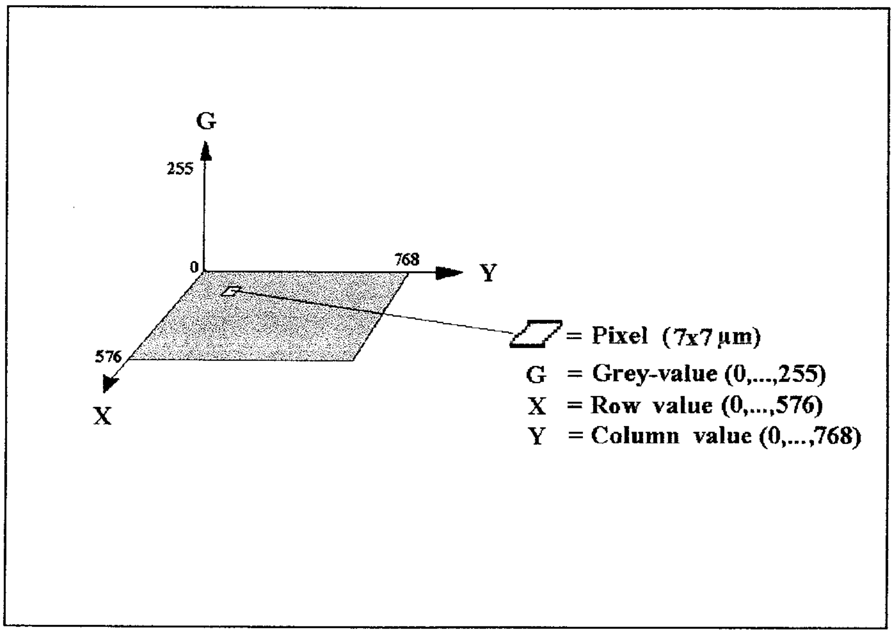

Fig. 3. Coordinate of image data.

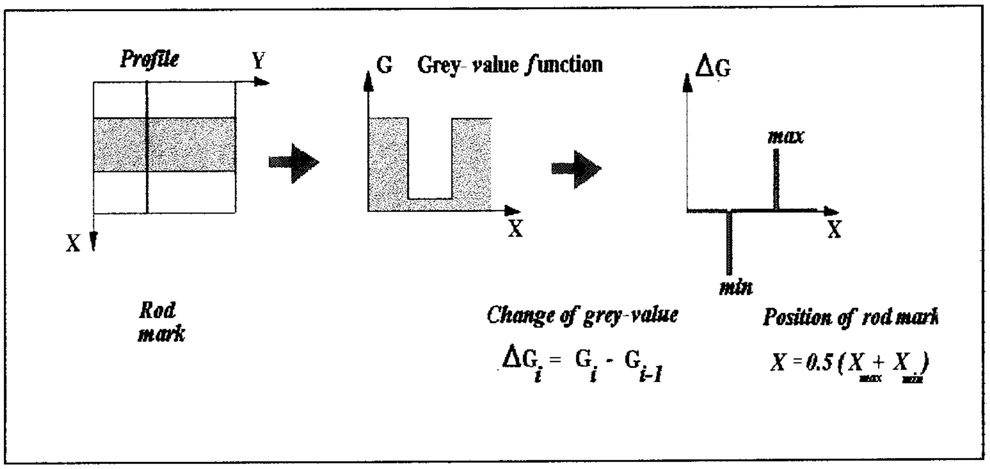

Fig. 4. Edge detection.

The reality, however, is not so simple, because the image data includes always noise. Especially graduation lines of old rods are often worn out i.e. their digital image is noisy. Therefore the location function can be presented as follows:

(1) $G(x) = s(x) + \varepsilon \sin(\omega x)$,

where $s(x)$ = signal function and
$\varepsilon \sin(\omega x)$ = noise function.

Hence we get by derivating (1)

(2) $G'(x) = s'(x) + \varepsilon\omega \cos(\omega x)$,

where the derivative of the noise function is very often much larger than the derivative of the signal itself

(3) $s'(x) << \varepsilon\omega \cos(\omega x)$.

This can be also easily proved from the real signal of a rod line (Fig. 5).

The determination of the position of a rod line using the comparator of the FGI is done in two phases: First the preliminary and then the accurate determination. In order to avoid the treatment of the whole image data (0.5 Mb), the first approach gives the narrow zones around the edge, which data is then used for the accurate determination.

To adjust the influence of poor edge sharpness and electronic noise caused by the CCD-camera in accurate determination, the grey values of the pixels from column 75 to 675, are summated by rows (Fig. 6). The edge is then determined from this summated data using a 3-order-polynomial-mask-filtering method. This gives the position in single pixel level accuracy, and the part pixel accuracy can then be achieved by fitting a parabola to the data in the narrow area around the found position.

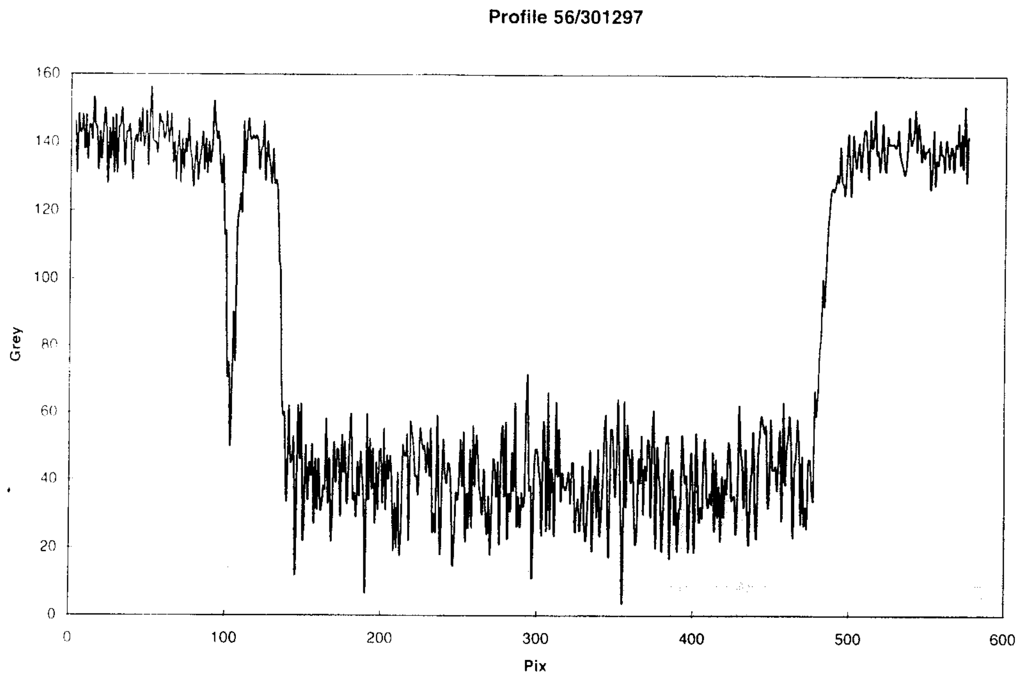

Fig. 5. A image signal of a rod line.

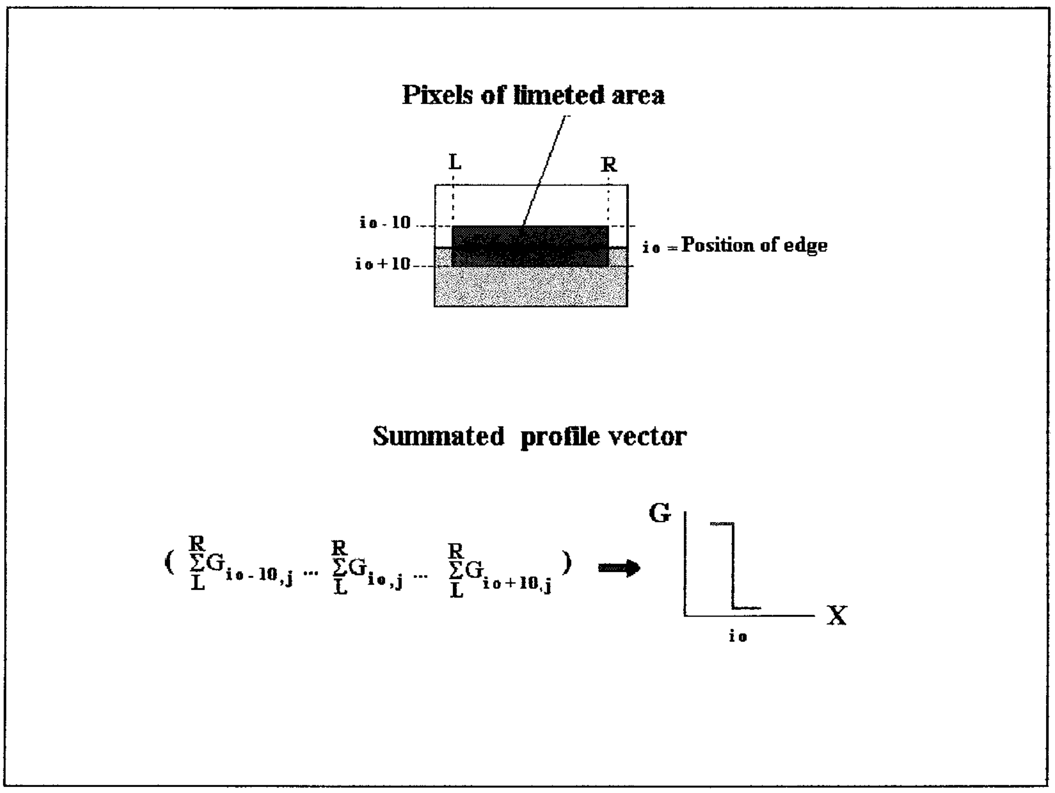

Fig. 6. Processing of image data.

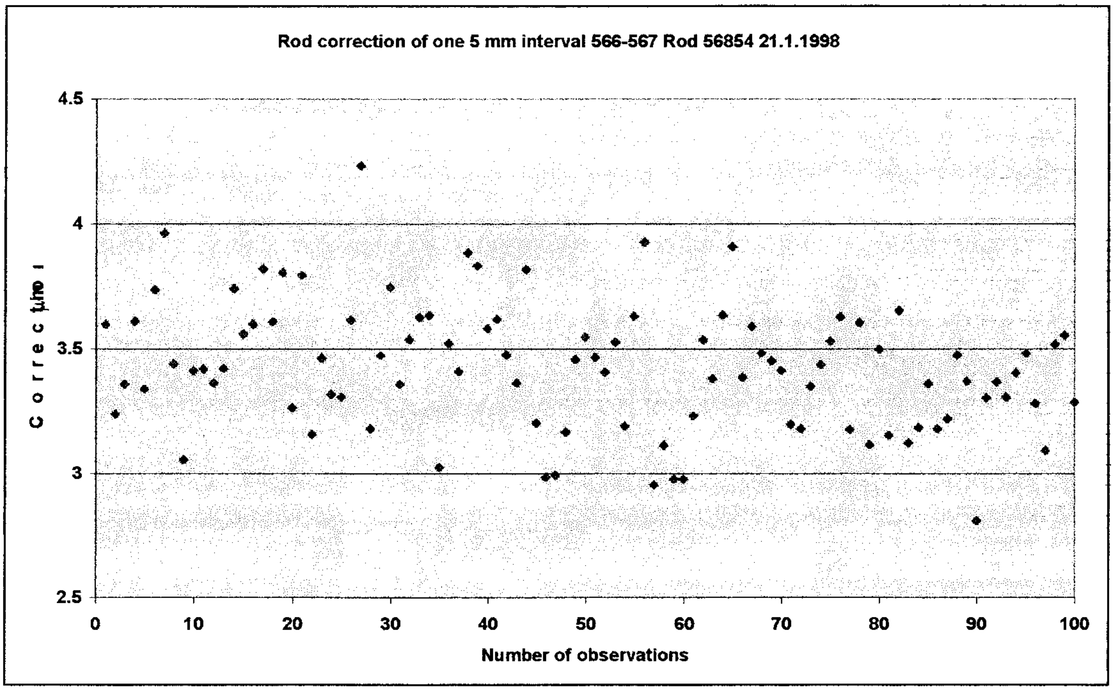

Fig. 7. The length correction of one 5 mm interval. The number of observations is 100 and the accuracy obtained ±0.2 µm.

An example of the high accuracy achieved using this technique is depicted in Fig. 7.

Improvement of the comparator construction

The FGI's laboratory in Masala, especially designed for rod calibration, is partly 8.7 m high for the vertical comparator and 5.2 m high for the horizontal comparator, 2.7 m broad

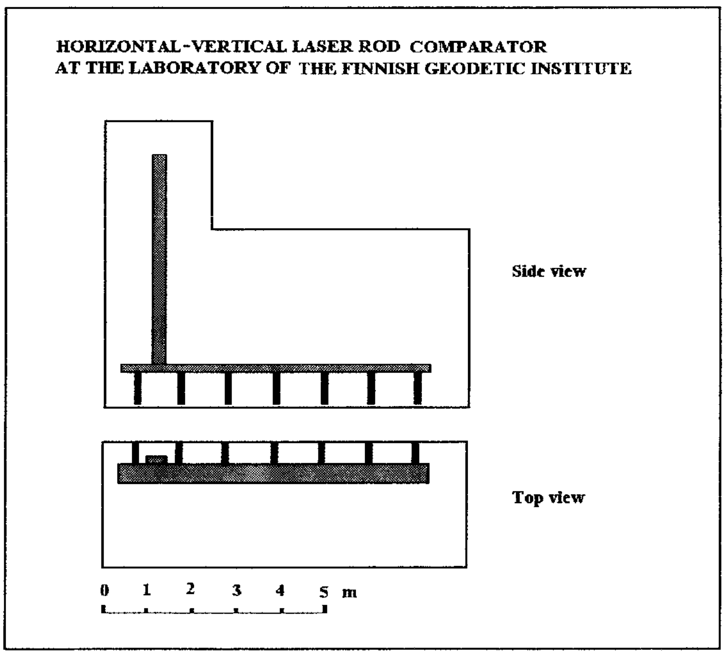

Fig. 8. The comparator room of the FGI.

and 10 m long. The location of the comparator frame is shown in Fig. 8.

The initial vertical temperature gradient of the laboratory was +1°C / 1 m. By eliminating all potential sources of heating e.g. lights, power sources, PC etc. and by covering appr. 10 m^2 of the floor below the vertical comparator with polyurethane foam plates, the vertical temperature gradient was reduced to be less than +0.2°C.

As a consequence the temperature and humidity sensor of the automatic weather station could be located approximately 1.5 m above the remote interferometer (Fig. 1) and near the path of the laser beam. The measured weather parameters thus represent well the mean circumstances for the laser beam.

The illumination of the object (graduation line), which is critical for successful determination of its position with the CCD-camera, was improved by two low powered neon lamps and three surface mirrors as shown in Fig. 9. The lamps give an optimal illumination for the object and do not warm up the CCD-camera.

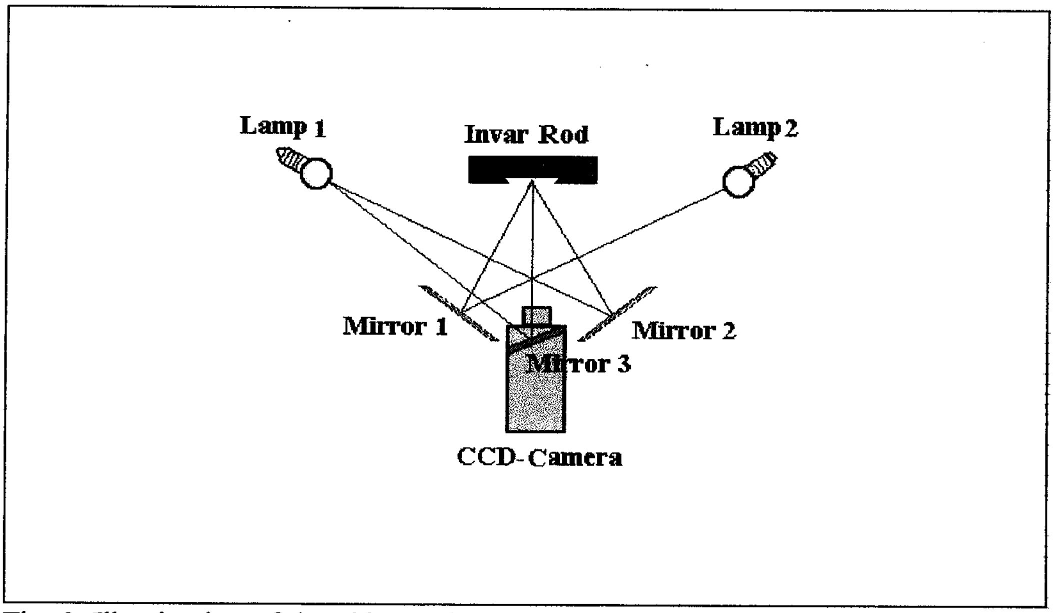

Fig. 9. Illumination of the object. The lamps are approximately 1 m removed from the CCD-camera.

Malfunction of rod compensator

In invar rods the upper end of the invar band is fastened to the frame of the rod with the aid of a rod compensator. The aim of this construction is to keep the stress of the invar band constant.

There are many types of rod compensator, used in the precise levelling rods, the most common are shown in Fig. 10.

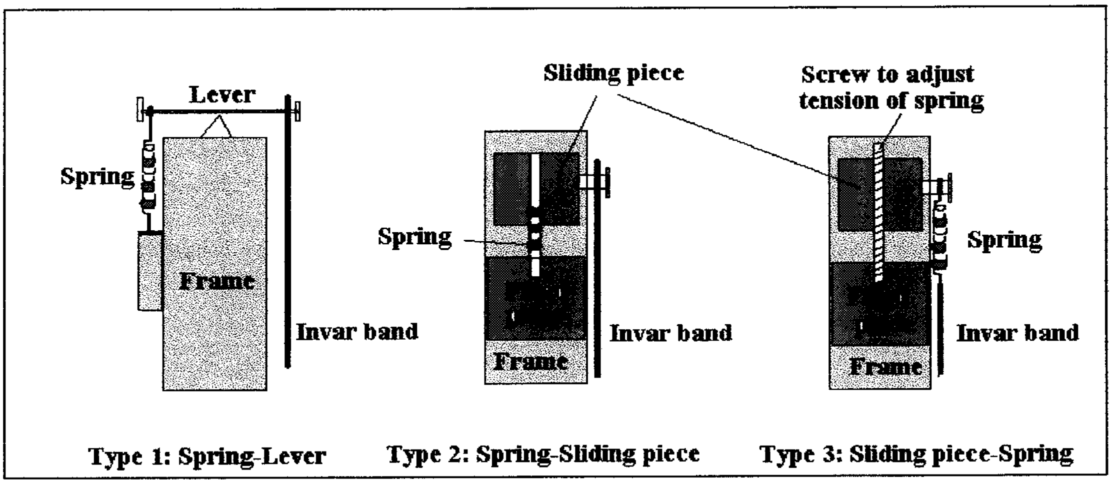

Fig. 10. Main types of rod compensator.

The type 1 is typical in older invar rods with wooden frames, the type 2 was used together with aluminium frames in the 80's and the type 3 is the modern solution typical for rods with aluminium frames manufactured in the 90's e.g. in bar code rods.

In the precise levelling, invar rods with aluminium frame are more often used than those with wooden frame. The aluminium frame is light and less sensitive against deformation, but its disadvantage, however, is the very high thermal expansion coefficient of 23.8 p.p.m / °C, much different from the invar band. The aluminium frame can change its length even 2 mm during the field work season. Therefore a malfuncion of the rod compensator can be very fatal due to the untraceability of the length of the invar band during the field work.

The malfunction of the rod compensator can be tested in connection with determination of the thermal expansion coefficient by measuring the length of the invar band at different temperatures. In these tests only great differences of temperatures ensure the detection of possible malfunction. Also the distance to be measured must be long enough.

In this spring, 1998, the thermal expansion coefficients of 5 pairs of invar rods with aluminium frames were determined at the laboratory of the FGI in vertical position. The rod compensator of three pairs were type 3 and two type 2 (Fig. 10). The ambient temperature of the laboratory could be varied by the airconditioner. The observations were accomplished in a cycle 20°C → 27°C → 37°C → 27°C → 20°C → 15°C → 7°C → 15°C → 20°C.

The measured distance for the classic rods was 2.885 m and for the bar code rods 2.930 m. A regression analysis was made of the observations.

The linear expansion coefficients obtained varied from 0.64 p.p.m/°C to 0.90 p.p.m/°C. All other rods, except No. 8619 (Fig. 11) behaved normally. The standard deviation was ±0.24 p.p.m/°C in case of No. 8619, 6 times larger than obtained for other rods.

The great deviation of the regression result indicates a malfunction of the compensator of type 2 (Fig. 10). A friction between the sliding piece of the compensator and the

The great deviation of the regression result indicates a malfunction of the compensator of type 2 (Fig. 10). A friction between the sliding piece of the compensator and the surface of the housing of the aluminium frame was obviously the reason for the malfunctioning of this rod compensator, because after cleaning and lubrication the compensator worked well. That can be recognized in Fig. 11 from observations made after maintenance.

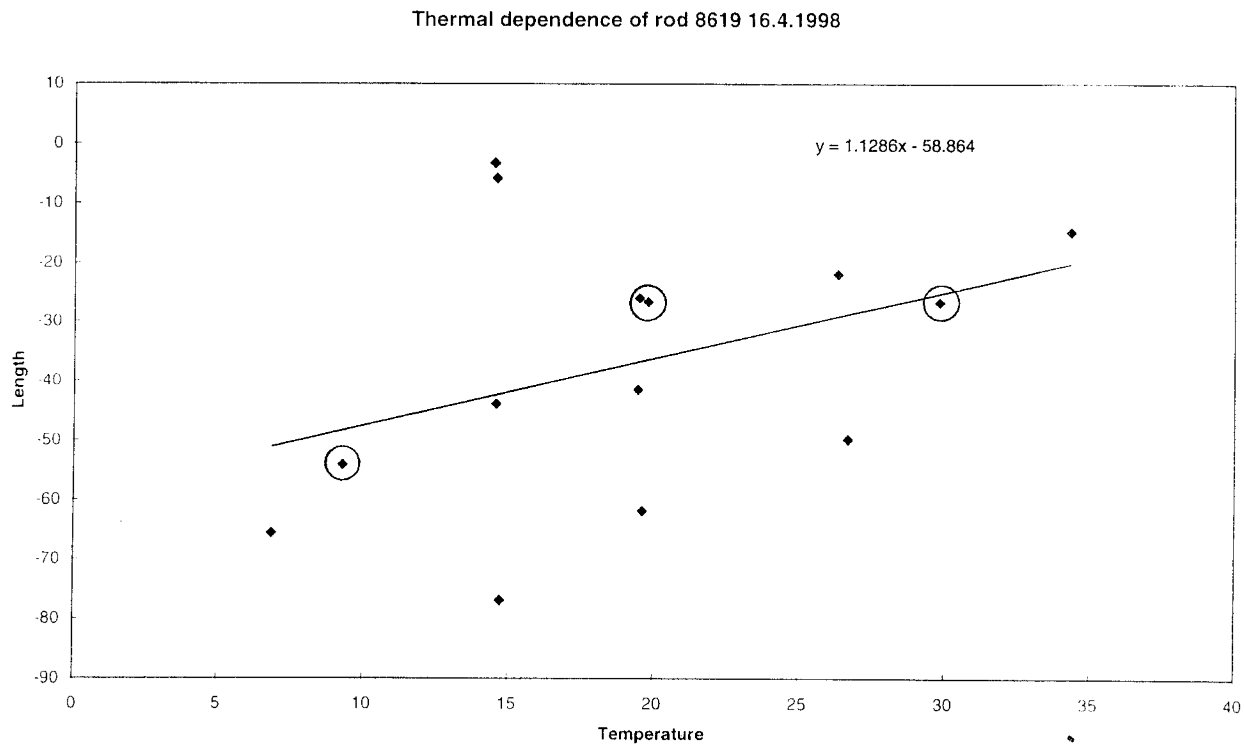

Fig. 11. Thermal dependence of the length of rod No.8619. The observations made before cleaning and lubrication of the compensator are black dots and those made after are denoted by circle around the dots.

As conclusion we can state that the compensator of the invar rods with aluminium frames must be tested and maintained regularly.

Calculation of correction for bar code rod

According to the manufacturer, each element of the Wild bar code rod is located a multiple of 2.025 mm. When calibrating the bar code scale we measure the distances from a defined beginning black-white border (edge) locating as near the bottom plate as possible. The nominal and the true position i.e. centre of each bar code are compared. The question then arises, how to use the obtained position corrections to correct a reading processed by the digital level. This problem was for the first time dealed with by INGMAR PETERSON (1991) in Sweden.

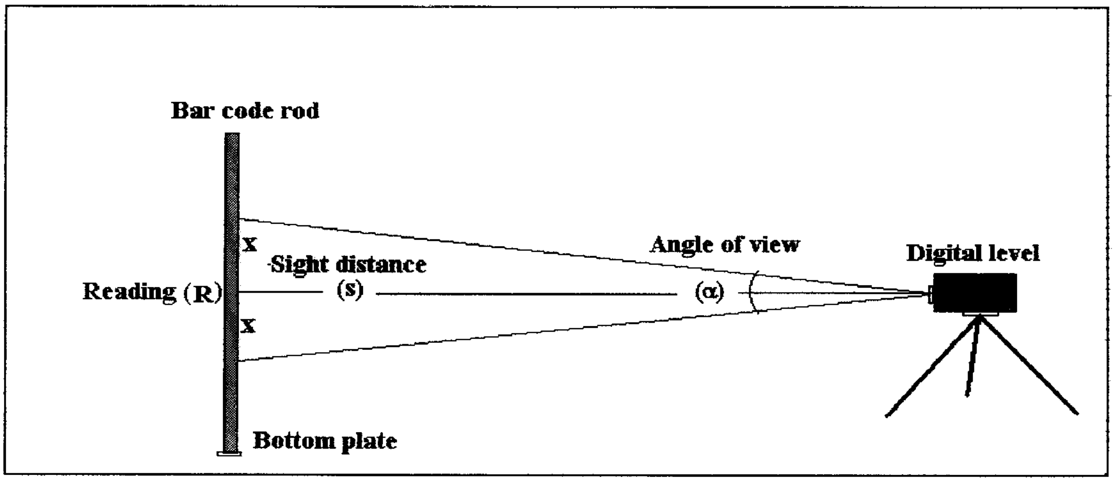

Fig. 12. The sketch of a digital level with a bar code rod.

During measurement the Wild digital level "looks at" the bar code scale in an angle of 2.2 gon (INGENSAND 1990) (Fig. 12). All bar codes within this aperture are then used to calculate the reading R_0', which is the distance between bottom plate of the rod and the point where the horizon level of the digital level crosses the rod (Fig. 12). Simultaneously, the sight distance s is measured by the digital level.

The size of the afore-mentioned limited area can be calculated with the equation

(4) $\quad x = s \tan \alpha/2$,

where $\quad \alpha$ = angle of view in gon.

The position of this area in relation to the bottom plate is

$$(R_0' - x, R_0' + x) ,$$

where R_0' is the observed reading in mm.

The calibration gives the correction table

$$\mathrm{Cor}\ (C_i),$$

where C_i is position of bar code i with regard to the bottom plate.

Hence we get the correction of the observed reading R_0'

(5) $\quad \delta R = \{\forall\ C_i \in (R_0' - x, R_0' + x) \mid \mathrm{MEAN}\ [\ \mathrm{Cor}\ (C_i)\ ]\}$

and the final corrected reading is

(6) $R_0 = R_0' + \delta R$.

Conclusion

The experiences up to now indicate that the vertical, automated laser rod comparator of the FGI is capable of calibrating both classical rods and modern bar code rods with an accuracy of 0.2 p.p.m.

In the future also the horizontal laser rod comparator will be automated. The tasks of the horizontal comparator are

to support the study of the length difference of invar rods between horizontal and vertical positions and

to calibrate special scales used only in horizontal position.

Also the malfunction tests of the rod compensators and the creating of a larger timeseries for the thermal expansion coefficient of the invar rods will be continued.

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Results of the Baltic Sea Level 1997 GPS Campaign

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Abstract

The Baltic Sea level project (BSL) was initiated in 1989, and the first GPS campaign was arranged in 1990. After that, two other GPS campaigns, in 1993 and 1997, have been arranged, the latter one combined with the EUVN (European Vertical GPS Reference Network). One of the goals of the BSL is to unify vertical datums of the countries around the Baltic Sea.

From the repeated observations one can study also the sea-level variation and land uplift of the area. We present the results of the 1997 GPS campaign, and combine them with the results of the previous ones. We also discuss on the problem of the reference frames and use of permanent GPS networks of the area to tie observations made at different epochs to the same system.

Introduction

The Baltic Sea level (BSL) project was initiated as an *ad hoc* working group in the General Meeting of the IAG in Edinburgh in 1989. After the IUGG General Assembly in Vienna 1991, it received the status of a Special Study Group (IAG SSG 5.147) and after the IUGG General Assembly in Boulder in 1995, status of Subcommission 1 of IAG Commission 8. All countries around the Baltic Sea have participated in the project.

One of the goals of the Baltic Sea Level Project is the unification of the vertical datum and studies of their time variability in the countries around the Baltic Sea. In order to achieve this, a series of repeated GPS measurements have been performed, namely in 1990, 1993 and 1997.

About 35 tide gauges around the Baltic Sea were observed with GPS and as a result of the campaigns, precise heights above the ellipsoid were computed. The network is shown in Fig. 1. The benchmarks observed were connected to the national precise levelling network and to the readings of the tide gauges. Long series of tide gauge observations give us a connection to the mean sea surface.

During the seven years from 1990 to 1997, there has been a vast development in receiver technology, precision of satellite ephemeris and data processing. Also, solar activity reached its maximum around the year 1990, whereas in 1997 the activity was still near its minimum. Thus the ionosphere activity has decreased during these campaigns. As one might expect, the increase in accuracy has been as dramatical as the change in technology and environmental conditions.

The preliminary results (Poutanen, 1998a) shows that repeatability of the 1997 campaign is about 1/10 of that of the 1990 campaign, namely 1 mm in horizontal and 2.6 mm in vertical component. As it was clearly seen, systematic errors can be considerably larger than might be expected from the repeatability. In (Poutanen, 1995b) we show an example from the 1993 campaign, where solutions of different computing groups deviates more than the repeatability of the results of one group.

In the 1997 campaign one may expect considerably smaller group-to-group variation. This is due to improved antenna phase centre tables in Bernese v.4.0 which is used in computation, as well as strong ties into fixed ITRF points, the coordinates of which are in ITRF96.

All three campaigns have been computed in different reference frames: 1990 in ITRF89 (epoch 1988), 1993 in ITRF1993 (epoch 1993.45), and 1997 in ITRF96 (epoch 1997.4). This give rise to a problem to combine the results into a common reference frame.

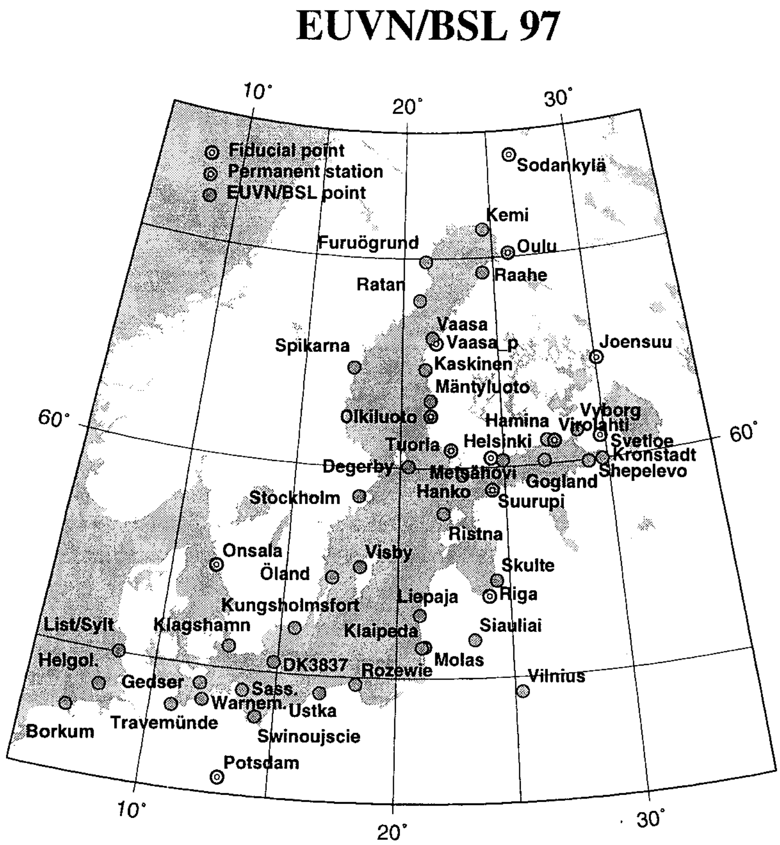

Fig. 1 *Area of the Baltic Sea Level 1997 GPS campaign*

The combined EUVN/BSL 1997 campaign

The 1997 BSL GPS campaign was made simultaneously with the EUVN in May 21–29 (DOY 141–149). The reason for combining the campaigns was that most of the points around the Baltic Sea were common to both networks. The observing sessions were 24 h. EUVN points were observed full 7 sessions whereas the points belonging only to the BSL were observed 3–4 sessions. The observing geometry is shown in Fig. 2.

Receivers were either Ashtechs, Rogues or Trimbles, most of them equipped with a Dorne Margolin type antenna (either the original one DM T, DM B, or with clone DM Ashtech or DM Trimble). On some sites original Ashtech or Trimble antennae were used. On EUVN sites permanent monumentation (masts, concrete pillars) have been used but on old BSL points the antenna was put on a tripod above the marker. A detailed description of the observing and data processing procedures will be published in (EUVN, 1998) and also in the proceedings of the BSL meeting to be held in St.Petersburg in June 1998.

Data processing was done using Bernese v.4.0 software. We computed both a "standard" 15° cut-off limit L3 solution and a 5° cut-off limit solution with elevation dependent weighting. In both cases, the QIF algorithm was used to resolve the integer ambiguities (Rothacher and Mervart, 1996). Local troposphere estimation was also done. The IGS antenna phase centre correction table was used (values relative to the Dorne Margolin B antennae).

First we computed a minimum network solution with 15° cut-off limit. Due to the geometry of the network, especially the southern part seems quite unstable and repeatabilities was not satisfied on all points. Therefore, some trivial vectors were added, to improve the geometric stability.

As previously discovered, the minimum network solution is more sensitive to bad data and thus one bad point in the middle can affect the solution of a relatively large area. Also, due to the geometry of the BSL network, it was almost impossible to avoid free ends, i.e. chains which do not end on a fixed point. The height accuracy of these points is not as good as those points which are in the chain which begins and ends on a fixed point.

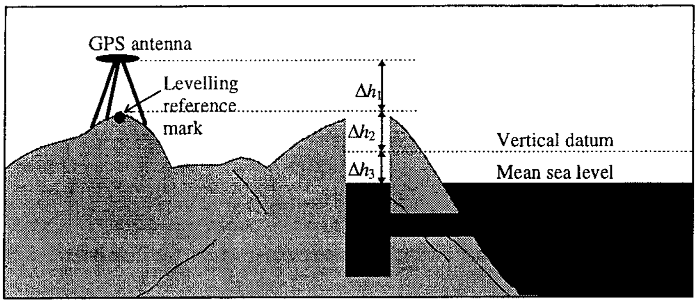

Fig. 2 *Observing geometry in the BSL campaigns.*

As expected, on most cases, the addition of trivial vectors did not make any major changes but only on some special cases where we had problems in the initial solution. We also tried to avoid "dead ends" and formed loops where a point is connected to at least on two other points. However, the overall repeatability did not change considerably.

The final computation was done in ITRF96 (1997.4). We used those permanent GPS stations in the network as fixed, the coordinates of which are given in the ITRF96 catalogue (IERS, 1997). These include both IGS permanent stations and stations belonging to the Euref permanent network. The stations are Sodankylä, Vaasa, Joensuu, Metsähovi, Riga, Svetloe, Onsala, and Potsdam. Thus the coordinates of the BSL stations are well tied to the reference frame defined by these eight stations.

The repeatability of the observations is quite good. After the final adjustment we have for the whole network the rms 1.0 mm in N and E components and 2.6 mm in height. Repeatabilities of individual stations can be seen in Fig. 3c.

An interesting feature is visible in Fig. 3c. On sites Molas and Klaipeda where we used a slightly erroneous phase centre correction table, the height repeatability is not as good as on the other stations.

The coordinate values of the sites did not change significantly from solution to another. Between the 5° and 15° solutions the heights are within 1–2 mm in most cases; with some individual exceptions up to 1 cm. This is mostly due to the heavy constraints of the fiducial sites which are situated well around the area. All the observations are so well tied to these that no major changes are possible.

We can say that the solution is well tied into the ITRF96, at the epoch of the observations. Because there are no major outliers in individual stations in the solutions, one could choose the 5° solution as the best one. But just remembering reality, deviations are so small, that the volume of the daily repeatability is smaller than the tip of your little finger. Systematic errors are undoubtedly much larger but those are not visible in this study.

Comparison of 1990, 1993, and 1997 BSL results

We have now measurements from three different BSL GPS campaigns. The time span of seven years should be long enough to see e.g. land uplift at the points. However, the change of reference frames during the years makes comparison a bit problematic.

In 1990 ITRF89 was used, in 1993 ITRF93, and for BSL97 the ITRF96 is in use. There is no clear solution to the problem. Coordinate transformation from one reference frame to another, using general, global parameters does not necessarily lead to the best solution. Although we are mostly interested in heights, even the height of the point above the ellipsoid does not remain constant if we change from one reference frame to another. Differences of more than a centimetre can exist, and this can be a function of latitude and longitude.

The ideal solution should be that the observing geometry (including selection of fiducial stations), receivers and processing strategies (and software) is kept unaltered. Unfortunately this is an unrealistic requirement. We have to overcome the problem in some other way.

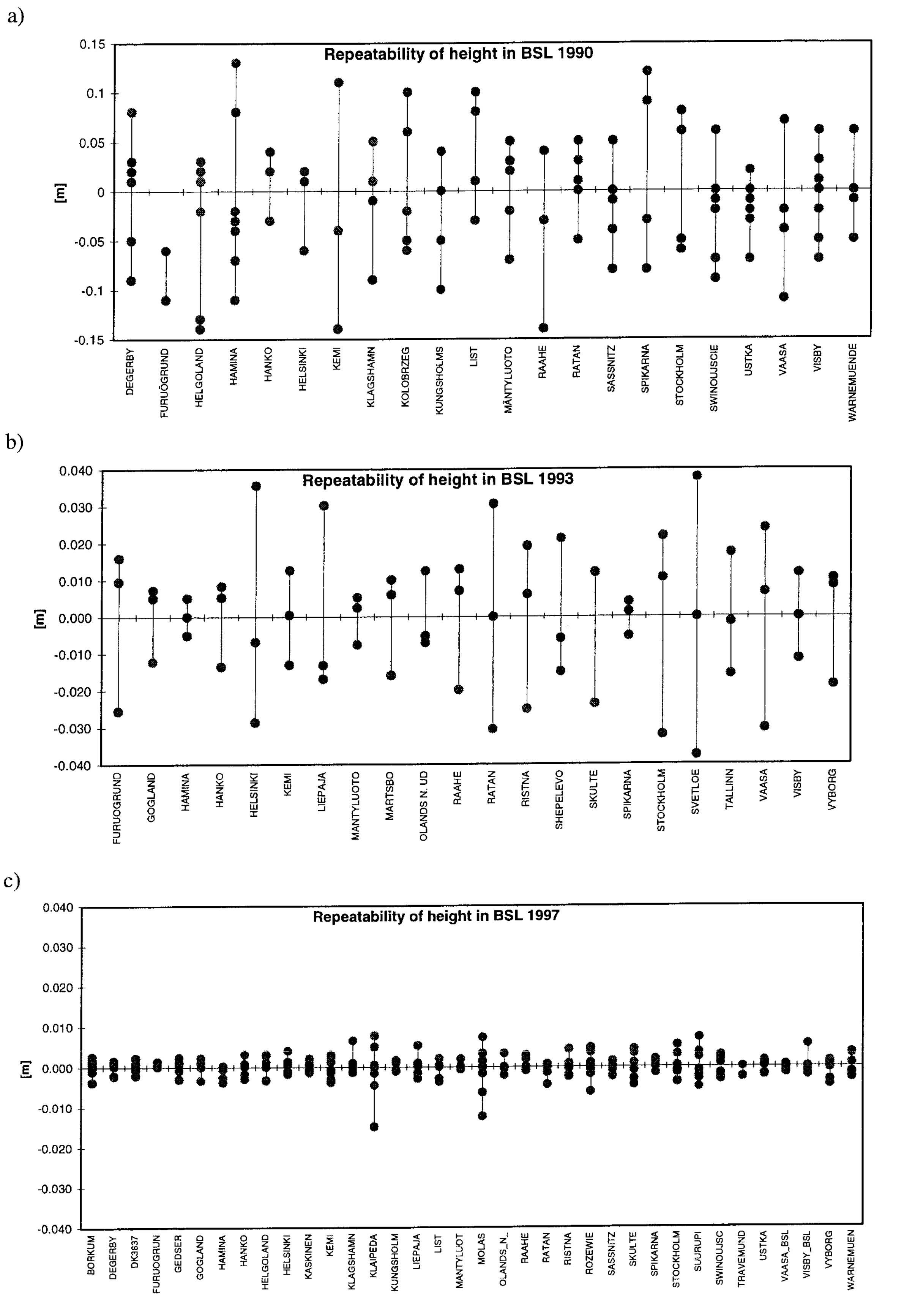

Fig. 3 *Repeatability of the height component in three BSL GPS campaigns. Note the different scale in the 1990 results. Data are from Koivula and Poutanen (1994), Poutanen (1995a) and Poutanen (1998a).*

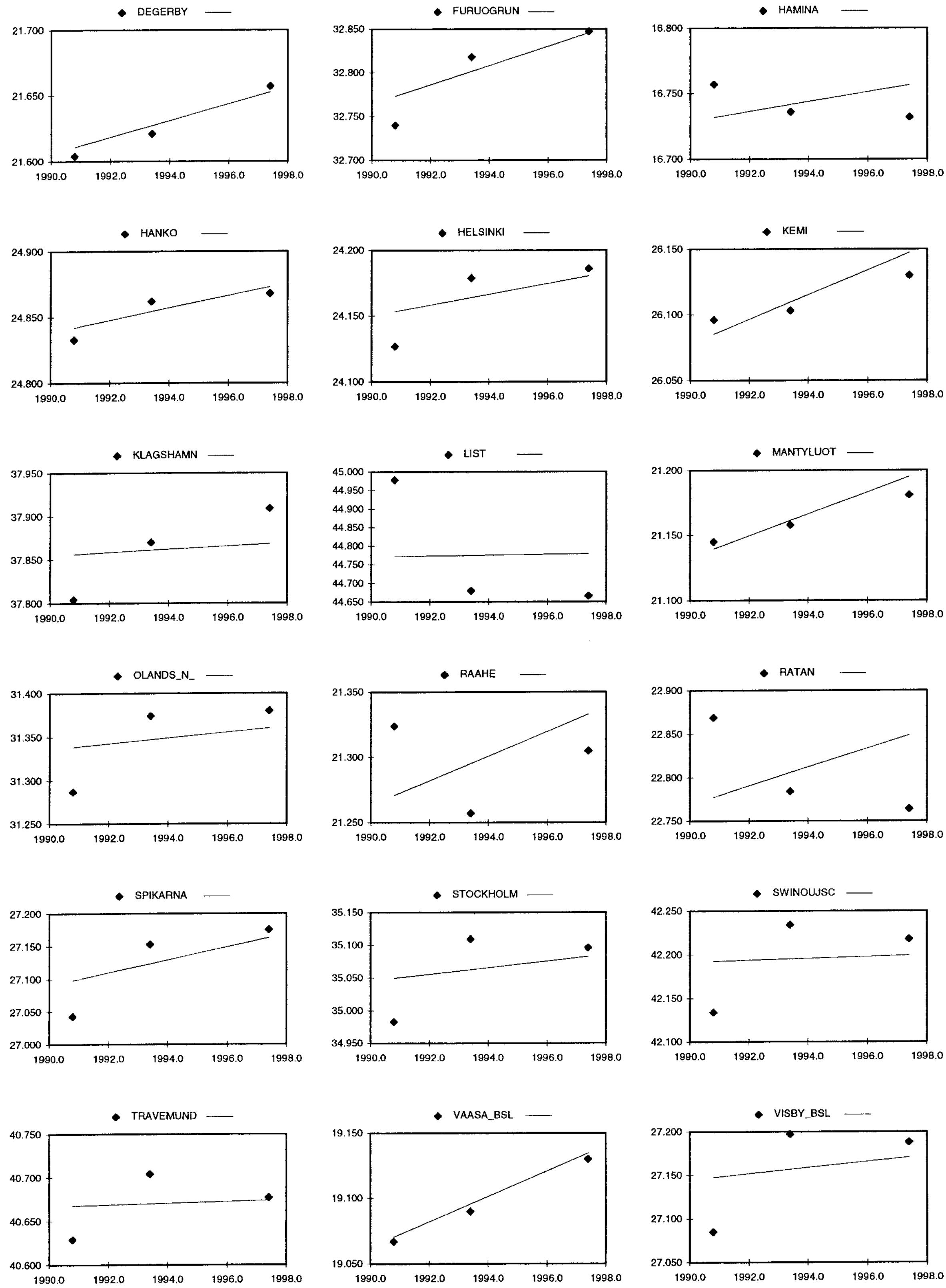

Fig. 4. *Ellipsoidal height h in 1990, 1993 and 1997 BSL campaigns. If the point has changed, the height value is reduced to the marker used in 1997. The trend line is drawn using the known uplift values from levelling. The eustatic rise of 1 mm is added to that value.*

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Nordic Geodetic Commission, 13th General Meeting
1998

Using mean sea surface topography for determination of height system differences across the Baltic Sea

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Abstract

The mean sea surface topography in the Baltic Sea and adjacent waters is reliably known in the Nordic height system NH 60. Using this knowledge we estimate differences between NH 60, based on the Amsterdam zero point, and Russian, Polish and German height systems along the southeastern coast of the Baltic Sea, based on the Kronstadt zero point. The differences agree within a few cm. We also make a simple study of the mean sea level difference between Kronstadt and Amsterdam (which is found to have been approximately 25 cm when sea level was still to be seen there).

1. Introduction

The mean sea surface topography of the Baltic Sea and its transition area to the North Sea is known with considerable accuracy in the Nordic height system NH 60. By using this knowledge it is possible to easily estimate differences between NH 60, based on the Amsterdam zero point, and height systems along the southeastern shore of the Baltic, based on the Kronstadt zero point.

Height systems can be treated in two different ways. From a "scientific" point of view, heights of both sea level and crust are defined relative to a zero point fixed in the geoid, typically a sea level at a certain epoch. From a "practical" point of view, heights are often defined relative to some point fixed in the crust, which might be moving relative to the geoid. In the first case it is possible to find a constant, expressing the height difference between the zero points of the systems. In the second case this is not possible; the height system difference will be dependent on time and location of the comparison. In the following, both views will be used.

2. The mean sea surface topography in NH 60

The height system NH 60 is fully defined and explained by Ekman & Mäkinen (1996). Therefore, its definition will only be briefly recapitulated here: 1. The zero point of the system is the Normaal Amsterdams Peil (NAP), originally mean high tide at Amsterdam in 1684. 2. The heights are based on geopotential numbers; since all sea surface heights are small, the treatment of gravity in the transformation to heights is not critical. 3. The permanent earth tide is treated in such a way that the heights refer to the mean geoid. 4. The heights on land are reduced for postglacial uplift to the epoch 1960. 5. The sea level data are reduced for secular change to the same epoch, i.e. 1960. 6. A "geostrophic connection" between Sweden and Åland/Finland across the Åland Sea is included.

The mean sea surface topography calculated in this height system is published in Ekman & Mäkinen (1996) and reproduced in Figure 1. It is based on 42 long-term mareographs connected by high precision levellings. The result can be considered reliable for three reasons. First, the NH 60 forms a consistent height system, relevant for oceanographic purposes. Second, the Åland connection has made it possible to close the long loop around the Gulf of Bothnia and, thereby, considerably strengthen the system. Third, comparisons with modern oceanographic models show an excellent agreement between the geodetic solution and oceanographic ones; the discrepancies rarely exceed 2-3 cm (Ekman & Mäkinen, 1996). Recently a refined oceanographic model of the Baltic Sea has been constructed by Carlsson (1998); the discrepancies between her model along the Swedish and part of the Finnish coast and the geodetic solution are everywhere less than 2 cm (Carlsson, 1998a).

The mean sea surface topography in NH 60 is based on sea level stations along all coasts of the Nordic countries and the adjacent part of Germany. The few additional stations in the former Eastern Europe were not included because of their somewhat unclear relations to height systems; they would in any case hardly have

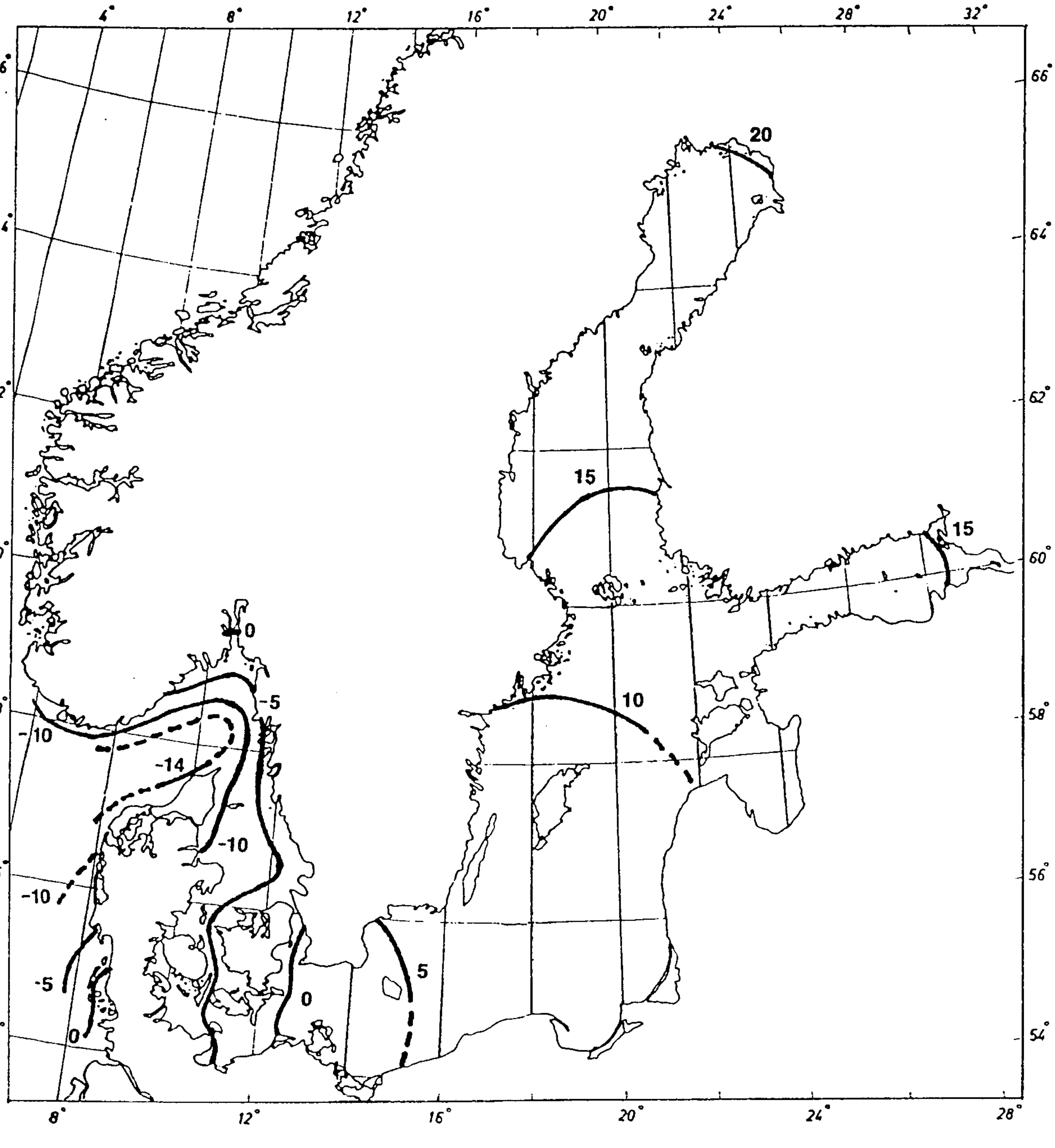

Figure 1. Mean sea surface topography (cm) in the Baltic Sea area in the height system NH 60 (Ekman & Mäkinen, 1996).

1954). Applying the mentioned value, the mean sea level difference between Kronstadt and Amsterdam becomes approximately ΔH = 16 + 9 = 25 cm; see also Table 2. However, due to the sea-walls constructed along the Dutch coast, Amsterdam is since a long time cut off from the sea. The same thing nowadays applies also to Kronstadt. Hence, strictly speaking, the numbers of the bottom line in Table 2 refer to sea levels no longer existing!

Table 2. Comparison of estimated sea levels at the fundamental European zero points of Amsterdam (if stable) and Kronstadt in the height system NH 60. Unit: cm.

Sea level	Kronstadt	Amsterdam	Difference
Zero point sea level	8	0	8
Mean sea level, original	8	-17	25
Mean sea level, 1960	16	- 9	25

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VREF1996
A new height reference surface for Norway.

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Abstract

In this article I describe the computation of Statens kartverks new height reference surface VREF1996. The difference between the geoid and the quasigeoid is shortly outlined together with a general method for adjusting gravimetric (quasi)geoids to GPS levelling measurements. I then show how the VREF1996 was adjusted before comparing this and several other geoids with geoidal heights derived from GPS and levelling. Finally I sketch what has to be done to improve geoids and height reference surfaces in the future.

Introduction.

I will here describe the computation of Statens kartverks new height reference surface VREF1996. This surface may, in combination with GPS, be used to derive heights in a height system which closely approximates the official Norwegian height datum NN1954. The surface was temporally called SKVREF96, and this name has and may still be used, but the correct denomination today is VREF1996. This surface is also an integrated part of Statens kartverks transformation program WSKTRANS.

I will shortly describe what kind of data that have been used and the methods. This is followed by a closer look at height reference surfaces showing that both geoids and quasigeoids are special instances of this more generalized class of surfaces. Then I will describe the computational procedures without going into details. After this introduction I will concentrate on how VREF1996 was computed, and which GPS and levelling data that I used. To get an estimate of the quality and accuracy of the new model, and the improvements that have taken place the last years, I compare several geoid models with different sets of GPS and levelling data. Finally I sketch what I think can be done to improve the accuracy and quality of new models in the future.

Basic model and data.

Statens kartverks new height reference surface has been computed by using the Nordic geoid model NKG96n as a starting point. The NKG96n model is the result of a Nordic cooperation where the actual geoidcomputations have been performed by Rene Forsberg at Kort- og Matrikel-styrelsen in Copenhagen.

NKG96n has been computed by the well known remove restore technique. The long wavelength part of the gravity signal is removed by the help of global geopotential models while the short wavelength part can be eliminated by using detailed digital terrain models. By applying Fast Fourier techniques on Stokes integral it is possible to estimate how much the reduced gravity anomalies contributes to the geoid, and by adding the effect of the terrain and the global geopotential models one finds the gravimetric geoid. Further details about geoid computations, Stokes integral, FFT techniques etc. may be found in the Lecture Notes from the International School for the Determination and Use of the Geoid.

Since the last NKG geoid, NKG89, was computed there has been a lot of improvements. I will only concentrate on the situation in Norway, where the gravity coverage is much better now due to the inclusion of land gravimetric data from the Geological Survey of Norway in Trondheim. The quality of the gravity data has also been improved by deleting dubious land data from the NKG gravity data base. At sea I have readjusted some older measurements, and we do now also have access to gravity data measured by several oil and prospecting companies. The terrain corrections are now made by using a 100 m x 100 m digital terrain model as opposed to a 1 km x 1 km for the NGK89 solution. Similar improvements have also taken place in the other Nordic countries. In addition the NKG gravity data base now includes data from the Baltic countries as well as parts of Russia. The Nordic cooperation has been extended to include the Baltic countries so the new geoid is no longer a Nordic only model but a joint Nordic and Baltic model. This will probably also be the case in the future for new models from the NKG.

The use of global geopotential models are of vital importance in the remove restore technique. When computing the NKG96n geoid, the EGM96 model was used, while the NKG89 solution was based upon the OSU89B model. OSU89B was computed at the Ohio State University under the supervision of professor Rapp. EGM96, the Earth gravity Model 1996, is the result of a joint cooperation between the National Imagery and Mapping Agency, NASA Goddard Space Flight Center and the Ohio State University in the USA. On a global scale EGM96 represents a significant improvement when compared to earlier geopotential models, but for our areas the changes are relatively minor. Since the late eighties the Nordic geoid has been computed by FFT-techniques. The interest in the FFT-methods lead to improved computational algorithms starting with Strang van Hees article in 1990. Further improvements was done by Haagmans et al. (1993) and Forsberg and Sideris (1993).

Geoid, quasigeoid and height reference surfaces.

Normally one does not explicitly specify which one of these surface one is talking about. In stead the word geoid is used as a common denominator for all these surfaces. This despite the fact that what most people compute today are quasigeoids, and that almost all surfaces and models which are adjusted to a local vertical datum, are height reference surfaces. I will only briefly discuss the differences between these surfaces here, and further details may be found in chapters 4 and 8 in Physical geodesy by Heiskanen and Moritz. Schematically the surfaces are given as.

- **Geoid.** An equipotential surface of the earths gravity potential W. Gravimetrical measurements are reduced from the earths surface down to the geoid. This requires knowledge about the variation of the earths density between the geoid and the surface. The geoid may in combination with GPS give orthometric heights.

- **Quasigeoid.** A geoid-like surface but no equipotential surface. Gravity observations refer to the earths surface so there is no need to make any assumptions about the earths density. This results in more complicated formulas where Stokes integral is the first term in an iterative solution. The quasigeoid may in combination with GPS give normal heights.

- **Height reference surface.** A surface adjusted to the local vertical datum like NN1954. This surface is generally neither a geoid nor a quasigeoid. It includes the land uplift. In addition all the errors in the geoid, GPS and levelling are somehow assimilated into this surface. If the levelling is perfect and there is no land uplift, this surface is either a geoid or a quasi geoid depending on the chosen height system (orthometric or normal heights). Combined with GPS this surface will give heights in the desired vertical datum.

The height reference surfaces represents a much more general class of surfaces where the geoid and the quasigeoid are special instances. Ordinary users will normally not have to care about details like these, but with the considerable increase in the accuracy of the geoid models it is my opinion that you, at least in principle, should know what kind of model you are using and what kind of heights you may get from your GPS measurements. In figure 1 the geometrical connection between the geoid and the quasigeoid is shown while it mathematically is given by

$$N - \zeta = \frac{\bar{g} - \bar{\gamma}}{\bar{\gamma}} H = H^* - H$$

H is the orthometrical height, H^* the normal height, N the geoidal height, ζ the height anomaly and $\bar{g}$ and $\bar{\gamma}$ are the average gravity and normal gravity values along the plumb line.

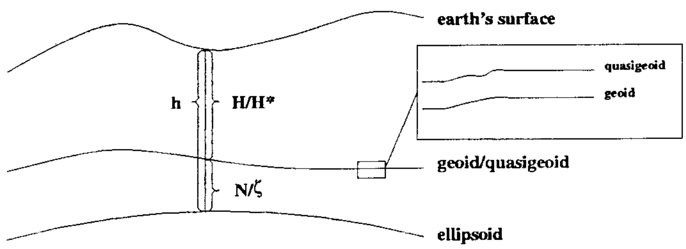

Figure 1: The relationship between the geoid, the quasigeoid, geoidal heights, height-anomaly, orthometrical height and normal height.

As a first order approximation the difference may be expressed like $N - \zeta = \frac{\Delta g H}{\bar{\gamma}}$ where Δg is the Bouguer-anomaly. Since the Bouguer-anomaly generally is negative on land this implies that the geoid normally is below the quasigeoid and that the othometric height is larger than the normal height. Since Δg varies with the height then $N - \zeta$ will be strongly dependent on the height. In Physical Geodesy by Heiskanen and Moritz (1967) there is as an example with $N - \zeta = -1.8\,\text{m}$ (Mt. Blanc, $H = 4\,807\text{m}$, and $\Delta g = \bar{g} - \bar{\gamma} = -360\,\text{mgal}$). Simple studies I have made of the Lærdal area in Norway gives a maximal difference of about 10 cm.
Neglecting effects like the land uplift the general procedure for estimating height reference surfaces may be given as:

1. Given a geoid/quasigeoid model G as a grid model
2. Given a set S with GPS levelling data and geoid-heights $N_{GPS/Lev}$

3. Find the value of G at the points constituting S, N_G.
4. Find the differences, $\Delta N = N_{GPS/Lev} - N_G$.
5. Grid the differences ΔN
6. Add the grid of ΔN to G giving N_{href}

The final result N_{href} will be a height reference surface adjusted to the dataset S based upon the geoid/quasigeoid G.

Land uplift.

I did not explicitly take into consideration the land uplift when describing the general procedure above. Due to the land uplift h, H and N are functions of the time t so the equation $h = H + N$ must be replaced by $h(t) = H(t) + N(t)$. Since $|\dot{N}(t)| \ll |\dot{H}(t)|$ and $|\dot{N}(t)| \ll |\dot{h}(t)|$ then $h(t)$ as a first order approximation is given by $h(t) = H(t) + N(t_0)$ where t_0 is a given epoch like 1954. By this approximation I am assuming that the geoid is constant within the considered time span i.e. $N(t) = N(t_0) = N_0$. Assuming that the land uplift varies linearly with time then $h(t)$ and $H(t)$ are given by $h(t) = h(t_0) + k(t - t_0)$ and $H(t) = H(t_0) + k(t - t_0)$ where k is a land uplift coefficient. This k will again be a function of the geographical position i.e. $k = k(\phi, \lambda)$.

The GPS/levelling derived geoidal heights N_0 are given by $N_0 = h(t) - H(t_0)$ where h, the ellipsoidal height, comes from GPS and H_0 is the levelled height. So to find N_0 from these data you need to know the land uplift. The gravimetric geoid however gives N_0 directly. The land uplift has so far traditionally been estimated by relevelling and tide gauge measurements along the coast. I will return later to alternative methods for estimating the land uplift. Jan Danielsen in the Geodesy division of Statens kartverk has computed a model for Norway. This model has been used when doing a land uplift correction to several of the GPS/levelling datasets that I have looked at in this article.

When you are using a height reference surface and GPS to find $H(t_0)$, then you are using the following formula $H(t_0) = h(t) - N_{ref}$ where N_{ref} is the height of the height reference surface above the ellipsoid. N_{ref} is also given by $N_{ref} = N_0 + k(t - t_0)$, so the height reference surface includes the correction for the land uplift. There are generally two ways to handle the land uplift when estimating the height reference surfaces. If you have access to a model of the land uplift you can find H_0 from GPS and levelling. If you look at the general computation procedure given above and use this N in step 4, keep step 5 unchanged, and in step 6 add the value of the land uplift for each grid node, you get the desired surface. Alternatively you may disregard the land uplift completely, and this is in fact what I did when computing VREF1996. In this case the height reference surface will implicitly include a model for the land uplift.

VREF1996.

When computing VREF1996 I did not use a model for the land uplift. As the geoidmodel G I used the Nordic geoid model NKG96n. This model is again derived from the Nordic quasigeoid NKG96. The dataset S consists of GPS and levelling measurements made by Statens kartverk, and the location of these points are shown in figure 2. The geographical distribution of these points are unfortunately a bit uneven. The gridding, step 5, was done by using Forsbergs program geogrid, and a contour plot of this grid can be found in figure 3. By then adding this grid to NKG96n, step 6, I found the VREF1996 height reference surface shown in figure 4.

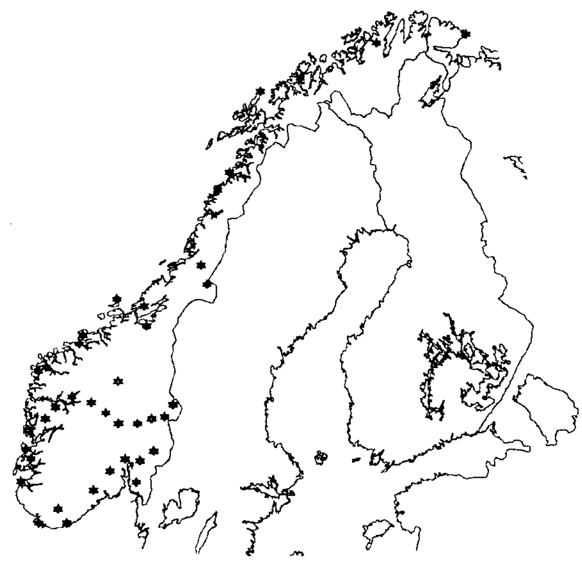

Figure 2: GPS and levelling points used when computing VREF1996.

This model, VREF1996, is only valid for points on land in Norway. Using this model for points outside Norway will probably result in poorer results than what one will get from the Nordic NKG96 models. To get similar models as VREF1996 for these areas one must contact the geodetic institutions in these countries. VREF1996 must not be used in connection with satellite altimetry for oceanographical studies either. In this case use one of the NKG96 models in stead, since at sea the geoid and the quasigeoid are identical.

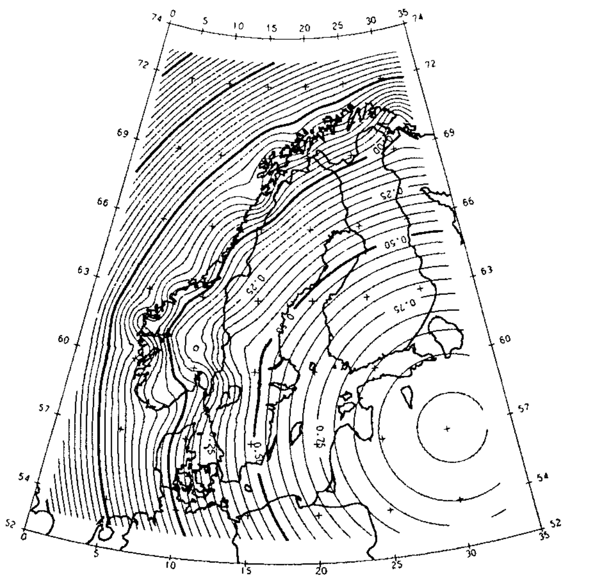

Figure 3: The difference between NKG96n and Statens kartverk's GPS/levelling dataset. Contourinterval 0.05m.

Figure 5: GPS/levelling points in the Torge-profile.

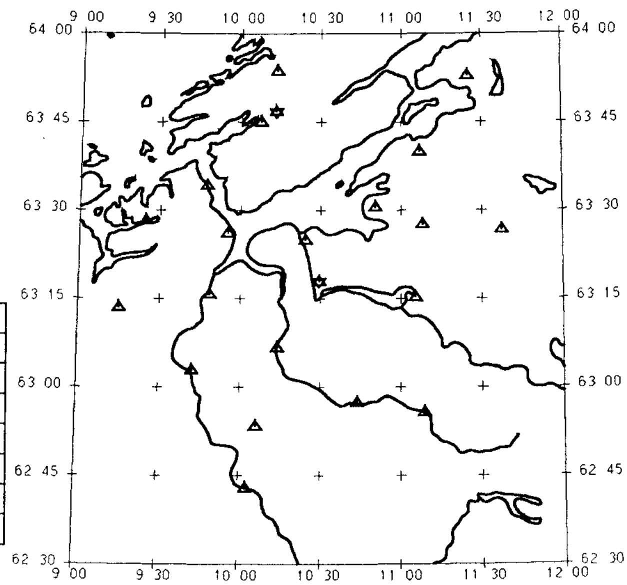

Figure 6: GPS/levelling-points in Trøndelag.

Quality control.

In order to get an idea about the accuracy and the improvements that have taken place the last years I will compare several geoidal models with different GPS and levelling data sets. The models are:

Model	
COL86	Collocation, Tscherning & Forsberg, 1986
FFT88	FFT, Forsberg & Solheim, 1988
NKG89	FFT, Forsberg, 1989
NKG96	Spherical FFT, Forsberg, 1996
NKG96N	Spherical FFT, Forsberg, 1996
NOR96n	Adjustment to GPS/levelling, Solheim, 1996
VREF1996	Adjustment to GPS/levelling, Solheim, 1997

These models are to be compared with the following GPS and levelling sets, see also figures 5-8.

Dataset	Adjusted for the land uplift
The Torge-profile, N-S traverse	Yes
Trøndelag, Geir Simensen	Yes
The SWET-profile	Yes/No
Lier, Heltne & Haavik	Yes/No

For both the Torge and the SWET profiles I am only looking at the parts of these profiles that are within Norway. The Torge profile continues southwards through Sweden, Denmark and Germany etc. while the SWET-profile, Scandinavian West East Traverse, goes all the way trough Sweden and Finland to the Russian border.

When doing comparisons, as I do here, it is important to know how you are dealing with the land uplift. When you are looking at height reference surfaces your GPS and levelling data sets should not be corrected for the land uplift while this must be done when comparing with (quasi)geoids. Except for the models VREF1996 and NOR96n, which was a very early and preliminary Norwegian height reference surface based upon only 19 GPS and levelling points mainly along the coast, all of the models are (quasi)geoids. The results of the comparisons I have done can be found in table 1 where STD is the standard deviation.

ESA and NASA have similar plans. These satellites will, if the fly, lead to an important improvement in our knowledge of the earths gravity field. This will give more accurate geoidal models, and it may eliminate some of the long wavelength errors that can be found in several geoidal models.
To what degree the geoidal models, at some time in the future, will be accurate enough to replace levelling for some purposes, is not settled. The opinions varies and I will not make any other statement than to say that we are facing a challenging task if we are to achieve this in a mountainous country like Norway. Obviously more and better gravity data will be important. Access to bathymetrical data and knowledge about the earths density will lead to more accurate terrain corrections and geoids. New theoretical improvements like the work being done by professor Grafarends group in Stuttgart, trying to use geopotenitals when computing geoids, Feistrizter (1998), are also interesting although there are mixed opinions about this method. Finally I would also like to mention that due to the tidal attraction from the moon and the sun there are not one but several different kind of geoids depending on how you treat the permanent part of the tidal signal. Further information about this may be found in Ekman (1988),

Conclusions.

In this article I have described the differences between geoids, quasigeoids and height reference surfaces. I have described how you can use GPS and levelling data to adjust your (quasi)geoid to get a height reference surface. I have briefly shown how I did this when I computed VREF1996. The accuracy of this and other models have been investigated by comparing them with several GPS and levelling data sets. These comparisons show that heights derived from GPS and VREF1996 will have a standard deviation of about 5 cm. Finally I gave a short description of what can be done to get better models in the future.

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Implementation of a new height datum in Denmark

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Abstract
In 1997 the new height datum DNN KMS1990 was published in Denmark. The major problems still to be solved, is to get this new height datum accepted and used in normal practice in Denmark. The paper covers the problems concerning measuring the detailed height points in Denmark (80.000 points) and the political and practical problems in a introduction of new height systems.

A. Background
In 1982 the field work of the 3. precise levelling started with the observations along the Danish/German border. Because of uncertainty of the magnetic influence on the automatic levelling instruments the north - south part of the levelling was carried out as traditional spirit levelling instruments, and the rest of the line was observed as motorized geometric levelling with an instrumentation from Zeiss Ni002.
The magnetic influence on the instruments was tested in the following years, showing, that the influence on the Ni002 was in the order of 0.01- 0.03 mm pr. km., and that the size of the influence was stable on every instruments, results that showed that the instrumentation was all right for precise levelling purposes.

Several test followed in the years from 1982 to 1985, to secure that the levelling technique was all right, and that the 1. Order benchmarks was in a condition that they afterwards could be trustworthy, in solving the problems about how Denmarks crust behaved.

In Denmark 95% of the 1. order bench mark are establish as stone pillars founded 1.5 to 2 m below the surface. All the bench marks from the 1. precise levelling were established along the main roads. These roads are in the years from 1890 till today getting wider and wider, so many of these benchmarks today is located directly under the road. Before these points could be used, we therefore have to dig down to the benchmark and place a "manhole" around the point. Totally about 450 of these points are now placed under manholes. The test from 1982 to 1985 showed that we could trust the behaviour and the height of the main part of these points.

The main part of the levelling was carried out in the years from 1986 to 1992, 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 1993 and 1994.
The connection between Denmark and Germany from Rødby to Puttgarten, and from Sealand to Funen was carried out as Hydrostatic levelling.

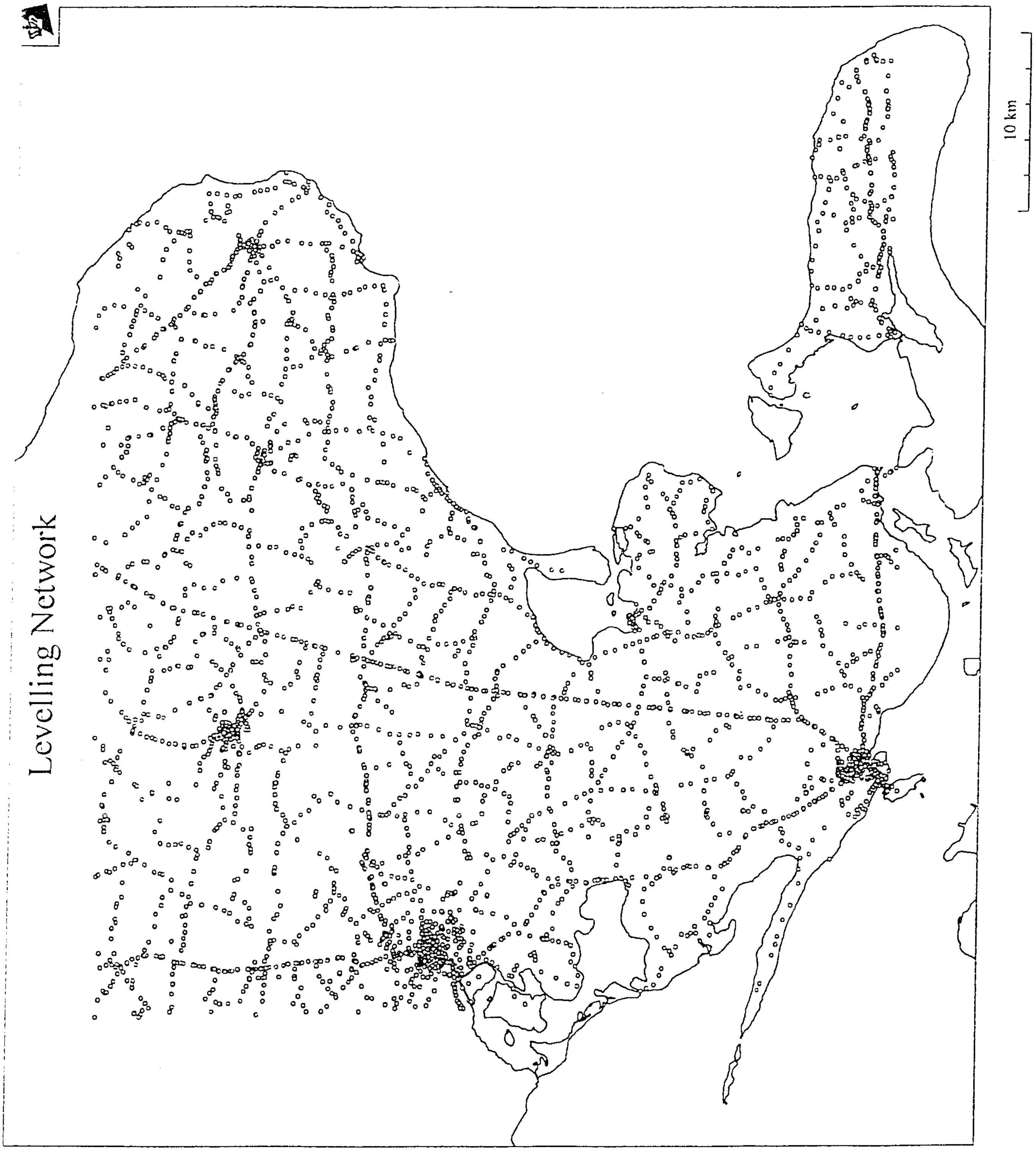
Twice a year all levelling rods were run trough a KMS's own compensator, and afterwards every single reading on the levelling rod were adjusted to its real value.

The adjustment were carried out as geopeotential 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:

5,572 m

4.3 cm lower than the old height system DNN GM from 1891.

Fig. 1 Map showing all levelling points, including precise levelling points, main levelling points and other points in he southern part of Sealand.



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, and about 7.700 points have now a new 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 periode. The main lines up till 1940, the densification in a very busy periode 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.

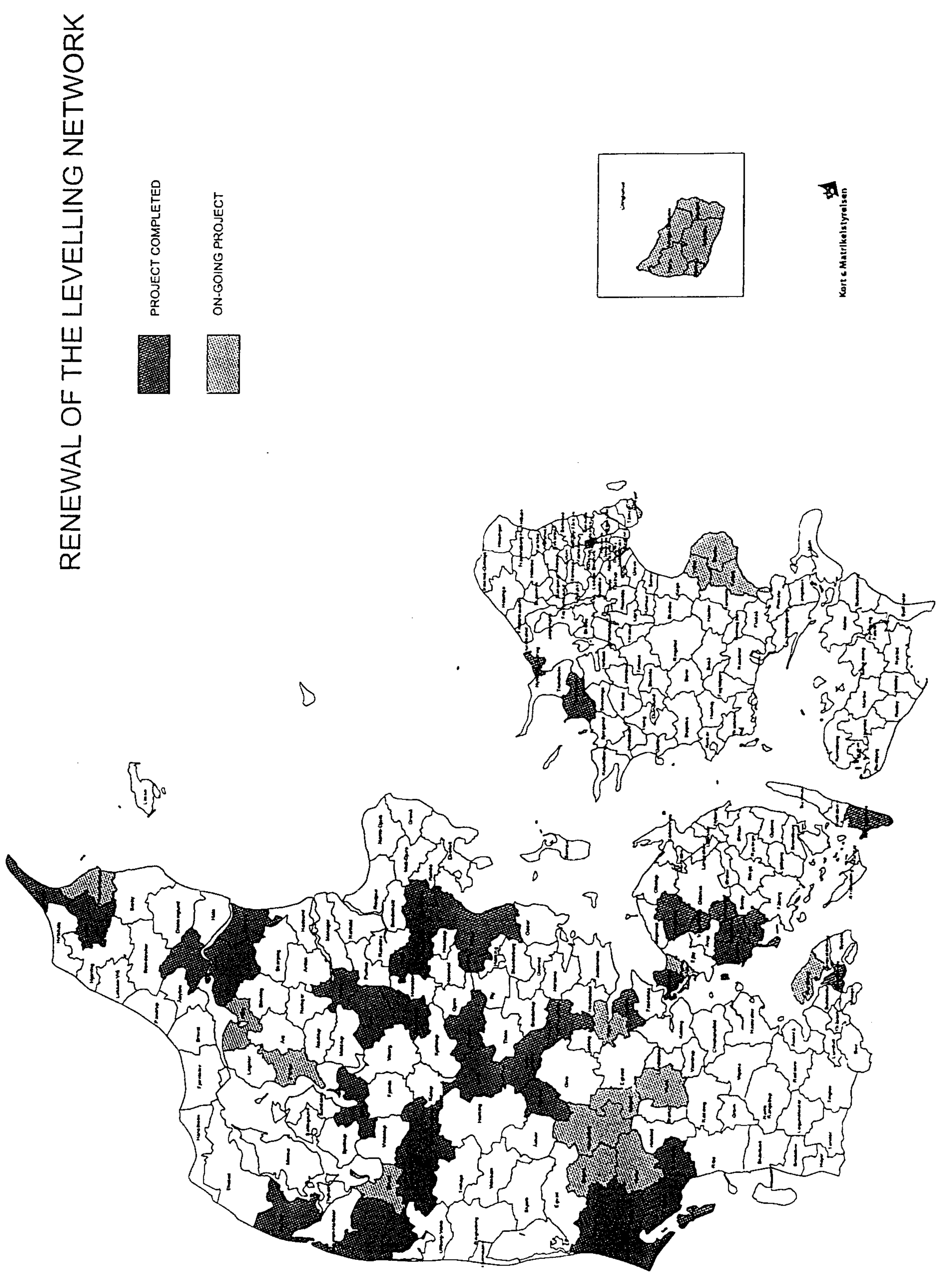
There is a strong need all over Denmark for good height information. Main part of the sewing system in the towns areas need a renovation, and many benchmarks are being destroyed, or worse, the height of the benchmark are not to be trusted because of local settings etc. Too, there is a need for bencmarks 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 og 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.

Fig. 2 Map showing where contracts with municipalities and counties of renewal of the levelling system is signed.



In this year we have started production with automatic registration equipment in the geometrical levelling. Until now one municipality is covered all over with levelling of this kind. No results are yet avaible, but the production seems promising.

In total the equipment, together with the equipment from the private surveyors, is in place to handling the height problem in a 10 years periode

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 contry 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 behavior of the contry, how the crust is moving, and how local settings and loss of benchmarks has reduced the quality of the existing height system.

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 60of Denmarks 275 municipalities has signed a contract with KMS about renoval 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

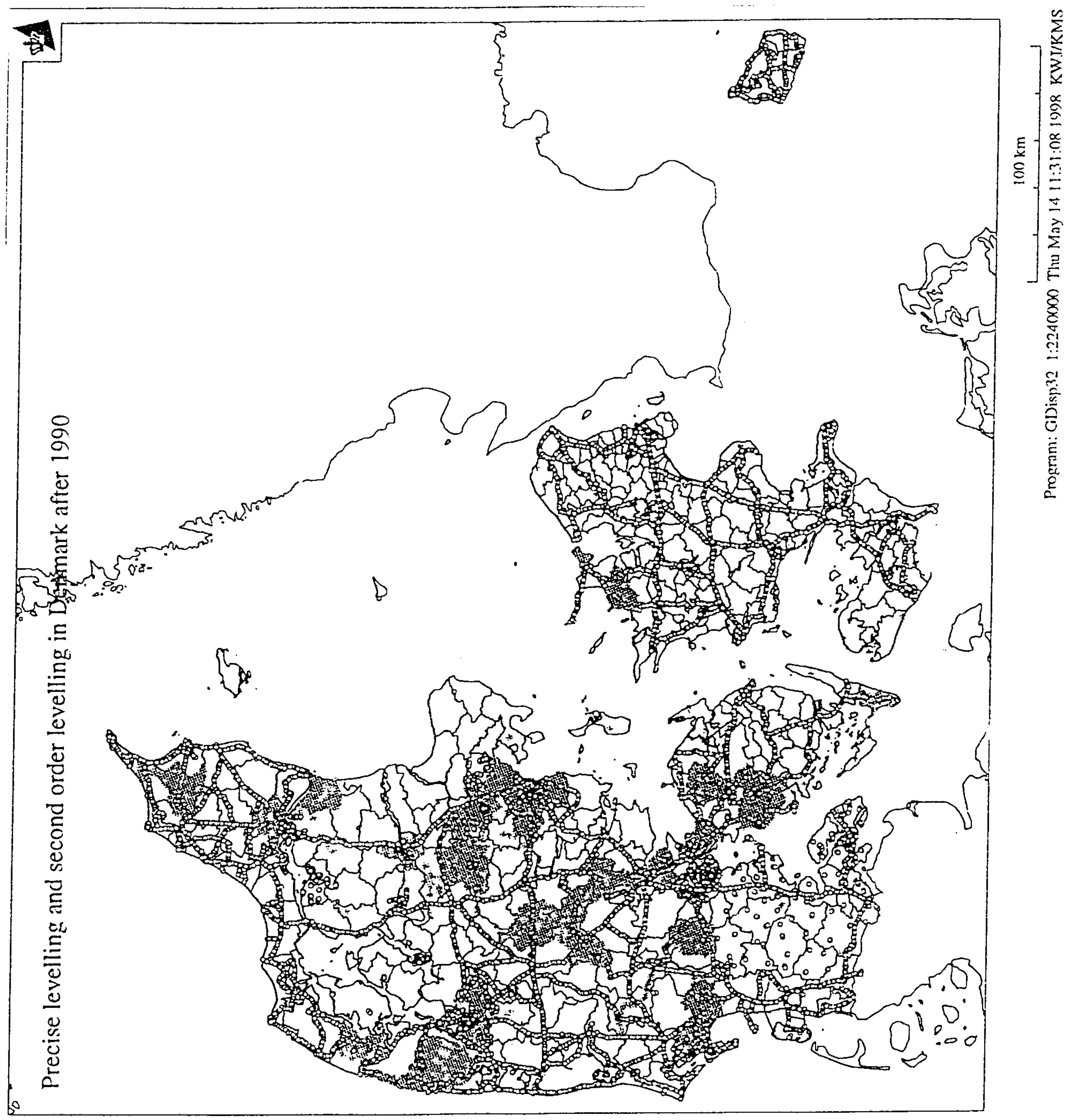
Two major problems in changing is still to be solved. How and when ?

In Denmark all the importent questions about the Reference Systems are discussed in the Council for Denmarks Geografical Reference Network. In the "Council" all the major user are members. KMS, Municipalities, Road Directorie, Counties are the most importent 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 inhomogenios old height system.
The working group suggested that the database changes should be done in two steps. First a transformation from the old inhomogenios system to a new renovated, but old system. The transformation was given by the differences in height to all bencmarks 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.

* In Denmark in addition to the new DNN. KMS there are in fact three different official height systems. DNN. GM91 covering Jutland, and DAN. GI44 covering Sealand and Funen. The third system still in use is the oldest system KN44, covering Copenhagen and some of the municipalities around.

fig.3 Map showing where new levelling is performed, and data are in the database. The dotted points shows the precise levelling line and the geometic levelled lines, the rest the trigonometric levelling.



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 geoide 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 is to be send to all major user in Denmark including the municipalities. The main question is how and when the change is to come.

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, mappeople. 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 22.000 benchmarks including the precise levelling line is relevelled since 1992. This year appox. 8.000 benchmarks are relevelled or levelled for the first time.

In KMS we are still optimistic.

Towards the unification of vertical datums in the Baltic Sea region

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Abstract

Results of several GPS campaigns, including the three Baltic Sea Level GPS campaigns, and several national campaigns are combined with a gravimetric geoid and precise levellings for constructing a unified vertical datum for the Baltic Sea region. Results from the permanent GPS networks of the area are also used.

Additionally, the Sea Surface Topography (SST) has been computed using the gravimetric geoid and GPS observations at the tide gauges. We discuss methods to connect different vertical datums and on the basic principles if one wants to establish a new unified vertical datum. An example is given how national datums could be transformed to a new one using a gravimetric geoid and GPS data.

Introduction

During last years there have been several large GPS campaigns in the Baltic Sea area. These include three Baltic Sea Level (BSL) GPS campaigns, SWET, DOSE, and BIFROST. Additionally, there have been several national camapaigns. A dense network of permanent GPS stations is also available.

The First Baltic Sea Level GPS Campaign was performed in October 1990, the Second Campaign in June 1993 and the Third Campaign simultaneously with the EUVN in May, 1997. In the BSL I a total of 30 tide gauges and nine fiducial stations were observed, whereas in the BSL II 35 tide gauges and 12 fiducial stations were observed, the total number of GPS receivers being more than 50. Additionally, several permanent GPS stations are included in the network of the third campaign.

The benchmarks observed were connected to the national precise levelling network and to the readings of the tide gauges. Long series of tide gauge observations give us a connection to the mean sea surface. The mean sea surface height is affected e.g. by the land uplift of the area

As a result of the campaigns, the height of the non-tidal crust above the GRS-80 ellipsoid was computed at the tide gauges. Results of the computations of BSL I and II are published in the *Reports of the Finnish Geodetic Institute* (94:2, 95:2) giving individual reports and summary of all computing groups. The results of the third campaign will be published in Poutanen (1998a, 1998b) and in the proceedings of the Baltic Sea Level meeting, to be held in St.Petersburg in June, 1998.

The quality of data of the BSL III was considerably better than that of the previous campaigns due to advanced receivers, more calm ionosphere and better separation of different GPS receiver types. A good tie to ITRF96 sites resulted that the repeatability in height is only 2.6 mm (Poutanen -98b). The European wide EUVN campaign has its goal to unify the vertical datums of the whole Europe (e.g. Franke *et al.*, 1998).

The SWET-92 (Scandinavian West-East Traverse) was a combined effort of the Finnish Geodetic Institute, Norwegian Mapping Authority, Swedish National Land Survey and Danish Survey and Cadastre to measure a GPS traverse from the west coast of Norway to the east border of Finland (Poutanen *et al.*, 1994). A total of 34 stations close to the latitude 60° was selected so that the mean distance between the stations became about 50 km. All points were either precise levelling bench marks or connected to the levelling network. The results of the campaign have been used e.g. in fixing the position and level of the new gravimetric geoids NKG96 and FIN95.

Additionally, results from several national GPS campaigns have been used in this report, as well as the coordinates of the Swedish and Finnish permanent GPS networks. Naturally, all the points included here have levelled (orthometric or normal) heights in the national height system.

National height systems

The time-independent part of the luni-solar tide should be treated in a consistent way in levelling and in geoid and GPS computations if orthometric (or normal) heights are to be obtained from the GPS observations. We may distinguish three cases: *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 effect of the permanent tide in GPS observations, see Poutanen *et al.* 1996.

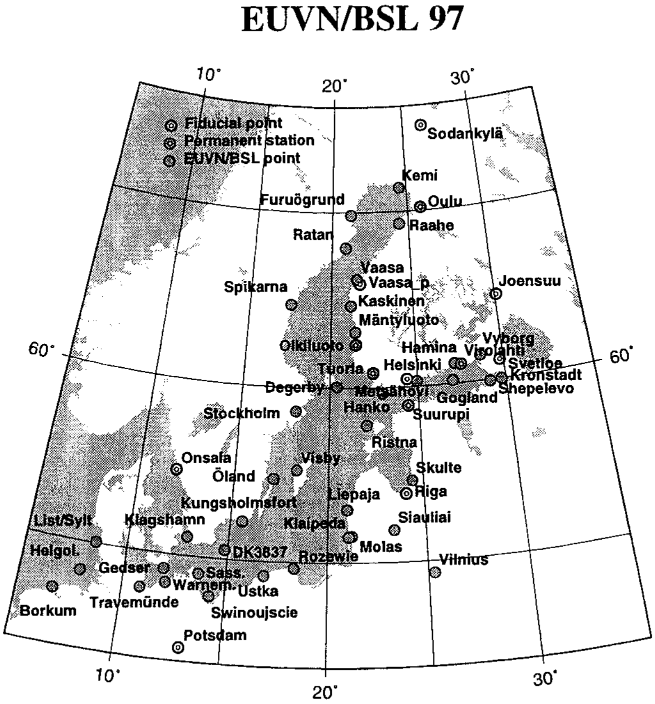

Figure 1. *Tide gauge and fiducial sites of the Third Baltic Sea Level GPS campaign.*

The GPS programs give the 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.

However, when the mean sea surface is referenced, one should use *mean geoid* and *mean crust* instead of non-tidal geoid/crust. The differences between various geoids/crusts amount to up to 10 cm in the area of the Baltic Sea. We can easily convert all quantities to their mean values using the formulae of (Ekman 1989):

$$\Delta H_m - \Delta H_n = 0.296\,\gamma(\sin^2\varphi_N - \sin^2\varphi_S) \quad [\text{m}] \qquad (1a)$$
$$N_m - N_n = (1 + k)(0.099 - 0.296\sin^2\varphi) \quad [\text{m}] \qquad (1b)$$
$$\Delta h_m - \Delta h_n = -0.296\,h(\sin^2\varphi_N - \sin^2\varphi_S) \quad [\text{m}]. \qquad (1c)$$

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. 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 OSU-89B spherical harmonic model $k = 0.3$, and $h = 0.609$ in the Bernese GPS software; these three are incompatible among themselves.

Above, φ_N and φ_S refers to the latitude of northernmost and southernmost station, respectively. We used $\varphi_S = 54.8°$ for the latitude of the southern station which is a junction point latitude between the Danish and German levelling.

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 system to the epoch of the observations (1993.4). For this, the uplift values given in the map of Kakkuri 1991 were used.

Country Height system	geoid	geoid type	epoch
Finland N60, orthometric	classical	mean	1960
Sweden RH70, normal	quasi	non-tidal	1970
Norway NN1954, orthometric	classical	mean	none
Denmark DNN GI, orthometric	classical	non-tidal	1950
W.Europe UELN73, orthometric	classical	mixture	(1960)
E.Europe Kronstadt, normal	quasi	?	?

Table I. *European height systems (Ekman 1995).*

To obtain orthometric or normal height from GPS observations one needs a geoid. There were two suitable geoids available: The Nordic Standard Geoid NKG96 (Forsberg, private communication 1997) and the BSL95A geoid (Vermeer 1995). The NKG96 is a non-tidal geoid because it is based on the spherical harmonic model where the permanent tide is removed (R. Forsberg, private communication 1997). The NKG96 is a cm-geoid without any major distortions due to a considerably better gravity coverage of the surrounding area than its predecessor NKG89. Its level was adjusted using results of several GPS campaigns.

Another gravimetric geoid, BSL95A, was computed by Martin Vermeer (Vermeer 1995). It also contains gravimetric data from the Baltic states and Russia and its accuracy is about the same as that of the NKG96. This geoid, however, is strictly not a mean or a non-tidal geoid because the original non-tidal geoid is fitted to the tide gauge heights which are measured from another mean geoid.

All these geoids are quasi-geoids and heights referring to these are normal heights. At the 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, approximately by

$$H_{norm} - H_{ort} = \frac{C}{\bar{\gamma}} - \frac{C}{\bar{g}} \approx \frac{\Delta g_B}{98\,200[\text{mgal}]} H_{ort} \tag{2}$$

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 (Heiskanen and Moritz, 1967).

Satellite altimetry

We cannot use the mean sea surface directly for vertical datums because of its semi-permanent displacement, the Sea Surface Topography (SST). Next, we will shortly discuss on this. One method to obtain the SST is to use altimetry data. We subtracted the NKG96 to obtain the SST of the Baltic Sea. Comparison with the SST from the GPS shows any bias and tilt w.r.t. it. After verifying the correctness of the geoid, we can use it to establish a common vertical datum for the area.

We computed an altimetric geoid for the Baltic Sea using 18 ERS-1 35-day Exact Repeat Mission arcs (Vermeer *et al.* 1995). In this, the traditional cross-over method was applied. In addition, we tested with new precise orbits two other methods to obtain the SST.

The first one just simply applies quadratic or higher order polynomial surface fitted to the observed sea surface heights above the reference ellipsoid. When the orbits are good and we use a long time span, individual outliers and biases cancel out. As a result, we obtain a smooth surface. like the one shown in Fig. 3. This can be compared to the surface obtained independently from GPS, shown in Fig. 4 which is of a similar resolution.

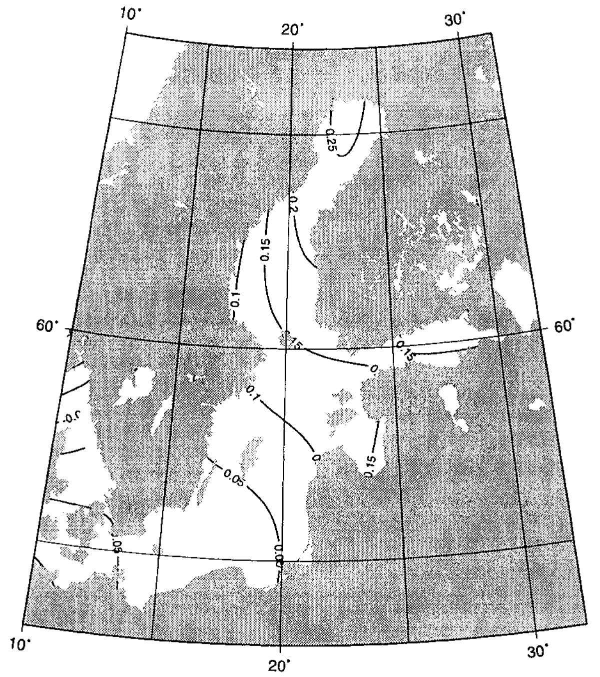

Figure 3 *Sea surface topography of the Baltic Sea from ERS1 satellite altimetry. A polynomial surface is fitted to the actual data to obtain the resolution similar to the Fig. 4 of the SST from the GPS measurements.*

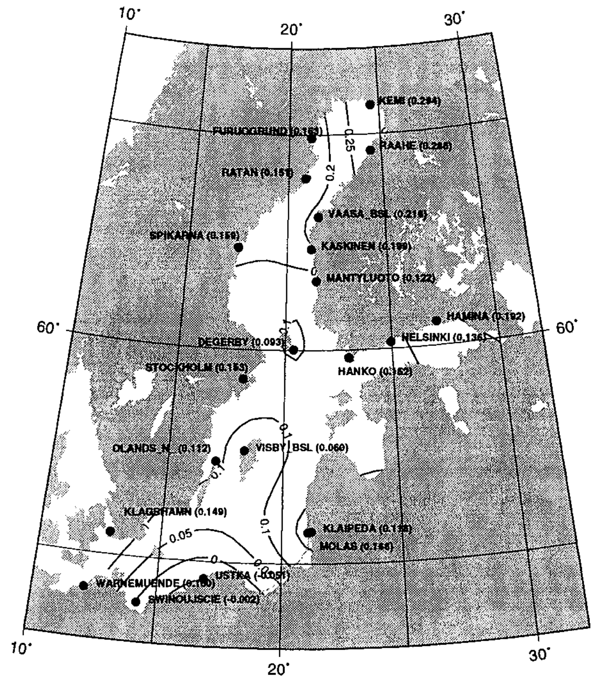

Figure 4. *Sea Surface topography of the Baltic Sea from the BSL GPS observations and mean sea level (Poutanen 1998a).*

Another method, still under test and development, applies the gravity gradients in geoid computation. Creating the dimensionless gradient values from the geoidal heights n_i observed with the altimetry, we get

$$g_i = (n_i - n_{i-1}) / d_{i,i-1} \tag{3}$$

where d is the distance between two successive points, i–1 and i. We fit on (3) a polynomial surface

$$G_{N,S} = \sum_{i=0}^{n} \sum_{j=0}^{m} b_{ij} \varphi^i \lambda^j \tag{4}$$

separately for ascending and decending arcs. Parameters φ and λ are computed for the middle of the two successive sub-satellite points. The surface is now expressed in orbit-bound variables. To convert the fit to the terms of deflection of the vertical, we combine the two separate fits as

$$\begin{aligned} \xi_{Alt} &= \cos A \sum_{i=0}^{n} \sum_{j=0}^{m} \frac{b_{ij,N} - b_{ij,S}}{2} \varphi^i \lambda^j \equiv \cos A \sum_{i=0}^{n} \sum_{j=0}^{m} c_{ij} \varphi^i \lambda^j \\ \eta_{Alt} &= \sin A \sum_{i=0}^{n} \sum_{j=0}^{m} -\frac{b_{ij,N} + b_{ij,S}}{2} \varphi^i \lambda^j \equiv \sin A \sum_{i=0}^{n} \sum_{j=0}^{m} d_{ij} \varphi^i \lambda^j \end{aligned} \tag{5}$$

where the subscript *alt* just refers to the method we use for obtaining the coefficients. A is the azimuth of the arc at the point computed.

In terms of geoidal height N, the components of the deflection of the vertical can be expressed as slopes of the geoid with respect to the reference ellipsoid (Vanicek and Krakiwsky 1995, p. 496):

$$\xi = -\frac{1}{R}\frac{\partial N}{\partial \lambda} \qquad \eta = -\frac{1}{R\cos\varphi}\frac{\partial N}{\partial \varphi}, \tag{6}$$

where R is the mean radius of the Earth.

If we approximate also the geoid of a relatively small and smooth area (like the Baltic Sea) with a polynomial surface

$$N = \sum_{i=0}^{n} \sum_{j=0}^{m} a_{ij} \lambda^i \varphi^j \tag{7}$$

then the components of the deflection of the vertical can be expressed as

$$\begin{aligned} \xi &= -\frac{1}{R}\frac{\partial N}{\partial \lambda} = -\frac{1}{R}\sum_{i=0}^{n}\sum_{j=0}^{m} i\, a_{ij} \lambda^{i-1} \varphi^j \\ \eta &= -\frac{1}{R\cos\varphi}\frac{\partial N}{\partial \varphi} = -\frac{1}{R\cos\varphi}\sum_{i=0}^{n}\sum_{j=0}^{m} j\, a_{ij} \lambda^{i} \varphi^{j-1} \end{aligned} \tag{8}$$

By inserting the coefficients from (5) into (8) we could compute the geoid N in (7).

As we know from other studies, the semi-permanent sea surface topography quite closely follows the change of the salinity of the Baltic Sea (Ekman and Mäkinen 1996). At the bottom of the Gulf of Bothnia where the salinity already is close to zero, the mean surface is about 20 cm higher than at the southern part. The change in salinity explains most of the height difference.

SST and comparison of the heights at tide gauges

In Poutanen 1998a, one obtains the orthometric height of the GPS benchmarks in national height systems, and the height difference between the benchmark and the mean sea level at the epoch of the observations. Due to the land uplift and the eustatic rise of the sea surface, this difference changes with time. However, the yearly variation is so large that the mean sea surface cannot be taken from the yearly mean but it must be computed with a least squares fit of tide gauge readings of several decades.

In Fig. 4 we have plotted the height difference of the mean sea level at tide gauges w.r.t. a common height system (Poutanen 1998a); for a previous determination, based on BSL II see (Kakkuri and Poutanen 1997). First, the geoidal height N of NKG96 is subtracted from the height above the ellipsoid, h, to obtain the "GPS orthometric" height H_{GPS}. This allows us to fix all tide gauges to the same reference frame. From tide gauge recordings of several decades, one can obtain the mean sea level in local vertical datum. However, now we have also the height difference between the GPS marker and the mean sea level, and thus we can express the sea height in the common reference frame, i.e.

$$SST = H_{GPS} - H_{msl}. \tag{9}$$

The altimetric geoid + SST is based on a relatively short period of time (less than two years) so that the mean sea surface can be off from its long-term average. If Figs. 3 and 4 are compared, one can see that we still are far from satisfactory. The level of altimetric SST is adjusted to the tide gauges just by fitting the heights at Helsinki TG to the same value. The amount of the overall SST change is about the same but the shape is different. One reason for this is that in Fig. 4, the shape is based on a relatively small number of points where the erroneous value of a single point can cause a big change in shape. Obviously, more work is needed on this topic.

Defining a unified height system – some theoretical and semi-practical perspectives

It is somewhat an oversimplification to compare height systems of two countries in the way it has been generally done. We have to more or less arbitrarily choose what values are compared. As an example, when speaking about Swedish and Finnish heights, we first convert Finnish orthometric heigths 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 good reason, if there is any sense to do 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 1996; 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.

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 the choices above.

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 it could also be a natural choice and may also give a good connection to the proposed World Vertical Datum (Rapp 1995). Of course, one can choose any potential value W^0 but this choise is also more or less arbitrary one. NAP has been used for several vertical datums.
2. In their paper, E&M used geopotential numbers. Their aim was mainly to study the SST and with the small heights there is no need to distinguish between normal heights and orthometric heights. While geopotential numbers are required for the datum realisation, the question of their conversion to metric heights 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 gravimetric geoid 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 computes global or regional geoids. In this paper we look the topic in the viewpoint of current common practice and data availability.
3. There is an extensive discussion on GPS and tide in Poutanen *et al.* 1996. Although there are some theoretical objections, we propose that the mean geoid should be used, together with the mean crust. 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 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. The height above the ellipsoid, h, is changed by the absolute uplift. However, with normal heights one has to use the apparent 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 Kakkuri and Poutanen (1996) or Ekman 1993. The size of the rise is 5 – 10% of the uplift value. Additionally, we propose the epoch 2000 for the new standard epoch because 1960 is already quite far in the past. This coincides also with the third precise levelling of Finland which will end in year 2001 (J. Kääriäinen, private communication, 1998). There has been preliminary discussions on refining the Finnish height system after that.

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THE GEODETIC REFERENCE NETWORK IN LATVIA – STATUS REPORT

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During the period of 1996 - 1998 the geodetic activities of the State Land Service of Latvia concerning reference network have been as follows:

1 Geoid

To follow up the growing need for accurate and common vertical reference network in 1996 at the National Survey and Cadastre of Denmark (KMS) we have computed Latvian Gravimetric Geoid (LGG96) what is a part of Nordic Geoid (NKG96). We have tested our geoid on 32 Zero Order GPS points and received standard deviation for LGG96 8 cm after fitting of the geoid. Gravimetric geoid contains information for approximately 12 000 gravity sites on our territory. Unfortunately, 95% of the mentioned sites are digitized from the gravity anomaly maps in scale 1 : 200 000, what is insufficient to reach geoid accuracy up to 1 - 2 cm. Big gravity data gap on the Baltic Sea disturbs our geoid, too, especially, at the Western part of Latvia.

Levelling data used for geoid test at the reference GPS sites are rather old, because the last levelling of those sites were carried out in 1950-ies. It is difficult to put all available geodetic data together into consistent system.

We can use derived geoid for the test of our Vertical Geodetic Network and to see how good is national GPS Zero Order, First Order and Second Order network in connection with our geometrical levelling network. We can ask how consistent is our Vertical Geodetic Network or are we satisfied with our LGG96?

To compute "better" local geoid we need gravity data on the Baltic Sea and we must be sure that digitized gravity data (~ 12 000 sites) correspond to the system of International Gravity Standartization Net 1971 and elevation model (DTM) used is reliable, too.

For the fitting of the geoid we must use GPS Zero Order, First Order and Second Order sites together with First and Second Order levelling data, which makes the base for a 4-parameter empirical datum fit to GPS/levelling stations. Received reliable geoid could be used for modern and efficient survey techniques and will not spoil up our national height system.

In 1996 the Sector Programme between Denmark and Latvia titled "Vertical Network Analysis and Modernisation in Latvia" was started. The Programme includes GPS, levelling, gravimetry and local geoid solution. We digitise first and second order levelling network catalogues and to close levelling loops inside country the future levelling plans are made with the consideration of our capacity.

2 Gravity

In 1998 the reference gravity network is started to create (contains 24 stations). It is necessary to complete gravity reference network and to continue detailed gravity measurements to verify and improve Latvia gravity database. Points are selected along the roads with the distances between points less than 5 km. Trigonometric points or levelling benchmarks are used in the detailed survey. Gravity surveys are based on three absolute gravity points measured by J. Mäkinen (October 1995).

3 Satellite positioning (GPS)

Reference system for our national GPS network in 1992 was established in accordance with the EUREF.BAL`92 campaign in 1992 4 points - RIGA (0201), ARAJS (0410), INDRA (0407) and KANGARI (0406) were included. GPS reference network was developed in 1993 and contains 44 stations in total. Later development of First and Second Order GPS Network was started. We should complete national GPS network in 1999. Mainly all GPS measurements in the country have been carried out by our 4 Trimble 4000 SSE type receivers of the State Land Service of Latvia.

Real need of EUREF89 national GPS network densification has been considered as continuation from EUREF.BAL'92 campaign. Results of the EUVN97 campaign must be evaluated for future use. We have not improved results of original EUREF campaign but we can assume that the results of our EUREF89 solution in Latvia could be improved by outcome of EUVN97 campaign because we have observed 5 EUVN97 stations in the country.

In our country we have two operating Permanent GPS-stations: RIGA and IRBENE. RIGA is already EUREF station, but in May 1998 the second Permanent GPS-station in Latvia: IRBENE (Lat. 57.55, Long. 21.85 degree) has been established by the assistance of the Onsala Space Observatory of Chalmers University of Technology (Sweden). IRBENE (LV04) is more as national Permanent GPS-station and it is included in EUVN97 project. Besides, our GPS-station RIGA functions together with Satellite Laser Ranging System, absolute gravity station, ground water registration spot and fulfil tasks for international geodetic society.

We think that implementation of the EUVN97 project (Table 1) will be a good base for developing of the Vertical Reference Network of Latvia. We would like to use the results of EUVN97 campaign.

The total number of points measured by GPS till the beginning of 1998 are about 1300. GPS measurements are carried out in 19 out of 26 administrative districts.

EUVN ident. code	Site name	Lat (odec)	Lon (odec)	Tide gauge sites (Y or N)	Type of receiver	Team
RIGA	Riga	56.949	24.059	N	T R	Astronomical Observatory, University of Latvia
LV01	Skulte	57.316	24.410	Y BSL93	T SSE	State Land Service
LV02	Liepaja	56.515	21.001	Y	T SSE	State Land Service
LV03	Ventspils	57.396	21.538	Y	T SSE	State Land Service
LV04	Irbene	57.554	21.852	N	T SSE	State Land Service

Table 1. EUVN97 site list in Latvia.

We would like to express acknowledgment to the countries and colleagues for great aid and we hope for future cooperation.

Nordiska Kommissionen för Geodesi

The New Norwegian National Geodetic Network - EUREF89

by

Oddgeir Kristiansen and Bjørn Geirr Harsson

STATENS KARTVERK

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Contents

The New Norwegian National Geodetic Network - EUREF89

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1 INTRODUCTION

The establishment of a new national geodetic network was given priority in Part 3 of the Norwegian National Mapping Plan. This Plan was officially promulgated in Government White Paper Number 1984:4 under the sub-heading "Geodesy". Completing this task therefore became one of the objectives assigned to the Geodetic Institute, a division of the Norwegian Mapping Authority, "Statens kartverk". However, in view of the dramatic changes taking place in satellite geodesy during the 1980's, it was difficult to select the best time to address this task.

A provisional datum for use by the offshore industry had meanwhile already been established early in 1989. This datum was called "WGS84*SEA" and was based on observations in the North Sea – Fennoscandia region using the TRANSIT Doppler Satellite System, the predecessor of GPS (Global Positioning System). WGS84*SEA became quite widely used for land purposes, even though it had been designed primarily to serve offshore needs.

In 1989, the Geodetic Institute participated in a major GPS campaign observing at 93 European stations, including seven in Norway. These were Eigeberg (Stavanger), Hønefoss, Vigra (Ålesund), Vassfjell (Trondheim), Renskåsen (Bodø), Tromsø and Domen near Vardø. Each of these stations was already, or has since been, connected to the national First Order Triangulation Network.

Meanwhile, at the EUREF Commission meetings in 1991 and 1992, it was recommended that the EUREF89 Reference System should be used as the European geodetic datum in the same way as North American Datum 1983 (NAD83) had become accepted in North America.

Post-processing of the 1989 observation data was completed by two independent computing centers which subsequently reported to the EUREF Commission, a sub-commission of the International Association of Geodesy (IAG). Solutions prepared by the Bern University Computing Center were used as a basis for final adjustments, which were then accepted in the Summer of 1992. The Norwegian Mapping Authority subsequently decided to establish EUREF89 as an official national geodetic datum from 1 January 1993. The objective then was that this datum would replace both of the existing datums, NGO1948 that had been used for technical and large scale mapping series, and ED50 that had been used for topographic and geographical mapping.

The EUREF Commission, however, recognising technical developments, arrived at a significant recommendation in 1993/94. This was that the improvements in GPS with its international tracking networks and reference frames should be employed to further refine and adjust EUREF89. Accordingly, the Geodetic Institute agreed plans for major GPS activities in the late summer of 1994. All of the original Norwegian EUREF89 stations, except Vassfjell, were re-observed and in addition connected to the SATREF stations. The permanent stations for navigational and geodetic purposes in Norway are called SATREF stations. Observations at the Vassfjell station were now found to be impossible due to electro-magnetic interference.

The EUREF-NOR94 GPS Campaign formed the basis for the initial establishment of the required new geodetic framework for Norway. It was followed up by a similar GPS campaign in 1995, EUREF-NOR95. The receivers during these campaigns were placed at each site for at least three days, and data were collected for twenty-four hours each day. These sites are called 3D stations or "Base Stations". The 3D indicates three days of observation and three dimensional adjustment.

Later campaigns observing for 4-8 hours at stations in between the base stations resulted in so-called "Main stations". These two kinds of stations comprise what in Norway is called the "Stamnett". The terms "3D station" and "Main station" are used in the context of the new Norwegian national geodetic network so that they are clearly distinguished from the stations in the existing classical triangulation network, especially with respect to the methods used to determine their respective coordinates.

EUREF89 is thus the name of the new geodetic datum for Norway. However, in view of the very high accuracies now available from world wide satellite observations, these new stations' coordinates will eventually become imprecise due to tectonic plate motion, and in Scandinavia, land-up lift. EUREF89 must therefore be clearly understood to be a statement of the positions of the measured stations as they were at the beginning of 1989. Continental drift and land-up lift is of course being continually monitored for scientific purposes, and, with this knowledge, future measurements can be corrected back to the 1989 epoch.

Practical positioning in the Norwegian area will in future be relative to the Stamnett, where the station coordinates are computed in the geodetic datum EUREF89. These coordinates will now remain unchanged for the foreseeable future. This also means that coordinates which are computed for future Stamnett stations will also remain unchanged for as long into the future as it is possible to foresee.

2 STAMNETT OBSERVATIONS - 3D STATIONS

2.1 RECONNAISSANCE AND ESTABLISHING 3D STATIONS

The work of creating the Stamnett began with the selection of the 3D stations that would be occupied in 1994. These were negotiated and agreed with the Flight Inspection Section (FIS) of the Norwegian Civil Aviation Administration (CAA (Norway)), and with the Norwegian Roads Directorate (NRD).

Detailed site requirements were issued well beforehand, while the Geodetic Institute designed how each station would be monumented in terms of foundations and type of fine mark. Some 70 marker bolts were produced by the Institute's workshops for this purpose. Each bolt was threaded so that a GPS antenna could be screwed onto it and thereby ensures forced and repeatable centring. The next 700 marker bolts for Stamnett stations were produced in manganese brass in accordance with Norwegian Standard NS 16565, by the company Mjøs Metallvarefabrikk. Manganese brass is stronger than pure brass and more resistant against corrosion. Fig. 1 shows the construction of a marker and a 3D station.

Reconnaissances to select stations were carried out together with CAA (Norway) and NRD. The efforts at coordination had the additional benefit that the local authorities and CAA (Norway) offered to participate at their own expense in the actual station construction work. They also agreed to assist with the actual observation programs.

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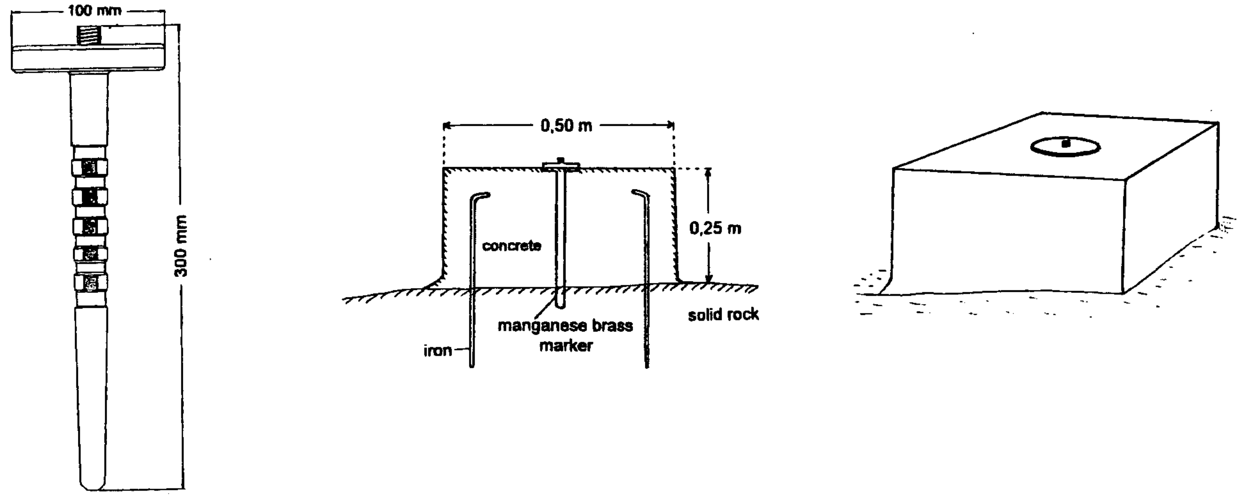


Figure 1. The marker constructed for Stamnett stations, and a typical 3D station

From the geodetic viewpoint, there was the absolute requirement that all stations would be built on bedrock, while there were clear needs for accessibility, security and a clear horizon at each station. Those who had previous knowledge of the areas made many excellent suggestions, and every effort was made to arrive at the most appropriate compromise solution in all cases. The final selections were made jointly by representatives from the Geodetic Institute, and from local authorities. Detailed building specifications for Stamnett stations were made in case of assistance in station construction work.

2.2 EUREF-NOR94, EUREF-NOR95 AND EUREF-NOR96 GPS CAMPAIGNS

A total of 11 SNR-8100 TurboRogue GPS receivers with Dorne & Margolin T antennas were used during the first EUREF-NOR Campaign in 1994, named EUREF-NOR94. These receivers were borrowed from UNAVCO in USA. The TurboRogues, together with the necessary associated equipment, were used to observe in a period of 24 days in September and early October at 3D stations in a total number of 63. The stations involved were spread all over the country. The observation time at each station was three days, 24 hours a day, starting at 00 GMT day one. Throughout the campaign, observed data was recorded at 30 second sampling intervals, and with an elevation cut-off angle of 15 degrees.

The observation program for 3D stations continued the following year in the EUREF-NOR95 campaign lasting from late August to early October 1995. In this campaign 51 stations were included. During this campaign, however, only eight TurboRogues were available, all of them coming from the Geodetic Institute's own resources. As in the previous year, the same Dorne & Margolin GPS antennas, observation procedure and observation time was used.

Apart from a few small changes, the whole observing campaign was completed according to plan. Station moves took place on schedule, with only few exceptions. The moves worked well on all but three where, for a variety of reasons, the receivers were not initialised until the following morning.

An additional observation campaign was mounted in the period from 26 August to 6 September 1996, equipped entirely with Turbo-Rogues and antennas of the same type as used in EUREF-NOR94 and 95 campaigns. The campaign was named EUREF-NOR96, and the goal was to observe at the remaining 3D stations. In all, 28 stations in North Norway were included. Eleven Turbo-Rogues were borrowed for this purpose from UNAVCO (USA), with the Geodetic Institute providing four from its own resources. The same observing specifications and routines were employed as in the previous two years. Having so many receivers available made it possible to complete this campaign with only a single move of each receiver.

The campaign was successful for 25 of the required 28 stations, while unfortunately, the quality of the data collected at three stations was not accepted in the computations.

Equipment losses during the campaigns were very slight, only two compasses and a flashlight being found to be missing. More importantly, none of the small TurboRogue flash cards were lost, implicitly indicating that none of the recorded observations were lost in transit between the observing sites and the point of downloading. Finally, the cooperation in the field between the various agencies involved went remarkably well.

2.3 DATA RECOVERY AND PREPARATION

TurboRogue receivers are arranged so that observed data is recorded on flash cards in the Turbo Binary format. The normal procedure then is for the data to be read into a PC. During the campaigns, data was read to a PC as near to the station as practicable for onward transfer by diskette. Otherwise, the flash cards were used as the delivery medium. In some cases, it was possible to download the observed data to the Geodetic Institute through the county mapping offices' computer network. On arrival at the Institute's computers, the data was later converted into RINEX format.

It has already been mentioned that the object was to establish a new national reference frame, EUREF89, in Norway. Therefore it was important to connect the new measurements to already existing EUREF89 stations in Europe. Accordingly, tracking data from 12 European IGS stations were obtained from the IGS Data Centre at JPL (Jet Propulsion Laboratory) in USA, all in RINEX format. Similarly, the tracking data from seven SATREF and five SWEPOS stations were obtained in RINEX format.

3 STAMNETT OBSERVATIONS - MAIN STATIONS

The Geodetic Institute planned major field campaigns for the 1995 season, involving observations at over 400 main stations in the Stamnett. These campaigns were concentrated in a Western Block extending from Rogaland to Sør-Trøndelag, and an Eastern Block around the Oslo Fjord. At the same time, the Stamnett in the area of Bodø in central Nordland would be completed by additional observations. The plan was then to complete the remaining Stamnett field observing schemes in the following year, 1996.

The actual field measurements were arranged as a single 4-hour session during the day, and a night session of at least 8 hours. In practice, the idea was to observe two 4-hour sessions during the night hours. On some of the days, two daytime sessions were also achieved giving a total of 16 observing hours on those days. It was concluded from trials that the overall observing plan for the season was indeed realistic, although it would involve long working hours for the field parties. The main stations were observationally well connected to the 3D stations.

Two observing groups carried out the season's field campaigns, partly because there were two types of geodetic GPS receivers to hand.

The Geodetic Institute's six Osborne TurboRogue GPS receivers were used for the Eastern Block around the Oslo Fjord and for the observations in the Bodø area in Nordland. A field party consisting normally of three observers recorded the observations during the period from early June to the middle of August.

The actual TurboRogue observations were recorded as usual in the so-called Turbo Binary format on the receivers' flash cards, which had, in fact, a data capacity sufficient to record up to four days' continuous observations. Meanwhile, the normal practice in 1995 was for the party leader to visit with each of his team once each day, and the opportunity was taken to download the observational records to his PC. Each day, therefore, the observational records from one to three observing sessions were secured. All other communications between the field party members were by mobile telephone.

The NRD, meanwhile, had a total of six Leica geodetic GPS receivers at their disposal, and these were deployed to the Western Block. The campaign observations were recorded between early June and early November, with a break between July and October when the receivers were required for routine NRD purposes. The Leica field party consisted of five to six observers. The data storage capacity of the Leica receivers was more limited, and it was essential to download the data each day without fail. This constraint was relieved, however, because there were two PCs at the disposal of the field party.

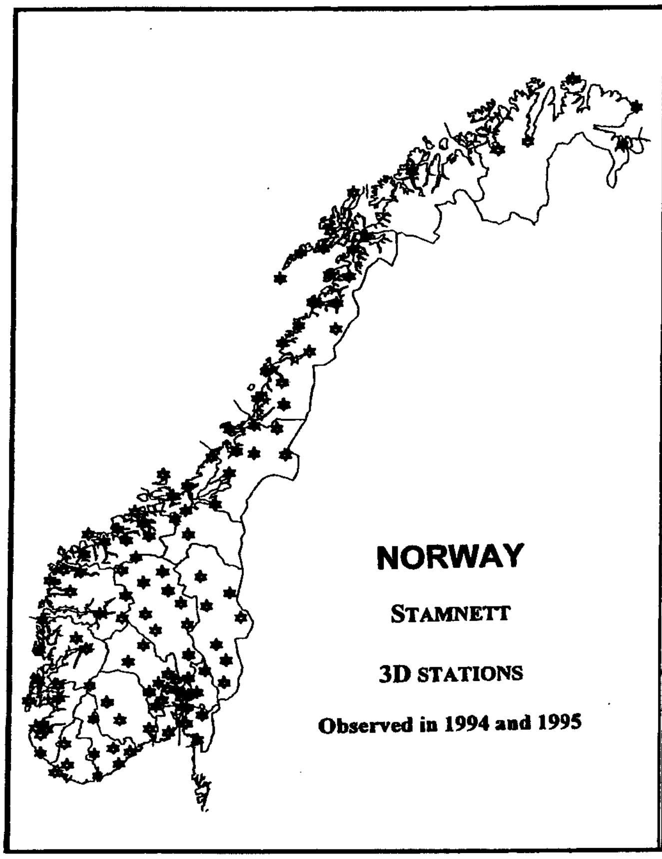
The observing specifications were that satellite data was only to be recorded when satellites were visible above an elevation of 15°. The TurboRogues were set to record data at 30-second intervals, while the Leica receivers used a 15-second interval. Meanwhile, the actual process of downloading a day's worth of observations to PC normally took over an hour for all the receivers in the team, and the actual volume of these records for the whole season would predictably be large. A system of observed data file naming was therefore devised in order to enable clear and unambiguous file identification with respect to the station, date, and session.

The original observational plan was closely followed, and the two field parties in the course of the field season occupied approximately 440 Stamnett stations. Some delays were encountered early in the season when some of the TurboRogues needed repair, although the lost observing time was immediately recovered. The last day of observation was on 5 November, set against a planned field completion date of 4 November.

The recorded observational data were during the autumn transferred to the Geodetic Institute's computers where they were converted to the RINEX format and their vectors were computed. The Ashtech processing software was employed for the TurboRogue data, while the Leica records were processed using the proprietary software that is delivered together with the receivers.

The usual important quality control procedures were executed before undertaking network adjustments. Special attention was given to checking all antenna data. The adjustments were computed holding 3D stations' EUREF89 coordinates fixed. The object, here, was to assess the quality of the networks. The first results indicated that a small number of additional vectors needed to be measured, and these were recorded by supplementary observations in the period March to June 1996.

Major Stamnett campaigns were also planned for 1996, with the aim of completing some 450 Stamnett stations. Two campaign areas were involved, one covering the remaining part of South Norway, and the other completing the remainder of the North Norway area.



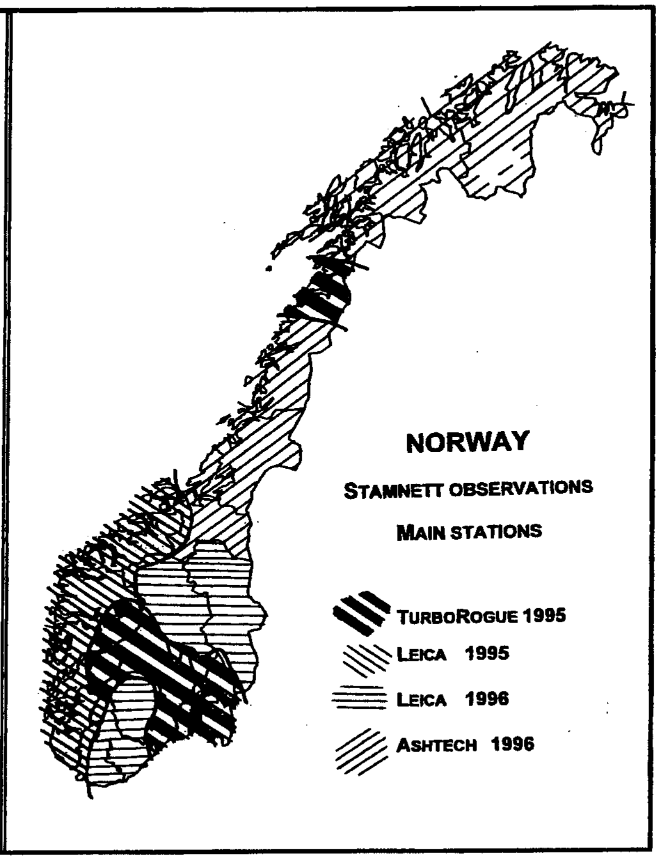


Figure 2. Stamnett, 3D stations, observed in 1994 and 1995.

Figure 3. Stamnett, main stations, observation areas 1995 and 1996.

As in 1995, two separate field parties carried out the observations. One was again equipped with the NRD's six Leica receivers, while the other used the Geodetic Institute's six new Ashtech receivers with Dorne & Margolin antennas. No TurboRogues were used in these field parties. The same mask angle and observing intervals were used as in 1995, except that the Ashtech receivers were set to sample at 20 second intervals which is an Ashtech standard.

Observations in the Southern Block started at the end of May. The field party was equipped with Leica receivers and consisted of four observers, from the Geodetic Institute, the County Survey Office, the County Roads Department and the NRD. The party began in South, Vest-Agder, and finished off the season in Hedmark on 30 August, having successfully observed at all the remaining 173 Stamnett stations in South Norway. Observed data records were downloaded to PCs using the same well-tried routines as in 1995.

The Ashtech field party began in Sør-Trøndelag early in June and worked through North Norway. The three observers in this field party were provided by the Geodetic Institute, and the various County Survey Offices and Roads Departments. In this field party, each observer was provided with a PC, and was thus able to take care of his own data downloading from the receivers. This saved much time, which would otherwise have been spent traveling.

The Ashtech field party finished the last observations in Finnmark on 13 September. At this point, the planned observations for the 1996 season were complete, bringing the overall completion of the Stamnett up to 90% nation-wide, as had earlier been projected during the original planning in 1994. The remaining 10% comprised the more deserted areas of the country.

The total achievement of the 1996 campaigns comprised therefore a total of 479 Stamnett stations.

4 THE TERRESTRIAL REFERENCE FRAME

During recent years, the international geodetic community has become increasingly interested in the study of crustal movements and Earth orientation. Space geodetic techniques such as VLBI (Very Long Baseline Interferometry), SLR (Satellite Laser Ranging), LLR (Lunar Laser Ranging) and GPS have been used. Common requirements for these investigations are the need for stable well-defined reference systems. All the relevant observations and the adopted theories and models used to describe the dynamics of the Earth and the surrounding space need to be coordinated. The selected coordinating centre has been the Central Bureau of the International Earth Rotation Service (IERS) in Paris, France. Two reference coordinate systems are needed, a celestial and a terrestrial reference system. The Central Bureau of the IERS is responsible to define in detail on how users can materialise such conventional reference systems. Clearly, with much data being observed and analysed independently by a number of separate institutions, comparative additional analysis requires that the results be all related to common standards. The IERS therefore recommends the appropriate physical models and standards to be used. These recommendations are published every year in the IERS standard. So long as these recommendations are followed, all analysis centres will be using the same reference system, an IERS Reference System (IRS) which can be either an inertial system (ICRS) or a terrestrial system (ITRS).

The term «reference system» includes these recommendations, with both the theories in the definitions and the descriptions of the physical environments. However, reference systems also need to be given physical and measurable value. This is achieved by assigning reference system coordinates and parameters to physically measurable artefacts. Star coordinates are naturally used to locate, orient and scale the celestial frame, while Earth station coordinates are used for the terrestrial frame.

Every year, the IERS publishes coordinates and velocity vectors of selected stations globally distributed on the Earth's surface. The velocity vectors arise due to the continental tectonic drift phenomenon. The station coordinates, meanwhile, come from an adjustment that incorporates the data from the different techniques, and thus provide the basis for the IERS Terrestrial Reference Frame (ITRF). It is Earth fixed, and therefore moves and rotates with the Earth.

All of the sites which deliver results to the IERS make either continuous or regular observations with one or more of the above mentioned techniques. In Norway, Tromsø and Ny-Ålesund are the two IERS contributing stations. Tromsø has been observed twice using mobile VLBI equipment, and once using mobile SLR technology, while it has been continuously tracking GPS satellites since 1987. The most northerly station in the IERS network, meanwhile, is in Ny-Ålesund on Spitzbergen where an Osborne SNR-8 Rogue GPS receiver has been in permanent operation since 1991.

Additionally, a permanent VLBI radio telescope at Ny-Ålesund was officially opened in August 1995, although it had been in place since 1994.

The ITRF Velocity Field thus represents the small tectonic movements (up to a few cm per year) which are continuously taking place on the Earth's surface. It is clearly therefore important that, when quoting station coordinates, the epoch of those coordinates are also given. Thereafter, with the help of the Velocity Field, it is possible to correct them backwards or forwards in time to compare with data from different epochs.

Station coordinates in the ITRF are given in a cartesian coordinate system (x,y,z). Subsequent transformation into geodetic coordinates (latitude, longitude and height) introduces the need for a defined ellipsoid. The IERS recommends the GRS80 Ellipsoid (Geodetic Reference System 1980). It must be noted, here, that heights in this context are above the chosen ellipsoid, and *not* heights above the geoid.

5 BACKGROUND FOR THE CHOICE OF ANALYSIS STRATEGY

Satellites in orbit around the Earth can be used to measure the baseline length between sites located on the Earth's surface. Both GPS and SLR have proven to be very accurate. A major advantage of GPS compared to SLR is the truly global and continuous GPS network. GPS can give continuous time-series, but it is a high orbiting system and therefore loosely bound to the centre-of-mass. SLR uses low orbiting satellites and is therefore tightly bound to the planet's centre-of-mass. Geocentric distances from SLR measurements, meanwhile, have shown themselves over many years to be of high accuracy and consistency, and are thus well suited to defining scale and the origin.

Realisation of a terrestrial reference frame requires a scale, an orientation and an origin. The scale is defined by fixing the speed of light and the GM (the product of the Gravitation Constant and the Mass of the Earth) value to the IERS standards. Using a well geometric distributed network of tracking stations attached to the Earth lithosphere in analysis will form a polyhedron. The solid Earth, which is realised by the origin of this polyhedron, would move with respect to the actual centre-of-mass. The terrestrial reference frame will be realised by the origin (or the geocentric distances) of the polyhedron and the baseline lengths.

Two different approaches have commonly been used in GPS analysis, the fiducial approach and the no-fiducial approach. In the fiducial approach some selected station coordinates are constrained to an already established reference frame, such as the latest IERS solution. To compute the coordinates of one station it is not necessary to compute the distances to all other stations in the frame. One need only to state the distances to any three other noncollinear stations *[Goldstein, 1980]*. Thus, once the positions of three stations are determined, constraints will fix the positions of all the remaining sites. This reduces the number of degrees of freedom in the fiducial technique to nine. The distances between these fixed stations are constants and therefore the number of degrees of freedom reduces to six. This means that in a fiducial approach it will be sufficient to fix the three cartesian station coordinates on the first station, two on the second station and only one on the third station. Normally one chooses to constrain coordinates from at least three globally distributed stations. Unfortunately there are some disadvantages with the fiducial technique. Some of the coordinates that are constrained may contain errors. These errors will be carried into the solution as errors when solving for parameters such as station coordinates, satellite orbital parameters and Earth orientation parameters. Another problem is in the case of realising a stable reference frame between epochs, such as creating station time-series. If one or more of the chosen fiducial sites drops out, and other stations are chosen as fiducial, this may lead to a realisation of a slightly different reference frame. In other words, reference frame consistency may be hard to maintain.

To avoid these problems, another approach is to put loose constraints on all sites in the solution. This requires a homogenous geometric distribution of satellite tracking stations, and will give a better internally consistent solution. The rotational part will be oriented around the true origin as it is realised in the gravity field model and in the included measurements. The rotational part will however, not be oriented around the origin realised through a priori coordinates, as is the case using the fiducial approach. This method is called the "fiducial free" technique, or the generation of a "free network" solution. The result of such a free adjustment then needs to be rotated and transformed to fit a previously defined reference frame such as the ITRF. The big advantage of this method is that the solution is independent of errors in individual station coordinates. This only becomes important when transforming to the pre-defined reference frame, but, on the other hand, such errors will also emerge during this transformation process, and can thus be corrected without having to repeat the whole of the GPS analysis. Errors in the eccentricity vectors can also be encountered here.

The transformation from the free network solution to ITRF can be written as follows:

$$\begin{pmatrix} x \\ y \\ z \end{pmatrix} = \begin{pmatrix} X \\ Y \\ Z \end{pmatrix} + \begin{pmatrix} t_x \\ t_y \\ t_z \end{pmatrix} + \begin{pmatrix} s & -\theta_z & \theta_y \\ \theta_z & s & -\theta_x \\ -\theta_y & \theta_x & s \end{pmatrix} \begin{pmatrix} X \\ Y \\ Z \end{pmatrix} \tag{1}$$

where (x,y,z) are the GPS derived coordinates, (X,Y,Z) are the ITRF solution, (t_x,t_y,t_z) are an offset in origin, s is the scale difference and $(\theta_x,\theta_y,\theta_z)$ represent differences in orientation.

Coordinates from 15 stations in Europe were held fixed for defining EUREF89 from the results of the 1989 campaign. Each of these stations was on the European Tectonic Plate, and they consisted of seven VLBI and eight SLR sites. ITRF90 was used as the reference frame, and the epoch was 1989.0 (1 January 1989). This reference frame was given the name ETRF89 (European Terrestrial Reference Frame 1989), and the associated reference system was called ETRS89 (European Terrestrial Reference System 1989).

The EUREF Commission [*Boucher, 1994*] recommended that the following procedure should be used to determine ETRS89, and to be able to include subsequent improvements both of GPS, international networks and the reference frame.

(i) Compute the GPS data in ITRS at Epoch t_c

The latest version of ITRF was used. After the EUREF89-NOR94 campaign it was ITRF93, therefore ITRF93 has been used as the IERS reference frame in all the campaigns to realise EUREF89. This entails obtaining the ITRF coordinates at Epoch (t_0) from the IERS. These are then updated to the observation epoch (t_c) using velocity components for the IGS stations that had been used for transforming from a free adjustment to the ITRF. The velocity components v are obtained from the ITRF Velocity Field, and new coordinates are computed using:

$$x_{ITRF93}(t_c) = x_{ITRF93}(t_0) + v_{ITRF93}(t_c - t_0) \tag{2}$$

Another method of verifying the solution's accuracy is to include stations with coordinates that are already well determined in an established reference frame such as ITRF93, and then compare these coordinates with the solution.

7 ANALYSIS STRATEGY

The results from the EUREF-NOR94 and EUREF-NOR95 campaigns were analysed based on a regional approach. Unfortunately it was not possible to use the same selection of international permanent stations for the EUREF-NOR96 computation. Consequently the data from the EUREF-NOR96 campaign was not used in this analysis. More details for the reason are given in Chapter 10.

Common to both campaigns, NOR94 and NOR95, are the use of precise GPS carrier phase and P-code pseudorange measurements. In addition precise GPS ephemerides and GPS calculated Earth Orientation Parameters (EOP) value from one of the IGS analysis centres, namely NASA's JPL, were used. Data were processed with the JPL GIPSY (GPS Inferred Positioning System) software [*Webb & Zumberge, 1993*]. In GIPSY all parameters are solved for in a batch-sequential square root information filter. The troposphere and the clock parameters, which are typically correlated process noise parameters, are solved for in a Gauss Markov process. The troposphere is solved for as a random walk parameter, while the clocks are solved for as white noise.

In this regional analysis, IGS data from 12 sites in continental Europe were used together with data from 11 mobile TurboRogue receivers at sites in Norway for the NOR94 data and 8 mobile TurboRogue receivers for the NOR95 data. In addition, data from respectively seven and five permanent tracking stations in Norway (SATREF) and Sweden (SWEPOS) were used. This provided a well distributed set of regional IGS sites from Ny-Ålesund, Svalbard in the north, Metsahovi, Finland in the east, Matera, Italy in the south-east and Mas Palomas on the Canary Islands in the south-west. The mobile receivers were placed at each of the sites for at least three days, and data were collected for twenty-four hours each day. The GPS receiver type mounted on the European IGS sites used was a mixture of TurboRogues and Rogues, and the antenna type was a mixture of Dorne & Margolin T, R and B antennas. In the analysis the station coordinates were solved for using weak constraints (10 and 100 meters).

The analysis strategy that was used is summarised as follows:

- Precise orbital parameters computed by JPL in the J2000 inertial system were used.
- Earth rotation parameters obtained from JPL were used.
- Opening coordinates for the IGS stations were obtained in ITRF93 at epoch 1993.0 and were reduced to the observing epoch using the ITRF93 Velocity Field.
- All station coordinates were corrected for ocean loading using coefficients provided by Hans-Georg Schernek of the Onsala Space Observatory.
- Clock errors were treated as white noise, with the hydrogen maser in Onsala chosen as the primary reference oscillator. The hydrogen masers in Ny-Ålesund and Wettzell were used when analysis difficulties were encountered, or when the instrument in Onsala itself had problems.
- At every site a nominal zenith dry tropospheric delay of 200-230 cm was given, and assumed to be the zenith dry delay. At every site 10 cm was used as nominal value for the wet zenith delay. The remaining zenith delays were solved for as random walk using a rate of change of 1 cm per square root of the hour.

- The free network technique was chosen. Relatively tight (10 cm) constraints were applied to the IGS stations, while the rest of the stations were constrained to be within 100 m. Only European stations were included and, because of the data obtained from JPL, it was not necessary to solve for orbital parameters.

Conversion from the free network results to ITRF93 was by transformation of each day's solution separately. Seven European IGS stations were used within the Helmert Transformation (equation 1) and the coordinates were diagonally weighted.

For each daily free-network solution inner constraints were applied to the covariance matrix leaving the coordinates unchanged [*Heflin, 1993*]. Five European IGS sites (Ny-Ålesund on Svalbard, Metsahovi in Finland, Wettzell in Germany, Madrid in Spain, and Matera in Italy) were used to estimate the transformation parameters. Statistical information from the transformed covariance matrix and the given standard deviations for ITRF93 were accounted for in this calculation. These selected sites define a strong geometrical network in Europe that can approximate a square of sites with Wettzell lying in the middle of Europe forming four triangles. All the mobile receivers lie in the two triangles in the north, which are Ny-Ålesund, Madrid and Wettzell in the west, and Ny-Ålesund, Metsahovi and Wettzell in the east. This implies that two of the IGS sites are very important sites in the realisation of EUREF89. These are Ny-Ålesund in the north and Metsahovi in the east.

8 REVIEW OF ANALYSIS RESULTS

All station coordinates were computed in the ITRF93 reference frame and reduced to the current epoch. These sets of coordinates were entitled EUREF-NOR94 and EUREF-NOR95. The inferred coordinate precision for each station was taken to be the RMS about the mean of the daily solutions.

Figure 4, 5 and 6 shows the daily scatter around the weighted mean value in the north, the east and the height for three of the constrained IGS stations Ny-Ålesund, Metsahovi and Madrid, respectively. Each point in the plots represents one single day solution. Take notice of the scale differences used for the horizontal components (figure a) and the vertical component (figure b). For the horizontal coordinates we have used (-10,10) mm, while for the vertical component we have used (-15,15) mm. Figure 7 and 8 shows the daily scatter of two additional IGS sites, namely, Tromsø and Onsala. The computed repeatabilities of the daily scatter, using equation 7, varies from 2 to 3.5 mm in the horizontal components and from 4 to 7 mm in the height for these IGS sites. These results are from the EUREF-NOR95 campaign. Figure 10 compares the repeatabilities of all the included IGS stations from the EUREF-NOR94 and the EUREF-NOR95 campaign. These figures clearly show that the repeatabilities are better than 6 millimetres in the horizontal components and better than 10 mm in the vertical. There is one exception and that is the height component for Kiruna in the 1994 campaign.

Figure 9 shows the daily scatter around the weighted mean value in the north, in the east and in the height for three mobile GPS receivers, respectively, MR09, NO13 and NO12. The repeatabilities are better than 2.3 mm in the north, 1.5 mm in the east and 2.5 mm in the height.

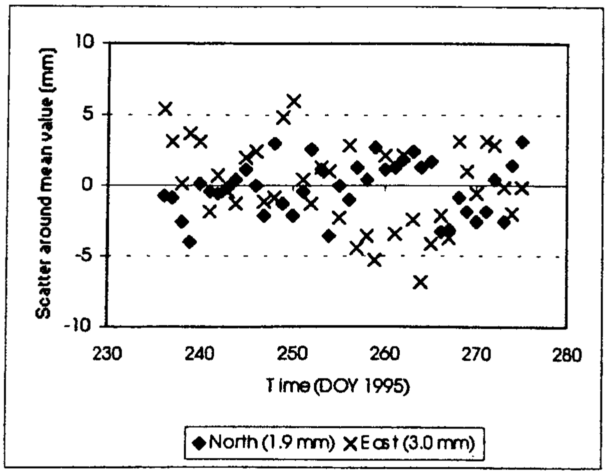


Figure 4a.The daily scatter around the mean north and the mean east for the IGS Rogue receiver in Ny-Ålesund, Svalbard. The repeatability is 1.9 mm in the north and 3.0 mm in the east.

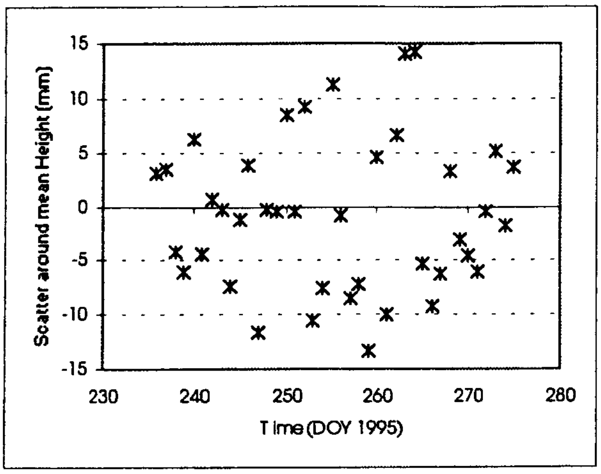


Figure 4b. The daily scatter around the mean height for the IGS Rogue receiver in Ny-Ålesund, Svalbard. The repeatability is 6.9 mm.

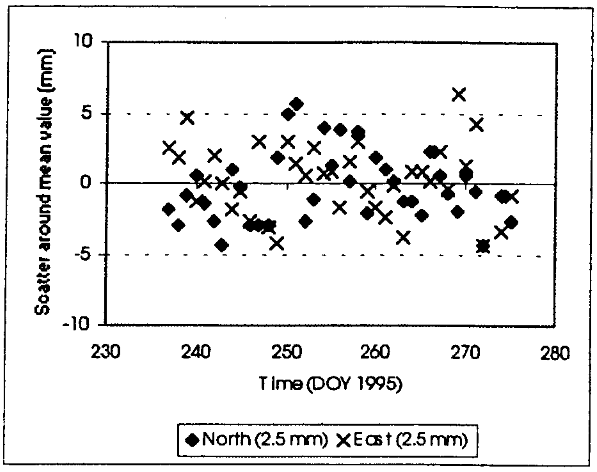


Figure 5a. The daily scatter around the mean north and the mean east for the IGS MiniRogue receiver in Metsahovi, Finland. The repeatability is 2.5 mm in the north and 2.5 mm in the east.

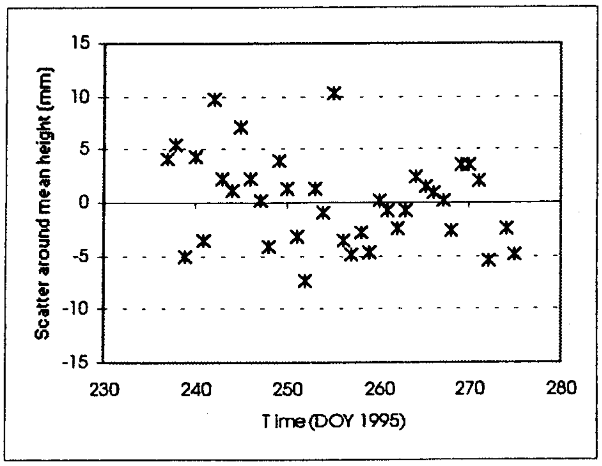


Figure 5b. The daily scatter around the mean height for the IGS MiniRogue receiver in Metsahovi, Finland. The repeatability is 4.1 mm.

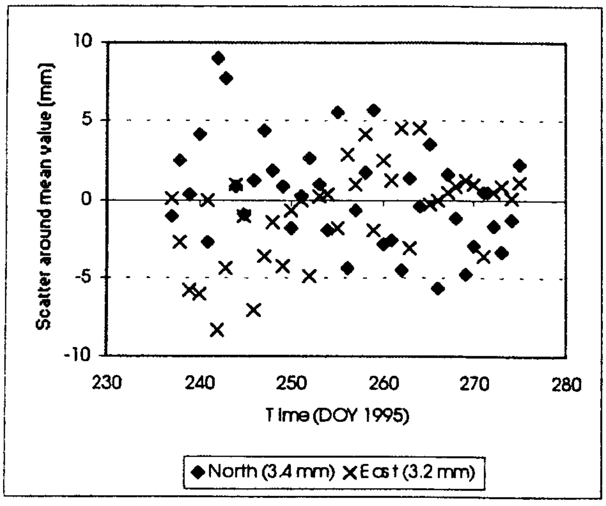


Figure 6a. The daily scatter around the mean north and the mean east for the IGS Rogue receiver in Madrid, Spain. The repeatability is 3.4 mm in the north and 3.2 mm in the east.

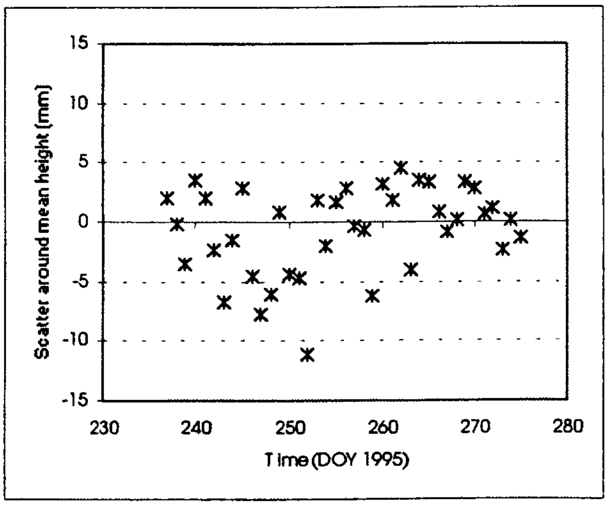


Figure 6b. The daily scatter around the mean height for the IGS Rogue receiver in Madrid, Spain. The repeatability is 3.7 mm.

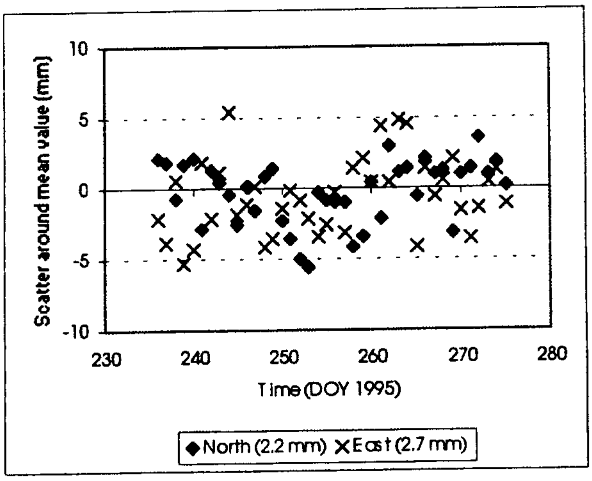


Figure 7a. The daily scatter around the mean north and the mean east for the IGS Rogue receiver in Tromsø, Norway. The repeatability is 2.2 mm in the north and 2.7 mm in the east.

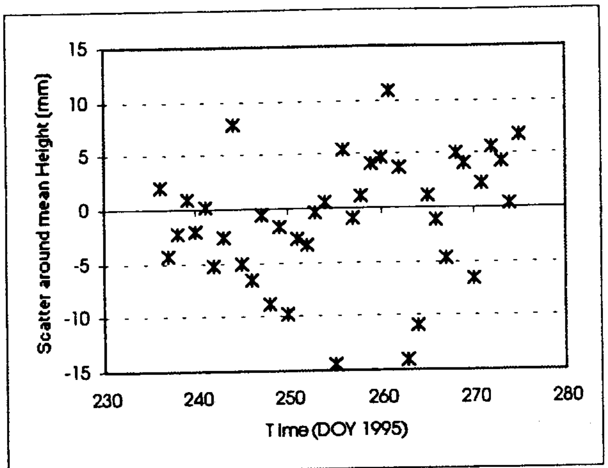


Figure 7b. The daily scatter around the mean height for the IGS Rogue receiver in Tromsø, Norway. The repeatability is 5.7 mm.

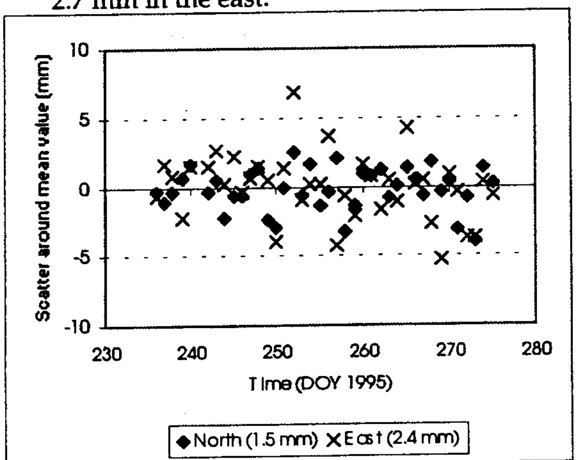


Figure 8a. The daily scatter around the mean north and the mean east for the IGS TurboRogue receiver in Onsala, Sweden. The repeatability is 1.5 mm in the north and 2.4 mm in the east.

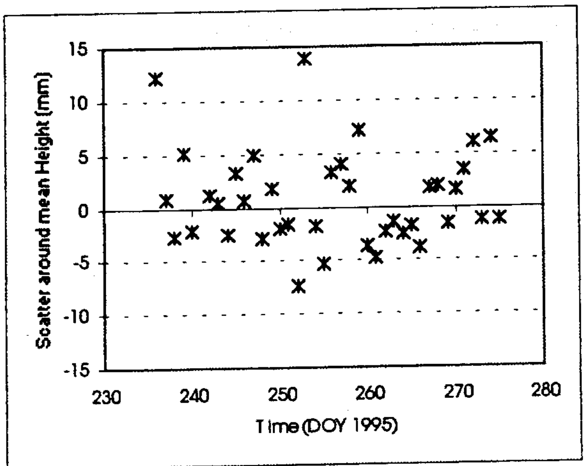


Figure 8b. The daily scatter around the mean height for the IGS TurboRogue receiver in Onsala, Sweden. The repeatability is 4.5 mm.

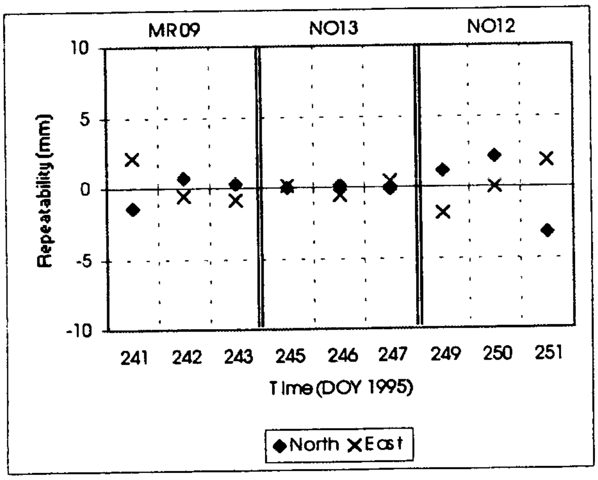


Figure 9a. The daily scatter around the mean north and the mean east for three mobile TurboRogue receivers in Norway. The repeatabilities in the north are 0.9, 1.4, and 2.3 mm and in the east 1.4, 0.4 and 1.5 mm for MR09, NO13 and NO12, respectively.

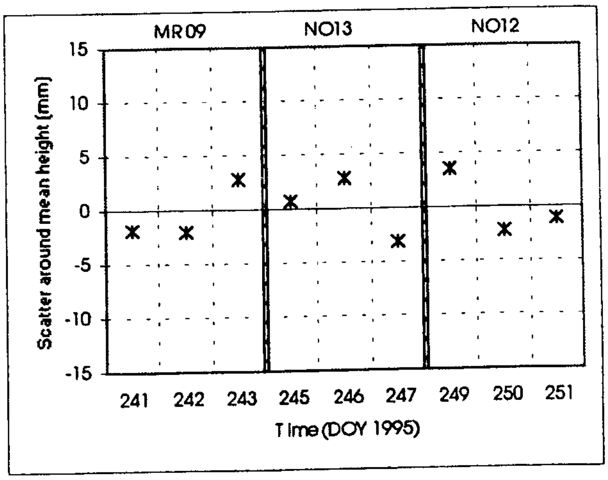


Figure 9b. The daily scatter around the mean height for three mobile TurboRogue receivers in Norway. The repeatabilities are 2.3, 1.5 and 2.5 mm for MR09, NO13 and NO12, respectively.

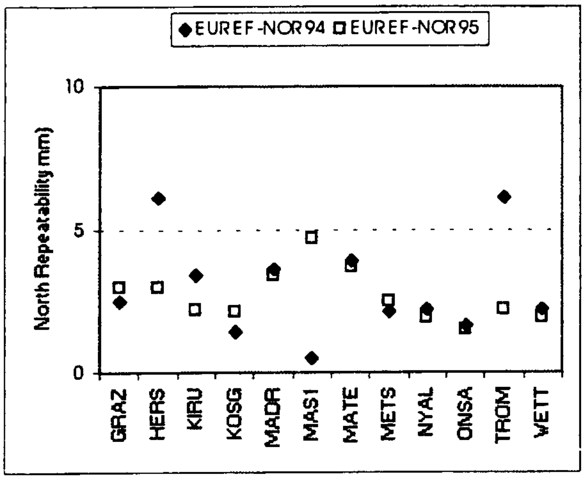


Figure 10a. The North repeatability for the IGS stations from the EUREF-NOR94 and the EUREF-NOR95 campaigns.

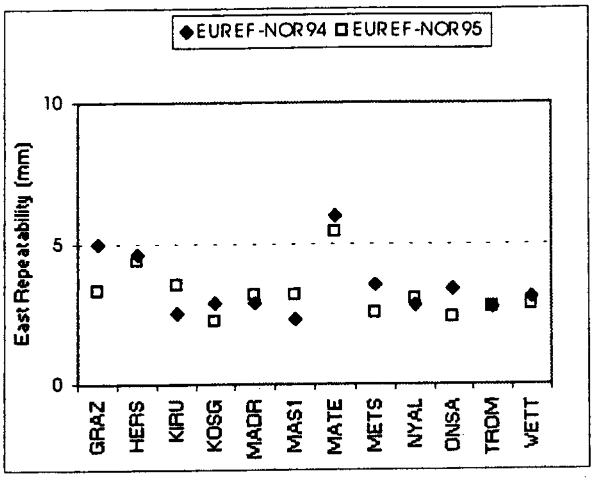


Figure 10b. The East repeatability for the IGS stations from the EUREF-NOR94 and the EUREF-NOR95 campaigns.

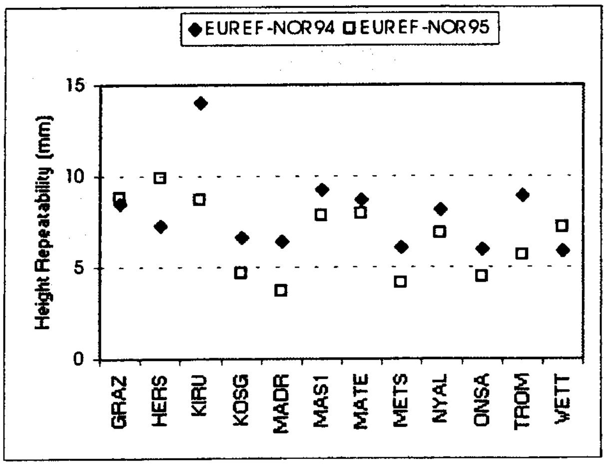


Figure 10c. The Height repeatability for the IGS stations from the EUREF-NOR94 and the EUREF-NOR95 campaigns.

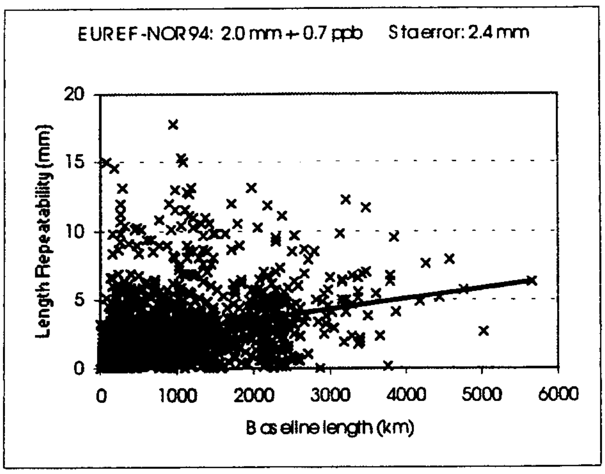


Figure 11a. Length repeatability of all GPS receivers from the EUREF-NOR94 campaign. The overall repeatabilities are 2.0 mm ± 0.7 ppb with a standard error of 2.4 mm.

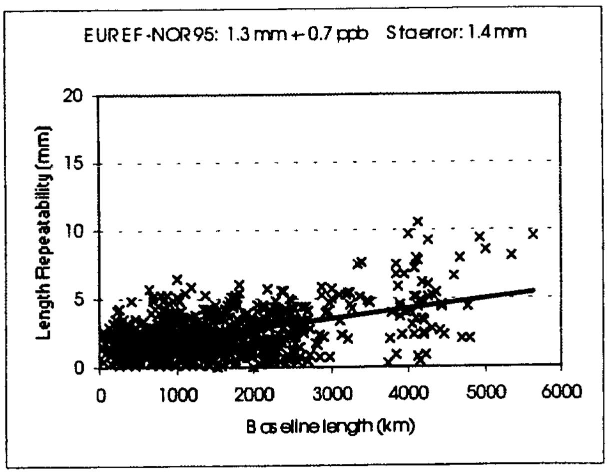


Figure 11b. Length repeatability of all GPS receivers from the EUREF-NOR95 campaign. The overall repeatabilities are 1.3 mm ± 0.7 ppb with a standard error of 1.4 mm.

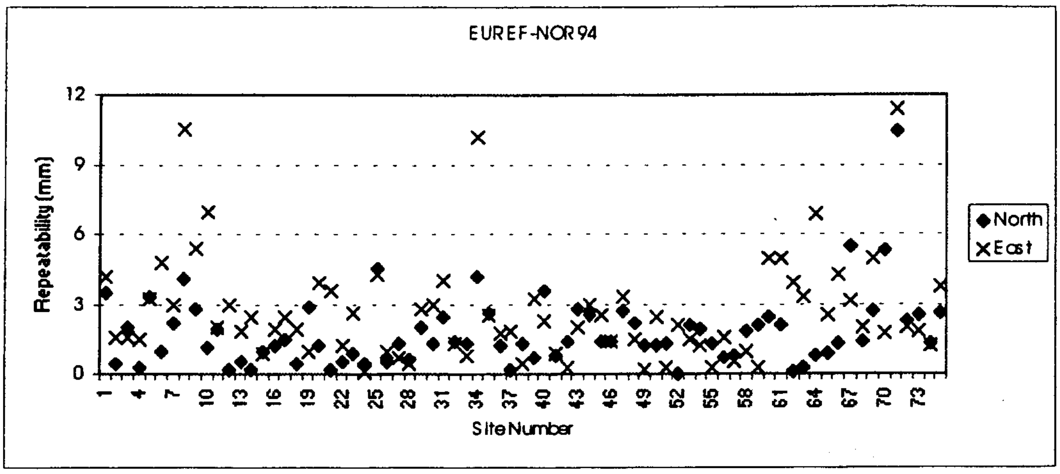


Figure 12a. The north and the east repeatabilities of the mobile GPS receivers in Norway from the EUREF-NOR94 campaign.

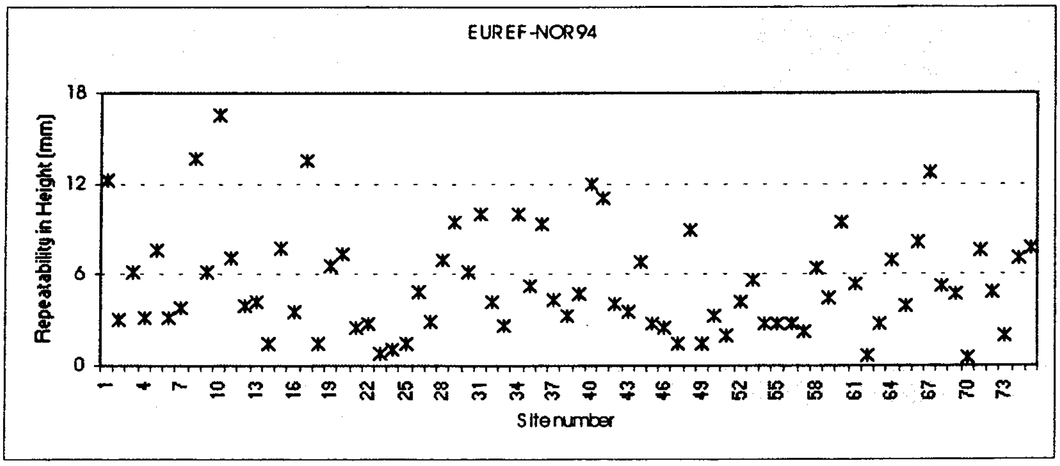


Figure 12b. The height repeatabilities of the mobile GPS receivers in Norway from the EUREF-NOR94 campaign.

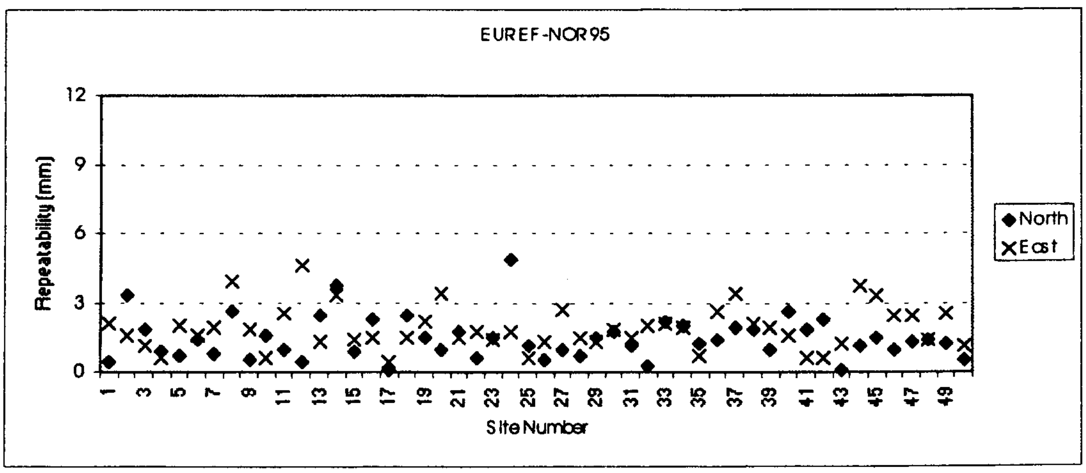


Figure 13a. The north and the east repeatabilities of the mobile GPS receivers in Norway from the EUREF-NOR95 campaign.

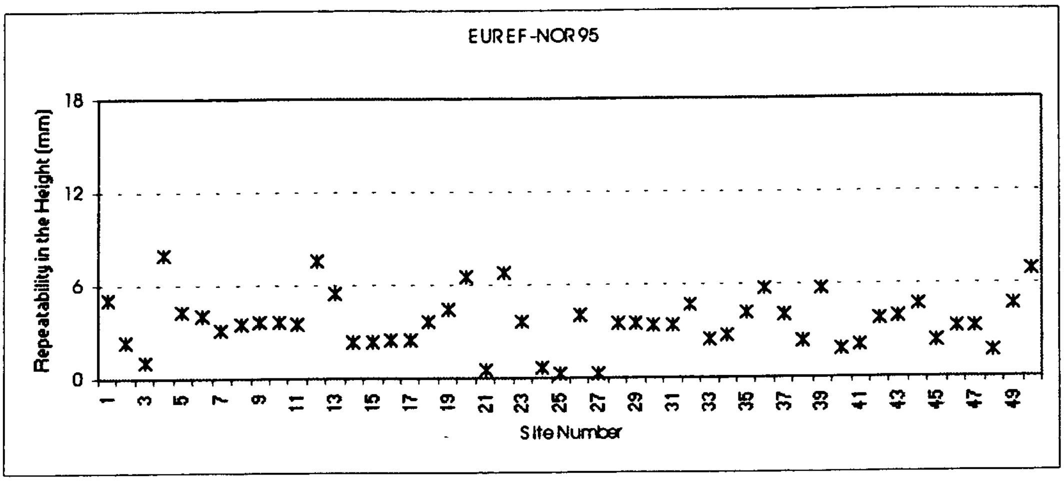
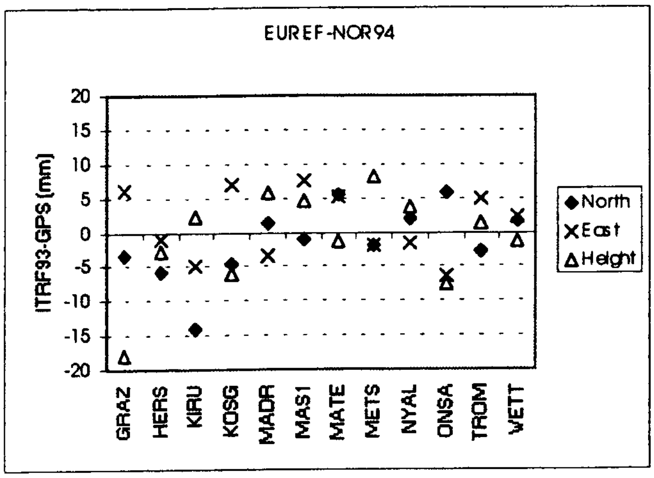


Figure 13b. The height repeatabilities of the mobile GPS receivers in Norway from the EUREF-NOR95 campaign.

Figure 12 and 13 show the repeatabilities of all the mobile GPS receivers from the EUREF-NOR94 and EUREF-NOR95 campaigns. The repeatabilities for most receivers in the northerly and the easterly directions are better than 6 mm in the 1994 campaign and better than 4 mm in the 1995 campaign. The repeatability from three sites shows values larger than 6 mm in the horizontal component, these are the SATREF stations Bergen (10.5 mm), Kristiansand (10.2 mm) and Vardø (11.4 mm). In the height they are better than 12 mm for most receiver in the 1994 campaign and better than 7 mm in the 1995 campaign. The mean repeatability in the northerly direction is 1.8 mm in 1994 and 1.4 mm in 1995 and in the easterly direction it is 2.7 mm and 1.9 mm, respectively. Furthermore the mean vertical repeatability is 5.4 mm and 3.6 mm in 1994 and 1995, respectively.

Figure 11a shows the estimated repeatabilities for the length component as a function of baseline length from the EUREF-NOR94 campaign. Each point in the plots represents one single baseline between the included stations. The overall repeatability is $2.0\,mm \pm 0.7\,ppb$. Figure 11b shows that the overall repeatability of the EUREF-NOR95 campaign is $1.3\,mm \pm 0.7\,ppb$. This demonstrates a significant improvement compared with the previous campaign. The scatter around the least square fitted strait line has improved, and the standard error of the predicted length repeatability has improved from 2.4 mm in 1994 to 1.4 mm in 1995. The reason for this improvement is a change in the analysis strategy [*Kristiansen, 1998*].

A remaining question to be answered was whether these estimates of precision were also a measure of the solution's accuracy. To answer the question, the solutions for the twelve IGS stations were compared with nominal ITRF93 coordinates obtained from the IERS. The uncertainty of the ITRF93 stations used is considered to be within 1 cm in all three axes. Figure 14 shows the agreements between the nominal ITRF93 coordinates and the computed GPS coordinates for the EUREF-NOR94 and the EUREF-NOR95 campaigns. All the computed coordinates for the IGS stations are within the assumed coordinate uncertainties, with the exception of the height component of Graz and the north component of Kiruna in both GPS campaigns and in addition the horizontal coordinates of Mas Palomas in the EUREF-NOR95 campaign. This demonstrates that the RMS values also give a good indication of the overall solution's quality.



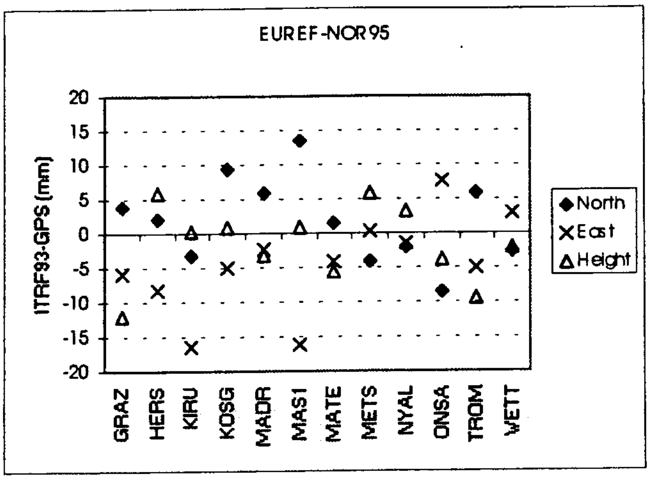


Figure 14. The agreements between the nominal ITRF93 coordinates and the GPS solution of the included IGS stations in the EUREF-NOR94 (left) and the EUREF-NOR95 (right) campaigns.

9 CONVERSION TO ETRS89

Transformation from the ITRF93 international reference frame to the European reference frame ETRF93 at epoch 1 January 1989 was done as described earlier using equation 3. The procedure was completed in two stages, where the first was from the epoch of observation to the EUREF epoch to correct for the tectonic plate movement in the intervening period. Then, since it was desired to retain the ITRF scale, the second stage was to apply an origin translation in order to reduce the results to the EUREF89 frame.

10 COMPUTING THE FINAL COORDINATES FOR ALL THE STAMNETT STATIONS

10.1 Test of quality in the network

The reductions of the observations from the two earlier campaigns (EUREF-NOR94 and 95) were both based on exactly the same selection of European permanent stations. During the reduction of the EUREF-NOR96 data using the GIPSY software as before, it transpired that some of these permanent stations, including Ny-Ålesund and Metsahovi, were unable to provide data covering the dates of the 1996 campaign. This resulted in a degree of uncertainty concerning the homogeneity of the 1996 campaign results with the work of the previous two years.

It was therefore decided that the results from EUREF-NOR96 could not be accepted in order to fix the final coordinates for these 25 last 3D stations in the same way as for the two earlier campaigns. Instead, it was considered more appropriate to compute these stations as if they were ordinary Stamnett Main stations, based on holding fixed the coordinates of the 3D stations in North Norway that had been observed during the EUREF-NOR94 and NOR95 campaigns.

Two separate sets of coordinates were thus produced for the stations included in the EUREF-NOR96 campaign, one using European permanent stations and the GIPSY software, the other using the coordinates for 3D stations from EUREF-NOR94 and NOR95 as fixed stations in

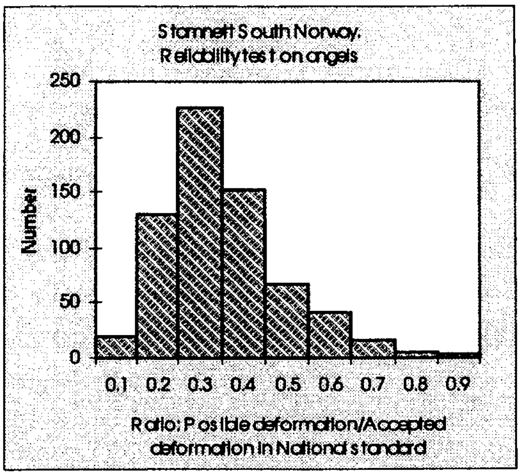


Figure 15a. Result of reliability test: Possible deformation on *angels*, South Norway.

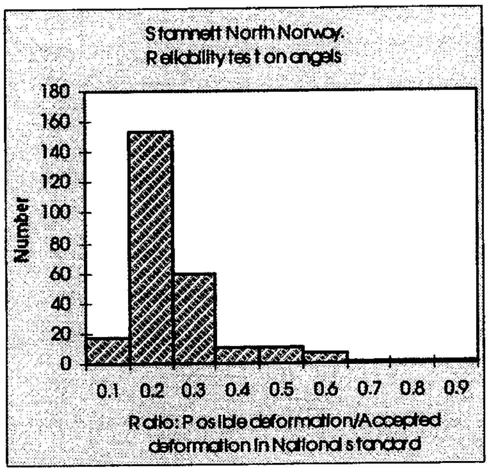


Figure 15b. Result of reliability test: Possible deformation on *angels*, North Norway.

The original objective for the Stamnett was that the average scale error between neighbouring stations should not exceed 1 ppm. The figures show that this objective has been achieved.

As the results demonstrate, the North Norway Block appears to be of slightly better quality than the Southern Block. Possible reasons are:

- Approximately 95% of the monuments in North Norway have screw bolt for force-centring of the antenna. This is more than in South Norway, where the figure is some 75 - 80%;
- Greater experience both in field observing and in reduction and computation;
- Better exploitation of the data, including more frequently using both diagonals in network quadrilaterals.

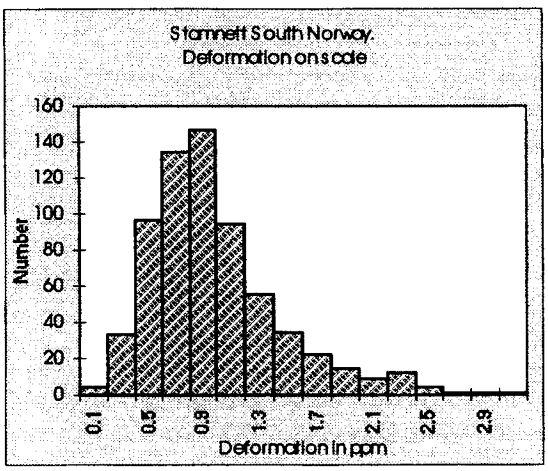


Figure 16a. Result of reliability test: Possible deformation on *scale*, South Norway.

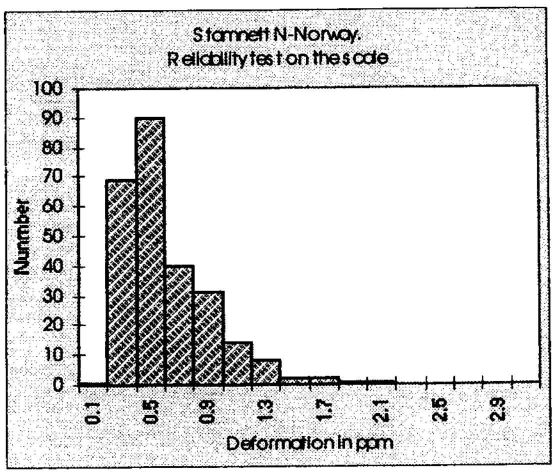


Figure 16b. Result of reliability test: Possible deformation on *scale*, North Norway.

11 CONCLUDING REMARKS

The work of establishing, observing and computing the new Norwegian National Geodetic Network, the Stamnett, is now considered completed. It took three years and resulted in a network consisting of 930 stations. Official $UTM_{(EUREF89)}$ coordinates have been published in County Lists dated 30 May 1997. It only remains to determine which of the UTM Zones 33 or 34 will be chosen for the area of Troms, which has got the coordinates computed in both Zone 33 and 34.

The budget for the Stamnett project was established in 1994 and is given in the table below in parenthesis. Actual costs up to 1996 are given above the budget figures in each line. All figures are in thousands of Norwegian Kroner. The original budget for the Stamnett project was in 1994 at 15.1 million Norwegian Kroner. Actual costs up to 1997 came to 15.8 million Norwegian Kroner as shown in the table.

The figures in Table 3, for the Norwegian Mapping Authority include both the Geodetic Institute's and the County Survey Offices' costs. Many Local Authorities contributed to the selection and monumentation of the Stamnett stations, and some also participated in the field GPS observations. The Norwegian Civil Aviation Administration also assisted with the observations in 1994.

Participant \ year	**1994**	**1995**	**1996**	**Total 1994-96**	**1997-**
Norwegian Mapping Authority	**1.544** (1.000)	**5.201** (5.500)	**5.976** (5.000)	**12.721** (11.500)	(3.900)
Cooperating Agencies =Roads Departments	**0.398** (400)	**1.847** (1.600)	**0.848** (1.600)	**3.093** (3.600)	
SUM (Budget)	**1.942** (1.400)	**7.048** (7.100)	**6.824** (6.600)	**15.814** (15.100)	(3.900)

<u>Table 3.</u> The cost and the budget of the Stamnett in millions of Norwegian Kroner.

In Norway EUREF89 is now in general use for land and sea mapping in the scale of 1: 50 000 and smaller. For the Norwegian Civil Aviation Administration all their airport installations have recently been measured and adjusted in EUREF89. Some of the municipalities have already started operating in the new geodetic datum.

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Appendix A. EUREF89 Coordinates for the Norwegian 3D stations

Sta. ID	Station name	X (m)	Sta dev (mm)	Y (m)	Sta dev (mm)	Z (m)	Sta dev (mm)
AK01	Frogn/Drøbak	3173770.4256	3.95	595855.3228	2.49	5481953.8080	6.34
AK02	Asker/Konglungen	3158610.4146	4.54	586273.0543	2.70	5491573.7252	7.42
AK03	Bærum/Fornebu	3151732.5004	3.45	591354.5109	2.16	5494974.8354	5.63
AK04	Skedsmo/Lillestrøm	3141294.5552	4.76	614061.7622	3.12	5498553.0711	7.89
AK05	Eidsvoll/Eidsvoll	3099618.6829	5.14	617374.7672	3.50	5521715.3832	8.55
AK06	Ås/NLH	3172302.7454	2.18	603839.3106	1.59	5481967.5048	3.47
BU01	Kongsberg/ Skollenborg	3187312.6166	4.42	544755.0613	2.90	5479521.3539	6.94
BU03	Øvre Eiker/Berg	3173493.0891	3.42	552660.8752	2.11	5486564.0180	5.57
BU04	Flesberg/Aslefet	3166703.8437	4.64	524374.5117	3.21	5493381.3259	7.33
BU05	Lier/Vefsrud	3148909.5496	3.44	574088.6293	2.18	5498631.1855	5.78
BU06	Modum/Vikersund	3151120.1950	4.31	555943.4520	2.68	5499048.9493	7.12
BU07	Sigdal/Tukudalen	3146682.6530	3.54	536793.3059	2.16	5503535.8667	5.84
BU08	Hønefoss/SATREF	3132539.0112	2.60	566401.7285	1.84	5508609.6576	4.22
BU09	Ringerike/Sokna	3126629.2545	3.46	549195.0877	2.09	5513697.5465	5.76
BU10	Nore og Uvdal/Dagali	3127907.9702	5.58	466746.9953	3.67	5521531.4435	9.18
BU11	Hol/Halne	3128277.7266	4.27	424193.1891	2.70	5524926.1992	7.16
BU12	Gol/Gol	3090990.7360	4.32	486285.2184	2.65	5539577.1759	7.38
FI01	Alta/Saga	2010883.7626	3.09	871740.8686	2.32	5969789.0904	7.87
FI02	Porsanger/Bergbukt	1977932.2429	3.76	922156.7892	3.26	5973108.7569	9.77
FI03	Sør-Varanger/Høybuktm.	1920597.0427	3.55	1105785.3851	3.00	5960811.1018	9.11
FI05	Gamvik/ Mehamn Lufth.	1838792.2287	3.76	970708.6952	3.42	6009521.5600	10.05
FIVA	Vardø/SATREF	1844607.6400	4.04	1109719.1102	4.18	5983936.0353	8.41
HE01	Trysil/Trysil	2988029.3195	3.34	655957.0301	2.13	5578669.0232	5.82
HO01	Etne/Lauvareid	3205325.8421	7.50	343742.1817	4.65	5485280.4949	11.91
HO03	Bergen/Flesland	3155043.7803	3.45	287797.6721	2.05	5517203.6822	5.70
HO04	Ullensvang/Bu	3129230.8293	3.94	375091.1562	2.44	5526674.6190	6.56
HOBE	Bergen/SATREF	3155871.4437	6.66	290902.7006	5.34	5516573.4035	11.14
MR01	Rauma/Hegerholmen	2919455.4740	4.25	392482.2474	2.72	5638243.0307	7.67
MR03	Sunndal/Svinberget	2899533.3704	4.05	433055.5197	2.95	5645763.6485	7.34
MR04	Molde/Molde Lufthavn	2905167.7206	6.25	369845.5963	4.33	5647091.6926	11.72
MR05	Rindal/Storslåttåsen	2856108.2143	3.28	471907.9851	2.09	5664889.8300	6.10
MRAL	Ålesund/SATREF	2938027.6027	5.61	319096.2143	5.26	5633413.7697	8.79
MRVI	Ålesund/Vigra	2931036.8127	3.61	312060.0205	2.50	5637267.9951	6.47
NO01	Bodø/Bodø Lufthavn	2395211.5089	2.83	611264.2186	2.00	5859950.9037	6.38
NO02	Fauske/Sjåheikulen	2382528.5931	3.87	657261.8024	3.09	5860248.7057	8.65
NO03	Tysfjord/Korsnesholmen	2277561.3493	4.14	655910.4150	3.62	5901596.5616	9.36
NO04	Hadsel/Børøya	2257596.5157	3.99	603053.2294	3.00	5914811.6178	9.49
NO05	Andøya	2175765.1308	2.72	624247.7872	2.46	5943414.6945	6.16
NOBO	Bodø/SATREF	2393811.9016	3.87	612747.6229	3.43	5860377.4801	7.74
NORE	Bodø/Renssåsen	2392456.5932	3.29	614434.8906	2.39	5860772.2880	7.12
NT01	Steinkjer/Elnan	2744242.1537	3.63	550212.9294	2.46	5712178.0173	7.02
NT02	Lierne/Nordli	2679611.7737	4.43	647590.5986	3.41	5732895.0361	8.97
NT03	Namsskogan/ Namsskogan	2638638.0341	4.07	616493.1499	2.78	5754941.5074	8.26
OE01	Rygge/Rygge	3199408.1075	4.68	611115.7433	3.29	5465433.7426	7.50
OP01	Lunner/Harestua	3122014.7854	3.50	589817.5628	2.13	5512228.3804	5.86
OP02	Øyer/Øyer	3030855.4965	3.84	557050.8556	2.52	5565813.2089	6.64
OP03	Lom/Sognefjell	3016021.3101	4.16	423675.9517	2.57	5586865.9042	7.25
OP04	Skjåk/Grotli	2974674.9588	4.14	401250.4533	2.56	5609877.2576	7.38
OP06	Lesja/Stuguflåten	2944197.2926	5.16	420762.0203	3.64	5623991.0246	9.21

Sta ID	Station name	X (m) Sta dev (mm)	Y (m) Sta dev (mm)	Z (m) Sta dev (mm)
OS01	Oslo/Ekeberg	3150510.9587 3.44	599711.2896 2.11	5494911.6668 5.65
RO01	Gjesdal/Ålgård	3297588.3668 3.71	339056.6854 2.23	5431012.6570 5.82
RO02	Sandnes/Lauvvik	3285064.9927 5.01	348831.8809 2.97	5437724.7453 7.97
RO03	Sola/Skadberg	3288229.0445 6.38	326151.9078 4.54	5437264.7572 9.38
RO04	Sandnes/Vier	3279044.0545 3.68	338684.3207 2.17	5441978.7474 5.82
RO09	Sola/Eigeberg	3291689.7777 3.70	321917.4610 2.41	5435482.3906 5.79
ROST	Stavanger/SATREF	3275772.8820 4.23	321117.4625 3.22	5445029.7359 6.48
SF01	Gulen/Sløvåg	3102550.0968 6.48	275733.5717 4.56	5547282.6797 11.15
SF02	Sogndal/Sogndal Lufth	3061523.8912 3.44	383415.1550 2.13	5564130.3111 5.90
SF03	Flora/Florø Lufthavn	3031274.7035 3.40	266626.1328 2.13	5586718.9689 5.96
SF05	Bremanger/Langesi	3005407.8667 4.86	286689.9821 3.23	5600018.8718 8.29
SP01	Ny-Ålesund/IGS	1202430.8186 1.00	252626.6390 1.00	6237767.4626 3.10
ST01	Mausundvær	2784636.6196 2.79	424346.7535 2.17	5703313.6018 5.07
TR02	Tromsø/Langnes	2102022.3737 4.13	719850.7445 3.15	5958615.0044 10.31
TROM	Tromsø/IGS	2102940.5251 1.08	721569.3509 0.86	5958192.0330 2.61
VA01	Farsund/Nordberg	3353280.3602 3.60	389105.2807 2.09	5393562.8088 5.52
VA02	Kristiansand/Kjevik	3335848.6487 3.77	473041.5077 2.26	5397525.7095 5.86
VA03	Sirdal/Tonstad	3300292.0854 4.15	390500.6200 2.41	5426065.0483 6.44
VAKR	Kristiansand/SATREF	3348186.1008 4.71	465040.8655 3.90	5390738.0808 7.15
VE01	Sandefjord/Torp	3224282.4278 6.48	583597.2830 5.02	5453904.7146 9.94
AA01	Arendal/Harebakken	3304373.0287 5.94	508264.2904 3.59	5413608.7142 8.89
AA02	Evje og Hornnes/Evje	3301789.3262 6.20	453546.5455 4.37	5420241.5004 9.78
AA03	Risør/Vinterkjær	3278078.2038 4.49	521843.8818 2.62	5428195.2409 7.09
AA04	Valle/ Valle	3243165.7226 4.86	429952.7183 3.19	5457302.6646 7.57

Table A.1. *Cartesian EUREF89 coordinates of the Norwegian 3D stations from the EUREF-NOR94 GPS campaign.*

ID	Station name	Latitude	Standard dev	Longitude	Standard dev	Ellipsoidal h	Sta dev
		Deg Min Sec	mm	Deg Min Sec	mm	meter	mm
AK01	Frogn/Drøbak	59 40 02.0197	0.60	10 37 59.3114	2.00	123.7354	4.80
AK02	Asker/Konglungen	59 50 23.6818	1.90	10 30 54.2429	1.10	41.5856	3.70
AK03	Bærum/Fornebu	59 54 01.7498	0.60	10 37 36.3424	1.50	57.2527	3.40
AK04	Skedsmo/Lillestrøm	59 57 46.8943	0.20	11 03 38.6191	2.10	157.9646	5.50
AK05	Eidsvoll/Eidsvoll	60 22 47.2489	0.30	11 15 52.6333	3.20	232.9816	1.90
AK06	Ås/NLH	59 40 01.1017	0.70	10 46 37.8200	0.80	156.2275	1.70
BU01	Kongsberg/ Skollenb.	59 37 22.1071	1.70	9 41 56.0852	0.30	203.0672	3.50
BU03	Øvre Eiker/Berg	59 44 59.3615	0.70	9 52 44.1581	0.50	87.6177	8.50
BU04	Flesberg/Aslefet	59 52 09.3874	1.40	9 24 08.2474	4.90	232.0873	9.00
BU05	Lier/Vefsrud	59 57 46.3846	2.90	10 19 56.3302	1.50	257.3173	4.20
BU06	Modum/Vikersund	59 58 21.8873	1.90	10 00 20.0569	1.70	104.6751	10.00
BU07	Sigdal/Tukudalen	60 03 08.3333	2.30	9 40 51.2750	1.40	169.3855	3.50
BU08	Hønefoss/SATREF	60 08 36.7353	2.00	10 14 56.5823	2.90	177.4140	9.30
BU09	Ringerike/Sokna	60 14 06.2132	2.30	9 57 44.7278	1.20	198.7121	7.70
BU10	Nore og Uvdal/Dagali	60 21 47.1352	0.30	8 29 13.4286	4.10	1079.6187	5.70
BU11	Hol/Halne	60 25 19.5379	0.60	7 43 19.8638	1.30	1247.9464	7.00
BU12	Gol/Gol	60 42 17.0641	1.20	8 56 26.4686	3.70	296.7437	1.10
FI01	Alta/Saga	69 57 51.5530	0.40	23 26 14.2001	1.80	118.8503	3.40
FI02	Porsanger/Bergbukt	70 03 12.3951	1.40	24 59 45.5951	6.70	34.2098	4.40
FI03	Sør-Varanger/Høyb.	69 43 50.4079	1.80	29 55 52.4011	4.20	121.5081	8.60
FI05	Gamvik/ Mehavn	71 01 58.9949	4.40	27 49 47.3654	2.80	42.4052	15.20
FIVA	Vardø/SATREF	70 20 10.9342	9.90	31 1 52.2757	12.50	174.8565	7.60
HE01	Trysil/VLBI-punkt	61 25 22.1988	1.70	12 22 53.8963	2.50	723.9131	6.50
HO01	Etne/Lauvareid	59 43 30.1256	1.30	6 07 15.8157	1.50	212.9243	11.10
HO03	Bergen/Flesland	60 18 00.6509	2.70	5 12 43.1947	3.70	92.3309	4.70
HO04	Ullensvang/Bu	60 28 15.6659	2.00	6 50 6.9170	2.60	166.6909	8.70
HOBE	Bergen/SATREF	60 17 19.4712	3.80	5 15 59.5509	11.30	93.8143	15.50
MR01	Rauma/Hegerholmen	62 34 21.6330	1.90	7 39 24.3141	2.00	51.8065	3.50
MR03	Sunndal/Svinberget	62 42 55.5916	0.40	8 29 40.3323	3.80	286.5827	3.10
MR04	Molde/Molde	62 44 43.6475	1.10	7 15 18.2761	1.20	56.1679	15.10
MR05	Rindal/Storslåttåsen	63 05 16.8094	2.60	9 22 55.4919	1.80	521.7393	11.00
MRAL	Ålesund/SATREF	62 28 34.9710	2.40	6 11 54.7459	5.00	189.8325	12.80
MRVI	Ålesund/Vigra	62 33 13.1492	2.90	6 04 38.0717	2.20	53.9610	2.50
NO01	Bodø/Bodø Lufthavn	67 15 55.3931	2.70	14 18 59.2237	5.50	39.3085	6.30
NO02	Fauske/Sjåheikulen	67 16 12.6373	3.50	15 25 20.7892	1.10	138.3692	7.90
NO03	Tysfjord/ Korsnesholm.	68 15 7.1543	2.90	16 3 56.7628	5.70	34.6607	5.80
NO04	Hadsel/Børøya	68 34 25.6755	0.10	14 57 20.7360	5.40	43.8800	0.80
NO05	Andøya	69 16 42.1343	2.60	16 00 31.2867	1.70	410.5787	8.10
NOBO	Bodø/SATREF	67 16 30.1491	1.20	14 21 28.0945	7.60	50.7416	17.10
NORE	Bodø/Rensåsen	67 17 01.6833	3.10	14 24 12.6533	4.00	69.6979	1.70
NT01	Steinkjer/Elnan	64 02 52.0235	2.60	11 20 14.3624	6.10	144.4965	6.70
NT02	Lierne/Nordli	64 28 7.1794	3.90	13 35 10.8595	2.80	519.6822	5.10
NT03	Namsskogan/ Namssk.	64 56 04.9598	1.90	13 09 02.5952	0.50	344.1584	5.80
OE01	Rygge/Rygge	59 22 30.6569	1.50	10 48 49.5141	2.80	103.5627	4.10
OP01	Lunner/Harestua	60 12 25.6590	1.70	10 41 54.0512	0.90	286.8543	3.50
OP02	Øyer/Øyer	61 11 26.9657	1.60	10 24 51.6653	1.60	220.0084	2.20
OP03	Lom/Sognefjell	61 33 53.1216	1.00	7 59 46.7075	2.20	1460.8278	4.00
OP04	Skjåk/Grotli	62 00 37.2847	0.50	7 40 55.8936	0.10	928.8928	1.50
OP06	Lesja/Stugufläten	62 17 15.6718	1.70	8 07 59.5188	2.40	558.7947	13.10
OS01	Oslo/Ekeberg	59 53 50.8976	1.40	10 46 39.0828	2.30	178.8705	4.20

RO01	Gjesdal/Ålgård	58 46 18.7635	1.50	5 52 13.7971	0.40	242.6423	2.50
RO02	Sandnes/Lauvvik	58 53 27.3620	0.10	6 03 40.9136	2.90	65.1931	5.70
RO03	Sola/Skadberg	58 52 56.6942	3.10	5 39 52.2340	0.50	100.6604	6.40
RO04	Sandnes/Vier	58 57 53.3321	2.50	5 53 49.2998	1.80	72.6604	6.90
RO09	Sola/Eigeberg	58 51 03.1218	1.20	5 35 08.1402	1.10	141.1128	9.30
ROST	Stavanger/SATREF	59 01 03.0098	2.70	5 35 55.3301	4.90	104.5196	9.60
SF01	Gulen/Sløvåg	60 50 59.8365	4.50	5 04 43.3787	3.20	74.2639	1.60
SF02	Sogndal/ Sogndal	61 09 12.7014	1.00	7 08 18.1296	0.60	586.5640	2.70
SF03	Flora/Florø Lufthavn	61 35 06.3045	0.30	5 01 36.1542	4.40	67.4297	3.10
SF05	Bremanger/Langesi	61 49 50.5238	1.00	5 26 56.5264	7.90	437.7948	9.50
SP01	Ny-Ålesund/IGS	78 55 46.4959	2.10	11 51 54.2901	2.50	78.4130	7.80
ST01	Mausundvær	63 52 6.1994	0.80	8 39 52.4203	3.90	50.5539	5.10
TR02	Tromsø/Langnes	69 40 33.7724	7.60	18 54 14.7225	4.00	33.6713	17.70
TROM	Tromsø/IGS	69 39 45.8853	4.40	18 56 17.9681	2.30	132.4342	7.70
VA01	Farsund/Nordberg	58 07 50.7103	1.50	6 37 7.8286	0.50	143.5592	4.00
VA02	Kristiansand/Kjevik	58 11 58.3647	1.70	8 04 15.7479	1.70	50.8584	5.00
VA03	Sirdal/Tonstad	58 41 05.3670	1.00	6 44 52.9310	2.90	343.6366	4.60
VAKR	Kristiansand/SATREF	58 04 57.6917	4.40	7 54 26.6934	10.50	147.7089	10.40
VE01	Sandefjord/Torp	59 10 20.7485	1.90	10 15 34.1961	5.80	116.8952	11.20
AA01	Arendal/Harebakken	58 28 26.1310	0.50	8 44 40.0380	2.10	98.2933	4.30
AA02	Evje og Hornnes/Evje	58 35 5.8144	0.60	7 49 17.1162	2.60	305.2404	2.10
AA03	Risør/Vinterkjær	58 43 30.4679	1.70	9 02 42.4780	0.20	107.2162	2.10
AA04	Valle/ Valle	59 13 39.3998	6.80	7 33 6.3920	2.20	408.2432	0.60

Table A.2. *Geodetic EUREF89 coordinates of the Norwegian 3D stations from the EUREF-NOR94 GPS campaign. GRS80 ellipsoid is used.*

Sta ID	X (m)	St dev (mm)	Y (m)	St dev (mm)	Z (m)	St dev (mm)
AA05	3281097.7601	1.25	488818.8489	.89	5429535.0978	1.96
HE01	2988029.3285	.91	655957.0295	.72	5578669.0333	1.51
HE02	3108471.0322	1.25	661270.4792	.94	5511756.1568	2.05
HE03	3069510.9315	1.19	652308.6032	.94	5534424.5574	1.99
HE04	3062695.4143	1.42	599495.5542	1.05	5544011.2643	2.42
HE05	3048372.0440	1.32	624628.3401	.94	5549217.1336	2.24
HE06	2988391.7737	2.40	583378.9968	1.75	5586113.0941	4.19
HE07	2952669.1518	1.50	620841.1914	1.17	5601588.5561	2.58
HE08	2941522.9700	2.50	554261.5073	2.15	5613899.9198	4.36
HO02	3206758.3148	2.49	303778.9195	1.77	5486623.6539	3.97
HO06	3116482.3520	2.52	350935.2594	1.93	5535349.1628	4.27
MR06	2967569.6706	2.24	315533.2638	1.64	5618183.4996	3.94
MR07	2933721.4380	2.53	351140.6722	2.11	5633628.3322	4.46
MR08	2882134.2599	2.65	413950.9540	2.14	5655885.3707	4.77
MR09	2865159.6956	1.26	391710.1904	.93	5665960.7512	2.24
NO12	2428747.2277	2.34	670337.5088	2.30	5839982.7222	4.98
NO13	2444606.2669	2.09	598587.4994	1.55	5840988.0053	4.35
NO16	2327352.6074	1.73	664908.4740	1.55	5881668.5350	3.82
NO17	2336651.2439	2.04	626667.8142	1.60	5881804.2635	4.44
NT04	2807246.5163	1.84	541526.1185	1.32	5682404.0838	3.38
NT05	2695637.7756	1.19	587198.8146	.94	5731536.9087	2.30
NT06	2701427.0316	1.42	551988.3477	1.06	5732130.9139	2.71
NT07	2667760.6098	1.78	525605.4680	1.32	5750241.8668	3.40
NT08	2646221.2906	2.27	571238.8942	2.07	5755786.7870	4.43
OE02	3220124.1395	2.12	642081.8618	1.41	5449858.1900	3.31
OE03	3176052.4219	1.40	637218.5772	1.05	5476179.4356	2.22
OP05	2954720.9997	2.78	479775.9658	1.98	5614303.4689	4.85
OP07	3082182.9512	1.12	576724.1451	.79	5536010.9325	1.89
OP08	3059018.9190	.89	500520.4576	.64	5556588.1691	1.50
OP09	3037060.0053	1.07	471822.0047	.79	5571394.7969	1.79
OP10	2998543.3849	3.46	532521.0114	3.04	5585869.2842	6.14
OP11	2983891.4340	3.92	501190.4824	2.97	5596423.2122	6.44
OP12	2983499.1886	1.64	449804.3377	1.47	5601015.9293	2.89
RO06	3328693.4922	2.33	350014.7364	1.61	5411300.7018	3.56
RO07	3246769.8980	2.31	296684.7284	1.60	5463592.7753	3.55
RO08	3227135.3484	2.36	353621.3400	1.61	5471875.2635	3.73
SF04	3000325.8838	1.28	319543.4474	.92	5600518.3965	2.22
SF06	3056768.8309	1.05	430008.7744	.77	5563783.3041	1.77
SF07	3039669.5193	1.33	312529.3458	.97	5579853.8510	2.26
ST02	2886591.4115	1.85	580292.0593	1.37	5639751.4802	3.30
ST03	2875769.9701	2.26	506403.2920	1.73	5652007.6769	4.06
ST04	2835698.8542	.94	487896.9555	.71	5673368.6744	1.69
ST06	2817277.7518	.92	454318.3795	.71	5685095.3519	1.68
ST07	2791829.9440	1.40	475919.0886	1.10	5695871.6376	2.56
ST08	2727006.3541	1.42	505993.8317	1.10	5724330.4679	2.64
TE01	3228310.9678	2.51	545142.8033	1.77	5455430.3689	3.94
TE02	3230138.6555	2.41	484264.8920	1.70	5460332.1099	3.82
TE03	3207524.5451	1.31	450404.8412	.99	5476668.3517	2.08
TE04	3189685.7673	1.62	403407.7144	1.22	5491274.9173	2.62
VA04	3334217.7316	1.90	408007.0799	1.35	5404078.1772	2.94

Table A.3. *Cartesian EUREF89 coordinates of the Norwegian 3D stations from the EUREF-NOR95 GPS campaign.*

Sta ID	Latitude Deg	Min	Second	Sta dev mm	Longitude Deg	Min	Second	Sta dev mm	Height m	Sta dev mm
AA05	58	44	49.4310	.48	8	28	25.0206	2.55	191.2394	6.28
HE01	61	25	22.1987	1.87	12	22	53.8961	.91	723.9262	1.57
HE02	60	11	55.9750	2.19	12	0	34.5717	1.28	268.7810	1.00
HE03	60	36	40.6598	1.14	11	59	51.2505	.77	239.0539	9.62
HE04	60	47	17.9809	.81	11	4	30.4070	2.43	162.0921	5.36
HE05	60	52	56.8899	1.74	11	34	47.7574	1.87	269.4976	4.81
HE06	61	34	5.1859	1.02	11	2	45.8795	2.27	402.5119	3.78
HE07	61	51	6.1967	1.98	11	52	27.4339	3.78	964.1643	3.90
HE08	62	5	35.8109	.77	10	40	15.3846	2.46	578.9560	5.27
HO02	59	45	4.2834	1.91	5	24	41.5026	.73	67.8263	4.47
HO06	60	37	48.2006	1.22	6	25	29.2651	3.05	123.1812	4.28
MR06	62	10	57.1600	.53	6	4	9.4590	5.78	165.0210	8.26
MR07	62	28	56.6271	3.00	6	49	31.1939	1.58	82.4411	6.81
MR08	62	54	59.1021	5.20	8	10	23.8760	4.61	160.3581	3.41
MR09	63	6	59.4320	1.00	7	47	5.7369	1.55	110.6613	2.85
NO12	66	48	9.8314	2.79	15	25	46.4765	1.82	217.7252	3.06
NO13	66	49	43.6719	.11	13	45	31.5359	.53	66.3230	3.05
NO16	67	45	59.6977	2.73	15	56	39.3764	1.67	407.4686	4.49
NO17	67	46	38.5307	1.87	15	0	46.4485	2.71	62.4139	5.42
NT04	63	26	44.7520	1.17	10	55	6.2851	4.16	45.7020	7.83
NT05	64	26	47.1431	2.27	12	17	20.1688	1.94	198.9741	.70
NT06	64	27	42.2435	.77	11	32	54.2354	1.97	41.7630	8.41
NT07	64	50	27.3350	1.81	11	8	44.6352	1.45	54.8079	3.77
NT08	64	57	30.3525	3.67	12	10	53.3930	1.37	40.9863	.50
OE02	59	6	3.8662	1.61	11	16	36.0733	.86	153.3296	.49
OE03	59	33	45.4072	.35	11	20	41.1389	1.74	263.8171	5.83
OP05	62	5	38.1067	1.47	9	13	22.6840	3.77	997.9608	.35
OP07	60	38	12.8506	.84	10	35	54.1345	1.73	453.2001	4.44
OP08	61	0	31.7623	1.79	9	17	32.8721	1.55	879.1634	4.03
OP09	61	16	43.9452	2.17	8	49	50.1110	1.90	1200.4911	3.73
OP10	61	33	42.6647	1.59	10	4	13.2525	2.08	502.7764	4.74
OP11	61	45	46.6369	.40	9	32	4.9454	2.28	406.7168	5.89
OP12	61	50	57.9785	2.21	8	34	24.8620	2.66	450.8590	3.16
RO06	58	26	2.1354	2.40	6	0	9.4968	2.39	125.2394	3.36
RO07	59	20	35.6078	1.32	5	13	15.9542	.78	72.7790	4.97
RO08	59	29	16.9442	1.49	6	15	12.1603	3.20	158.5841	6.73
SF04	61	50	48.1678	2.02	6	4	45.3042	3.87	49.0937	4.06
SF06	61	8	26.9200	1.66	8	0	26.9928	1.45	971.2204	1.76
SF07	61	27	19.4821	1.23	5	52	13.2446	2.41	96.6000	6.72
ST02	62	35	23.5614	3.25	11	22	.0382	2.08	756.4663	2.42
ST03	62	50	3.2199	2.18	9	59	13.2144	.74	496.5256	2.55
ST04	63	15	43.7646	2.53	9	45	44.8468	.75	206.1575	4.86
ST06	63	29	57.4624	.18	9	9	38.6509	1.36	75.0006	4.69
ST07	63	42	56.1726	1.41	9	40	26.8785	4.29	139.5305	5.48
ST08	64	17	58.5009	1.53	10	30	41.9053	4.02	59.4694	2.71
TE01	59	11	59.7697	1.23	9	35	5.0175	3.01	65.7117	3.90
TE02	59	16	56.7452	1.56	8	31	34.7744	2.91	300.5104	4.01
TE03	59	34	3.2077	1.69	7	59	35.8261	1.35	507.2185	1.69
TE04	59	49	8.0047	1.29	7	12	29.0682	2.38	1057.5323	4.53
VA04	58	18	28.2916	.58	6	58	35.6867	1.38	291.5082	8.65

Table A.4. *Geodetic EUREF89 coordinates of the Norwegian 3D stations from the EUREF-NOR95 GPS campaign. GRS80 ellipsoid is used.*

Appendix C. GPS Antenna information

Stations	GPS receiver	Antenna type	Distance ARP - TCR (m)
SATREF	ROGUE SNR-8000	Dorne-Margolin T	0.102
SWEPOS	ROGUE SNR-8000	Dorne-Margolin T	0.102
Mobile	ROGUE SNR-8000	Dorne-Margolin T	0.102

Table C.1. GPS antennas used and offset from the Antenna Reference Point (ARP) to the Top of the Choke Ring (TCR).

Station	IERS no.	GPS receiver	Antenna type	Distance marker to ARP (m)	Distance ARP-TCR (m)
Ny-Ålesund	10317M001	SNR-8	DM B	5.216	0.070
Tromsø	10302M003	SNR-8	DM B	2.473	0.070
Metshovi	10503S011	SNR-8C	DM B	0.000	0.070
Onsala	10402M004	SNR-8000	DM B	0.995	0.070
Graz	11001M002	SNR-8C	DM B	2.086	0.070
Wettzell	14201M009	SNR-800	DM B	0.000	0.070
Madrid	13407S012	SNR-8	DM R	0.000	0.070
Mas Palomas	31303M002	SNR-8100	DM T	0.033	0.102
Herstmonceux	13212M007	SNR-8C	DM B	0.200	0.070
Matera	12734M008	SNR-8	DM B	0.135	0.070
Kootwijk	13504M003	SNR-8000	DM B	0.105	0.070
Kiruna	10403M002	SNR-8100	DM T	0.062	0.102

Table C.2. GPSreceivers, antenna type and IERS number for the IGS stations used. ARP means Antenna Reference Point and TCR means Top of Choke Ring. DM is Dorne & Margolin.

Antenna type	Phase	North	East	Height
Dorne & Margolin B and R	L1	0.0000	0.0000	0.0079
Dorne & Margolin B and R	L2	0.0000	0.0000	0.0264
Dorne & Margolin B and R	LC	0.0000	0.0000	-0.0207
Dorne & Margolin T	L1	-0.0004	0.0022	0.0166
Dorne & Margolin T	L2	0.0004	0.0015	0.0433
Dorne & Margolin T	LC	-0.0016	0.0033	-0.0247

Table C.3. Distance from TCR (Top of Choke Ring) to the phase center of the Dorne & Margolin antenna in meter. L_C is a linear combination between the two frequencies L_1 and L_2.

$$L_c = L_1 + 1.546 \cdot (L_1 - L_2).$$

Densification of the EUREF89 network in Finland

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During the years 1996 and 1997, GPS campaigns called EUREF-FIN were performed, where the FGI measured a network of 100 points over Finland. The permanent network was a part of this campaign.

The main part of the observation sites were first order triangulation points, however, in order to connect the GPS network to the national height system (N60), some tide-gauge sites as well as Precise Levelling benchmarks were included in the network.

The GPS observations were carried out using six field receivers. The 12 stations of the Finnish permanent GPS-network (FinnRef) served to define a reference frame throughout the campaigns. In the final computation the reference frame was transformed to coincide the original EUREF89 frame.

The GPS solution of the network was performed using the Bernese v. 4.0 software. The RMS values computed according to the discrepancies between different sessions show that the accuracy of the coordinates is better than ± 20 mm.

1. Introduction

1.1 The Finnish permanent GPS network, FinnRef

The Finnish Geodetic Institute has established a permanent GPS network called FinnRef to Finland (Fig. 1-1.). FinnRef is part of a Nordic GPS network together with Norwegian and Swedish networks. Four stations (Metsähovi, Vaasa, Joensuu, and Sodankylä) belongs to the European-wide permanent EUREF network. The main objectives for FinnRef are to connect and compare the local reference frames to international ones and to study the land uplift and other local movements. (KOIVULA *et al.*, 1997, 1998, JOHANSSON *et al.*, 1997).

1.2 Reconnaissance of the densification sites

Finland is a forest covered country. This was confirmed during the reconnaissance of the GPS network. The aim was to select the first order triangulation points so that they form a uniform network over the whole country. In some areas this aim could not be fulfilled partly because of the polygonal structure of the first order triangulation network in the southern part of the country, partly because a large portion of the triangulation points is not suitable for GPS sites due to the thick forest.

The reconnaissance was needed only in Southern and Middle Finland because in Northern Finland most of the hills are bare. In such areas the network planning could be done in the office by the aid of maps only. The reconnaissance was made in 1996-97 before the GPS observation campaigns. The observers visited in two years altogether 163 triangulation stations and 27 precise levelling benchmarks. 79 of these stations were suitable for GPS observations.

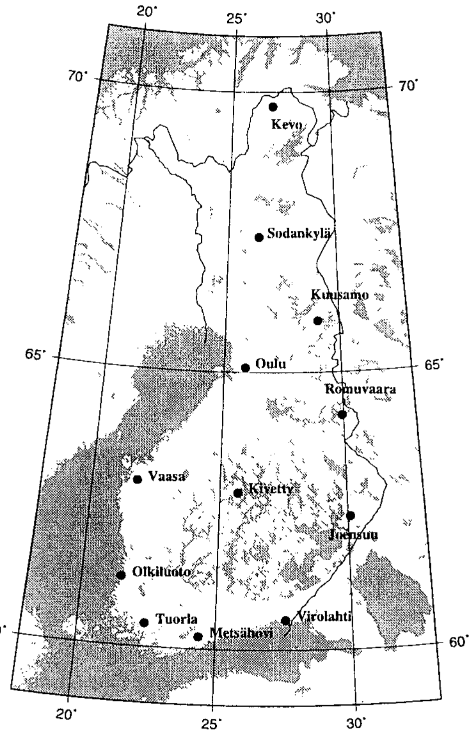


Fig 1-1. The Finnish permanent GPS network FinnRef. All 12 stations are running at the moment. Metsähovi is a part of the global IGS network and together with Vaasa, Joensuu and Sodankylä a part of EUVN network which is used for determining the unified European height system. These four stations are also included to the permanent EUREF network.

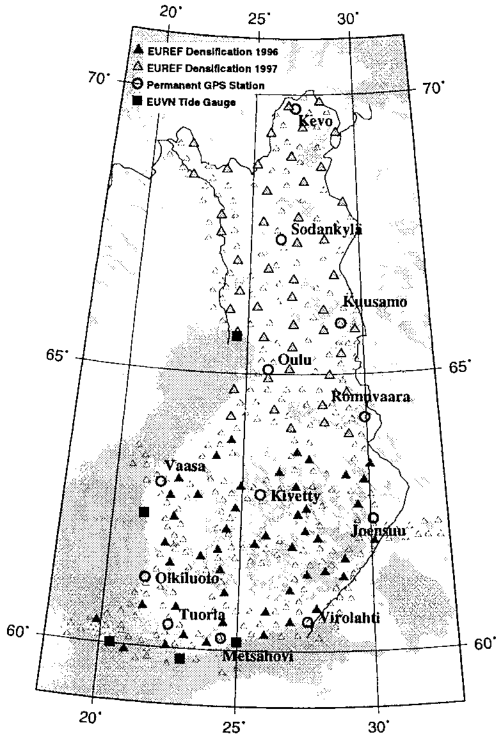


Fig. 1-2. Densification points.

1.3 Description of the densification stations

Because one aim of the densification of the EUREF89 network was to improve the connection between the EUREF reference frame with the old geodetic system used in Finland, the main part of the GPS sites were first order triangulation points. The orthometric height of the triangulation points has been determined by spirit levelling at a part of the stations only. At these stations the error in orthometric height is less than 0.1 m (KORHONEN 1966). The orthometric heights for the remaining points were based on trigonometric or barometric determinations. According to the classification made by KORHONEN (1966) the height error at these stations may amount to as much as 2 metres. In order to improve the connection between the GPS network and the national orthometric height system, N60, some tide cauge stations and precise levelling benchmarks were also included.

Due to the surrounding trees at many triangulation points, the GPS observations were not possible to carry out on the centric benchmark. At such places a reserve marker was used for the GPS observations, or at some sites a new marker was established for the GPS observat-ions. The centring elements between the triangulation station and GPS benchmark are based either on the determinations by Mr. Jorma Jokela or on older determinations which were obtained from the publications of the FGI. The computation of the centring elements is explained in the next chapter. The entire network is illustrated in Fig 1-2.

2 Observations

The GPS observations were performed in two years, devised in each year in two periods. The dates of the periods were: 1996 Aug. 5.-Aug. 22. (DOYs: 218-235), 1996 Sept. 2.- Sept. 13. (DOYs: 246-257), 1997 Aug. 7.-Aug. 21. (DOYs: 219-233) and 1997 Sept. 10.- Sept. 27. (DOYs: 253-270). The GPS sites observed in each period are shown in Figs. 2-1a...2-1d.

2.1 Personnel

In each year 13 persons took part in the GPS observations. Normally the whole observing team contained 12 persons divided in 6 groups containing 2 persons, but during the observations of the northernmost part of the network, the whole observing team contained only 5 persons. In this part of the network the transportation was made by the aid of a helicopter. At that time the measurements were done by two persons in turn, who were flying with the pilot from station to station, while the others were taking care of the data transferring, copying, battery loading and transporting the cars from village to the next.

2.2 GPS receivers

The number of the GPS receivers used for the observations was 7 in each year. All receivers were Ashtech Z-12 type, which is 12-channel, dual-frequency, carrier-phase GPS receiver. At most time, however, only 6 receivers were used simultaneously, the seventh receiver served as a spare one. All antennas used for the observations were Dorne Margolin Ashtech choke ring antennas. Because the FGI owned in 1996 only 4 that type of antenna two antennas were loaned from *Estonian Land Survey*. In 1997 the FGI purchased 3 new Ashtech Z-12 receivers and Dorne Margolin choke ring antennas. Since then it should have been possible to use all seven receivers simultaneously, but due to malfunctioning of one receiver that happened only during the second period in 1997. One of the receivers, used in 1996, was loaned from the *Helsinki University of Technology*, all others are owned by the FGI. The antennas were mounted on the benchmarks using tripods and tribrachs equipped with optical plummets.

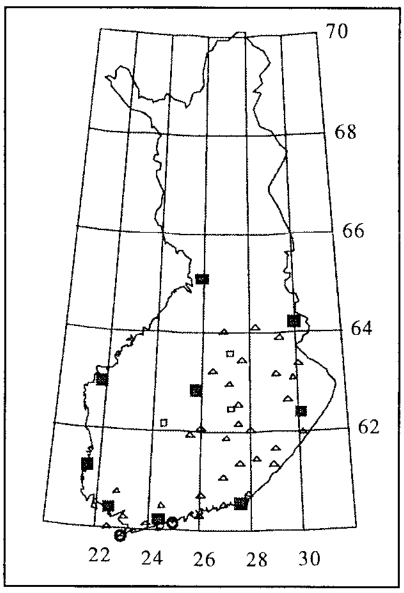


Fig. 2-1a. The GPS sites measured during the first period in 1996.

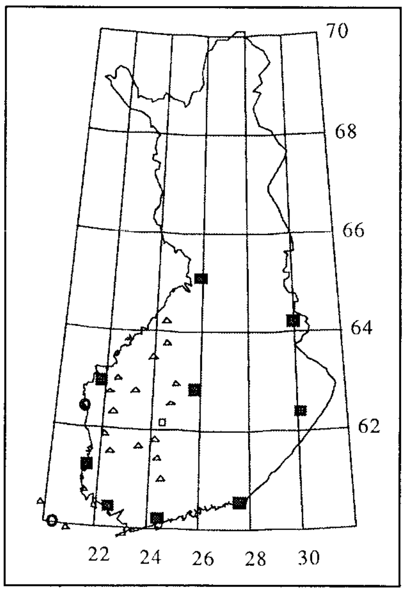


Fig. 2-1b. The GPS sites measured during the second period in 1996.

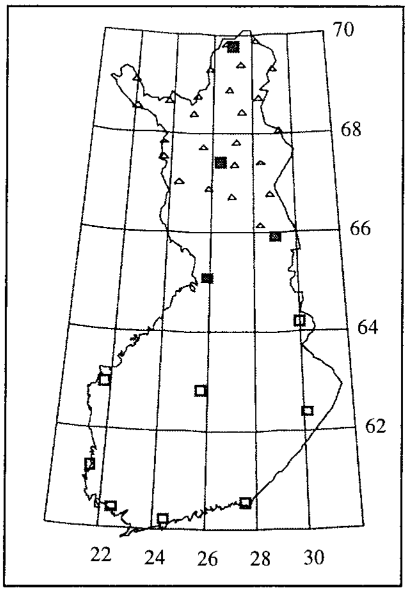


Fig. 2-1c. The GPS sites measured during the first period in 1997.

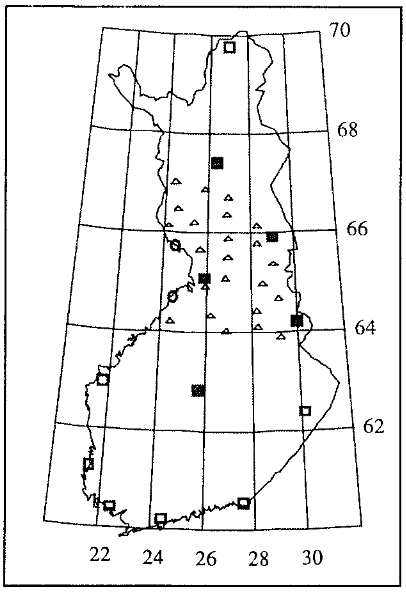


Fig. 2-1d. The GPS sites measured during the second period in 1997.

2.3 Organisation of the observations

Each observing group was moving individually from station to station according to the schedule. The observation time at each station was planned to be at least 48 hours. At 5 stations the nominal observation time could not be reached due to malfunctioning of the receivers or power loss of batteries. However, the shortest observation time was 36 hours. Although all stations are connected to each other through the observations at the permanent GPS stations we decided to strengthen the connection between the stations included in consecutive sessions by keeping one common station in both sessions. That is why the observation time at the junction stations was prolonged to 4-5 days. The pieces of the network observed in consecutive years were connected to each other by occupying 4 junction stations in each year.

The observations were started at each station either in the morning or in the afternoon depending on the time when the observations were stopped at the previous station. At the first stations of the observation period the observations were started in the afternoon and continued 48 hours. The next session was then started in the next morning in order to ensure the necessary travelling time for the expedition to a new configuration. The next move of the expedition was performed in daytime and groups were ready to start the observations in the next afternoon at new sites. During the computation the observations were divided in 24 hours' sessions starting in midnight. Thus 48 hours observations at a station were divided in three sessions: the lengths of the sessions were 8-14 hours, 24 hours and 8-14 hour in the first, in the second and in the third session, respectively. The statistics of the observation sessions is shown in Fig. 2-2. The number of the stations which has more than 5 observation sessions is 33, which is 30% of all stations. Fig. 2-3. presents how the observation sessions at the junction stations are related to each other.

The recording interval was throughout the campaign 30s. According to the normal routine the observations were performed continuously except the brakes of some minutes needed for downloading the data from the receiver to PC and changing the batteries. The height of the antenna was measured in the beginning, and at the end of the observations at each station. Besides, the antenna stability was checked once in the intermediate day of the observations, but otherwise it was not touched. The data collected from the stations was transformed to RINEX format and saved on diskettes both in original Ashtech format and in RINEX format.

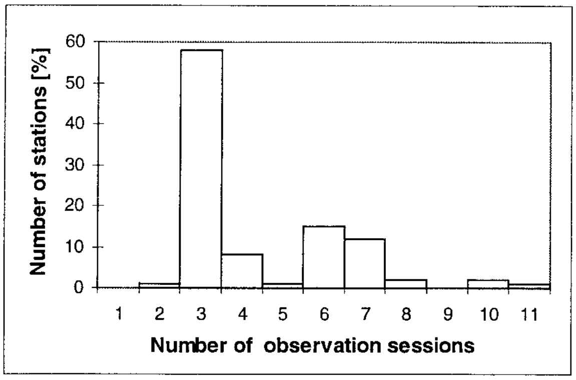


Fig. 2-2. Number of observation sessions.

															1	9	9	6																																				1	9	9	7														
Station	Nbr. of										1	1	1	1	1	1	1	1	1	1	2	2	2	2	2	2	2	2	2	2	3	3	3	3	3	3	3	3	3	3	4	4	4	4	4	4	4	4	4	4	5	5	5	5	5	5	5	5	5	5	6	6	6	6	6	6	6	6
	Session	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7	8	9	0	1	2	3	4	5	6	7
ROKO	6	+	X	X	X	X	+																																																														
HEVO	6				+	X	X	X	X	+																																																									
SAUK	6							+	X	X	X	X	+																																																						
TAHK	6										+	X	X	X	X	+																																																			
VUOK	6													+	X	+																																		+	X	+															
KALL	11													+	X	+																																		+	X	X	+												+	X	X	+
PIHJ	7													+	X	+																																		+	X	X	+														
6802	6													+	X	X	X	X	+																																																
KIAP	6																+	X	X	X	X	+																																													
1029	6																			+	X	X	X	X	+																																										
ISOL	6																						+	X	X	X	X	+																																							
NISU	7																									+	X	+																																				+	X	X	+
PETT	6																									+	X	X	X	X	+																																				
SAMM	6																												+	X	X	X	X	+																																	
OTSA	6																																		+	X	X	X	X	+																											
KAUN	5																																			+	X	X	X	+																											
KORS	6																																					+	X	X	X	X	+																								
KANG	6																																								+	X	X	X	X	+																					
AALI	7																																											+	X	+												+	X	X	+					
KIRI	10																																											+	X	+									+	X	X	X	X	X	+					
ORAT	6																																											+	X	X	X	X	+																		
KARI	7																																														+	X	+						+	X	X	+								
HYYP	7																																														+	X	+						+	X	X	+								
VIIT	7																																																	+	X	X	X	X	X	+											
PESI	7																																																	+	X	X	X	X	X	+											
PYHI	7																																																				+	X	X	X	X	X	+								
HOPI	7																																																							+	X	X	X	X	X	+					
VAMM	7																																																										+	X	X	X	X	X	+		
KEMI	7																																																										+	X	X	X	X	X	+		
VIIN	10																																																		+	X	+								+	X	X	X	X	X	+
PALJ	8																																																				+	X	X	+					+	X	X	+		
HERV	8																																																							+	X	X	+		+	X	X	+		
RAAH	7																																																													+	X	X	X	X	X	+

Fig. 2-3. The observations sessions in the junction station of the network.

X: The length of the session 24^h

+ : The length of the session $8\text{-}14^h$

3. Data processing

The processing was done with *Bernese v4.0* software (ROTHACHER and MERVART 1996) on DEC 3000 300 workstation running Open VMS operating system.

Computation of the initial coordinates, i.e. processing the permanent network, FinnRef, was a part of a considerably larger projet where the whole data history of the permanent network will be processed in an internally consistent way.

The observations made in different years were computed separately, and only in the last stage, the normal equation matrices were combined to make the final solution of the project.

3.1. Computation of the Coordinates of the Permanent Network

Two sets of data from FinnRef were chosen for processing the initial coordinates for EUREF-FIN campaign. In total 23 weeks of GPS data from years 1996 and 1997 was used for determining the initial coordinates. The processing was devided into 2 major parts. First a free network solution was performed keeping only Metsähovi's ITRF94 coordinates fixed and saving the normal equations. On a second step all normal equations were combined fixing Metsähovi, Joensuu, Vaasa and Sodankylä coordinates to their official ITRF96 values.

3.1.1. Daily free network solutions

The IGS precise ephemerides were given in ITRF94 reference frame in which also the processing was done. Since the ITRF coordinates are dependent on time, Metsähovi's coordinates were transformed to the epoch of the observations according Metsähovi's velocity vector.

The baselines for the network were created automatically by connecting all the points to their 3 closest neighbours. Of course the same baselines were not used 2 times (two directions). This procedure means that we have trivial vectors on our network so we have used baseline correlation in all the processing. The phase differences were created for the chosen baselines. Cycle slips were detected and removed by running the program MAUPRP twice. The program GPSEST was used for parameter estimation. First the ambiguities were solved baseline by baseline using QIF (Quasi Ionosphere Free) method. This is done baseline by baseline fixing one end of the baseline. Three coordinates of another end, one tropospheric zenith delay parameter per 6 hours, and ambiguities were solved. From this solution the ambiguities of which typically more than 85% were solved were saved. In the next step GPSEST is run again. This time we use L3 linear combination and pre-eliminate the ambiguities. From this solution the Normal equations are saved.

3.1.2. Final Combined Solution

For processing the initial coordinates for EUREF-FIN campaign 23 weeks of normal equations from daily solutions of years 1996 and 1997 were used. 11 weeks from 1996 and 12 weeks from 1997 (Table 3-2 and Fig. 3-1). This data set covers also the time period of the EUREF-FIN campaigns.

Table 3-2. The periods of data used for processing the initial coordinates for FinnNet stations.

Year	Date	GPS weeks	GPS days
1996	30.6.-14.9.1996	860-870	182-258
1997	20.7.-15.9.1997	915-926	201-284

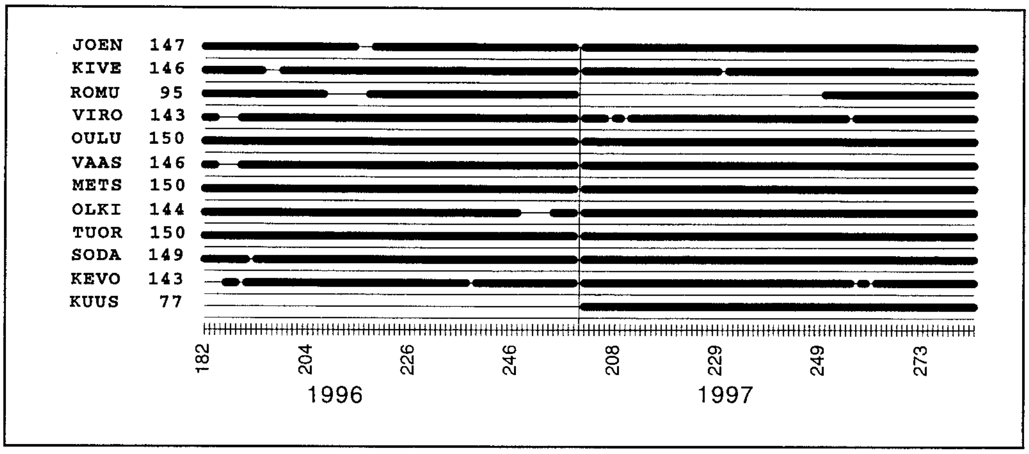


Fig. 3-1. Overview of the data used for processing the initial coordinates. The amount of days of data is shown after the station name.

We tied permanent GPS network to the 4 EUREF points, namely Metsähovi, Joensuu, Sodankylä and Vaasa. Metsähovi has been part of IGS network since 1992 and EUREF network since 1996. The other 3 stations have been part of Euref network since 1997. The coordinates of all these four stations were available in ITRF96 reference frame for epoch 1997.0. The epoch is in between our two EUREF-FIN campaigns and was chosen to be a reference epoch for initial coordinates. As a result we get coordinates for all 12 FinnNet stations in ITRF96 system on epoch 1997.0. Fig. 3-2. shows the deviation of daily solutions in respect of final solution. The figures show that the deviations is only few millimetres in North and East components and far less than a centimetre on up component. The rms's for individual stations of the final network solution are shown in Fig. 3-3.

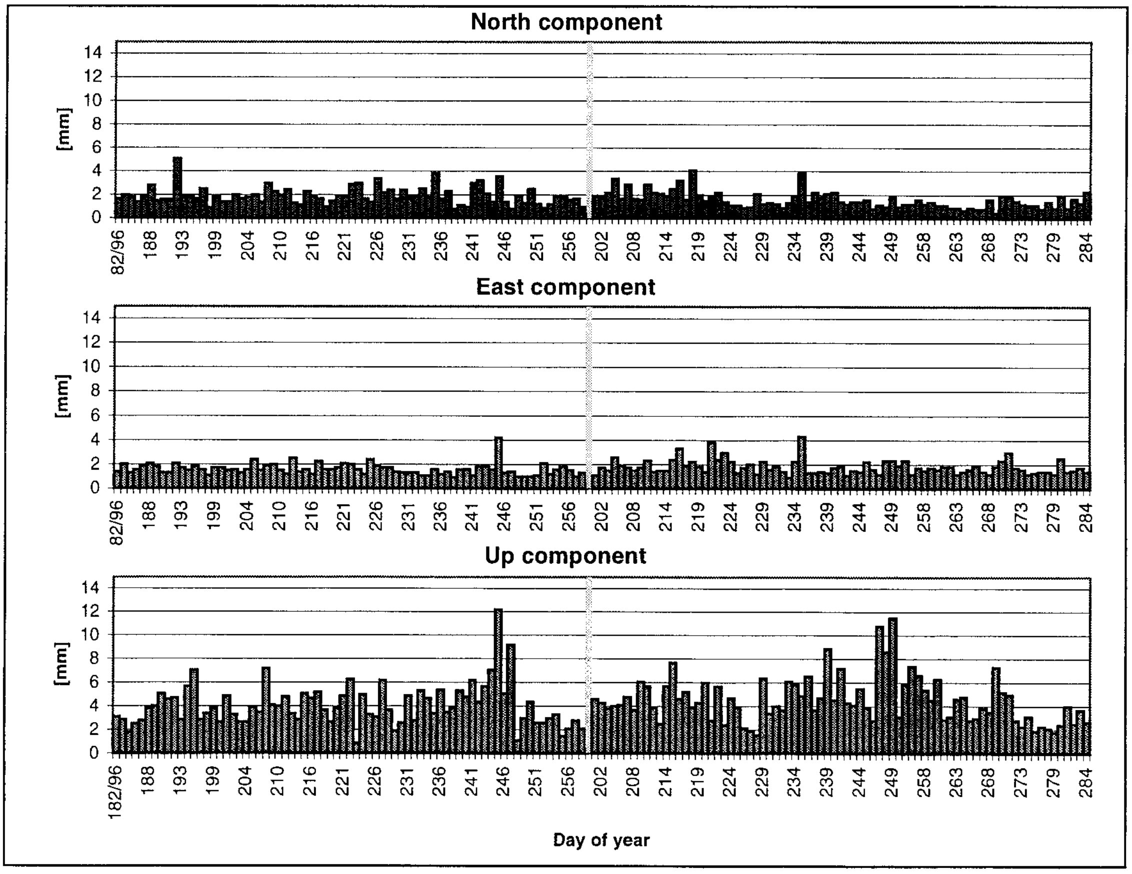


Fig. 3-2. Deviation of the daily solutions from the final solution when 4 stations were fixed.

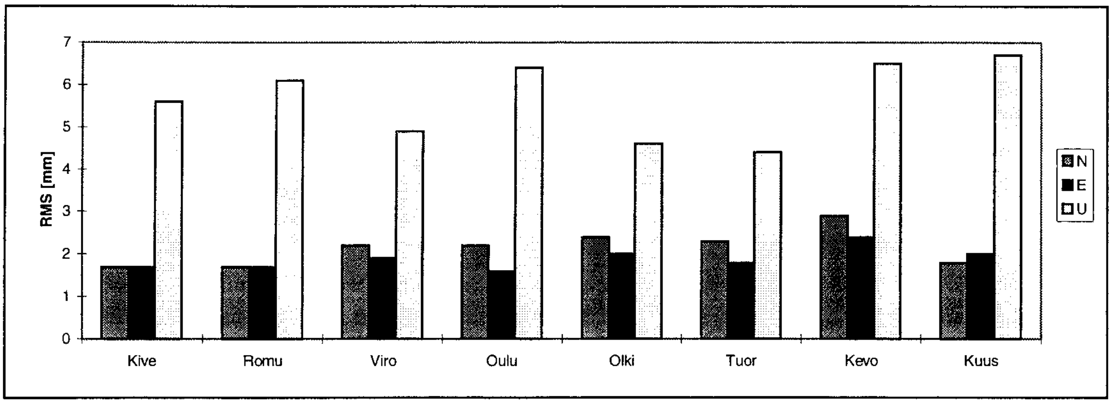


Fig. 3-3. Rms's from the final solution where METS, VAAS, JOEN and SODA were fixed.

3.2 Computation of the coordinates of the densification stations

The computation of the densification points were done in two phases, i.e. 1996 and 1997 data were processed separately. We used the IGS preceise ephemeris for the GPS satellites. The orbital elements were given in ITRF94 for the whole period of 1996/97. This ensures the unaltered reference frame for the whole densification.

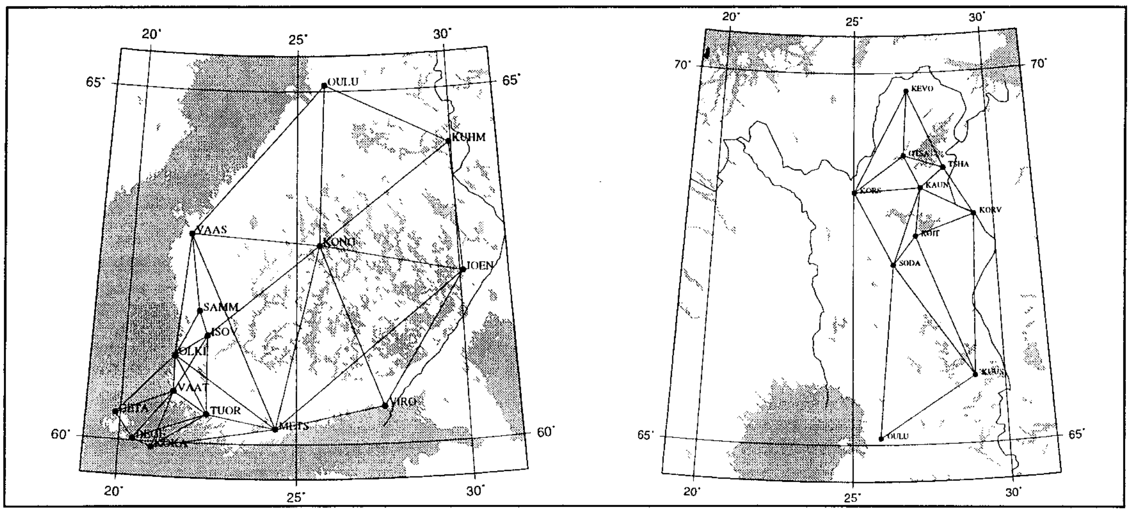


Fig. 3-4. Examples of vector definitions.

We used a self-made semi-automatic program to define the vectors in a session. The program connects each point to three nearest points which are closer than a pre-defined distance. For the cutoff distance we used 300 km. The final selection was made manually removing and adding some vectors. Examples of the final vector selection are shown in Fig. 3-4.

Additionally, the skeleton of the FinnRef stations were used in all sessions so that stations METS, VIRO, TUOR, OLKI, VAAS, KIVE, JOEN, ROMU and OULU were included in 1996. In 1997 stations KIVE, ROMU, OULU, KUUS, SODA and KEVO were used. The coordinates of these stations were constrained to their approximate mean values at the mean epochs of the observations in 1996 and 1997 as described above.

The ambiguities were resolved using the QIF algorithm (MERVART 1995). Typically, 70–90% of ambiguities were resolved, but occassionally, some very poor vectors were also discovered. On these, only 20–50% of ambiguities were fixed.

The final session-by-session solution was made with L3 combination eliminating the ambiguities resolved in the previous phase. Because of redundant vectors, we used baseline wise correlations only. Normal equations were saved, each session separately. The local troposphere estimation (4 parameters/session) was made keeping one station fixed and the troposphere parameters were saved with the normal equations. The whole processing was done using the 15° cut-off angle and no elevation-dependent weighting.

We made first a free network solution fixing Kivetty in 1996 and Sodankylä in 1997. Coordinates of other FinnRef station were compared to their initial values. This showed if there were any inconsistencies or erroneously resolved vectors which could distort the network. Especially, bad repeatability was an indicator of possible problems.

The final adjustment was done with the Bernese module ADDNEQ. We combined the solutions of years 1996 and 1997 first separately. This gives us a good overview of possible bad sessions or points. Some tests were made to find the optimal way to do the adjustment. Finally, all FinnRef stations were constrained to their ITRF96(1997.0) coordinate values and the final adjustment was made.

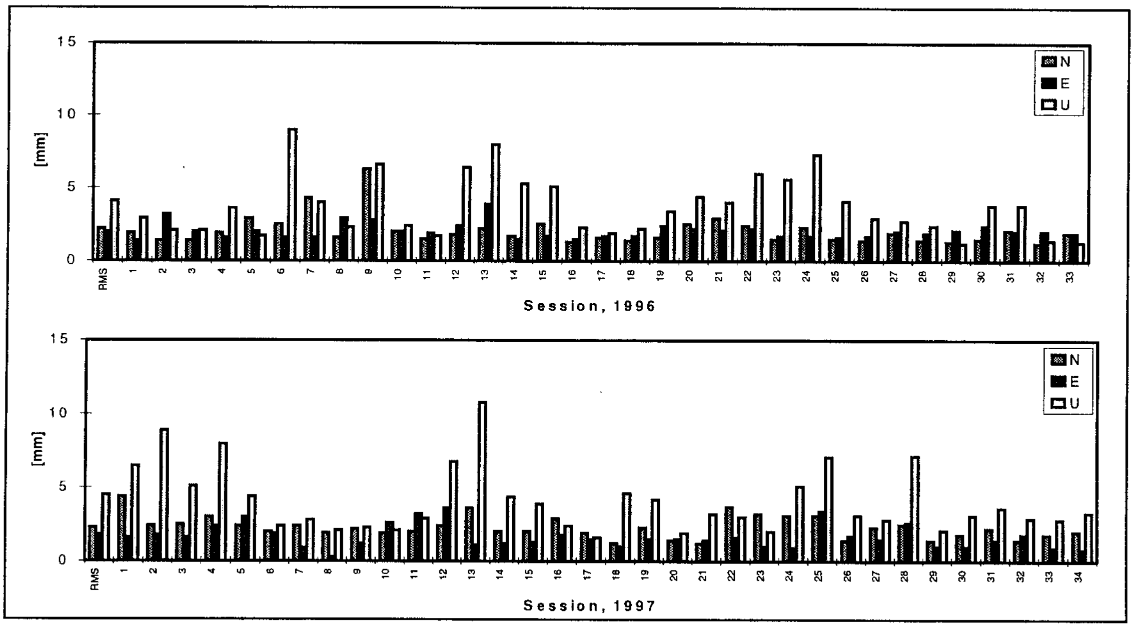


Fig. 3-5. RMS of sessions in 1996 and in 1997.

3.4 The definitive coordinates

The main aim of the GPS campaign was to densify the network of stations connected to the EUREF89 coordinate system. When aptempting to get the resulting coordinates in a system as close as possible to the original system several difficulties arise. The GPS solution was made in the ITRF96, the epoch of which was 1997.0. This does not coincide with the EUREF89 system. One way to transform the coordinates from ITRF96 to EUREF89 is to use the transformation parameters and plate motion velocities published by IERS. If this principle should be followed we have to accept the fact that the EUREF89 coordinates of the initial point would change from year to year depending on the ITRF solution in question.

In order to see how large is the deviation of the coordinates resulting in the different ITRF solutions we made a simple experiment with the GPS coordinates of *Metsähovi*. We took the ITRF coordinates of Metsähovi (BOUCHER *et al.* 1991a, 1991b, 1992, 1993a, 1993b, 1995, MCCARTHY 1996) and transformed them to the EUREF89 system (GURTNER *et al.* 1992) according to the principle by BOUCHER and ALTAMIMI (1995). The velocity components of *Metsähovi* and the transformation parameters were taken from the publications of IERS. The

EUREF89 coordinates transformed from various ITRF coordinates are illustrated in Figs. 3-6a, 3-6b and 3-6c. The deviation of the reduced EUREF89 coordinates from the original EUREF89 coordinates is several centimetres depending on which ITRF system is selected as the initial system. If these discrepancies are introduced to a national reference frame by using different initial coordinates year after year the resulting small scale geodetic networks will contain errors which are coming from global GPS solutions. The situation is not better if the ETRF coordinates are used as initial coordinates; the differences between the reduced EUREF coordinates and original EUREF89 coordinates were similar as those illustrated in Figs. 3-6a...3-6c.

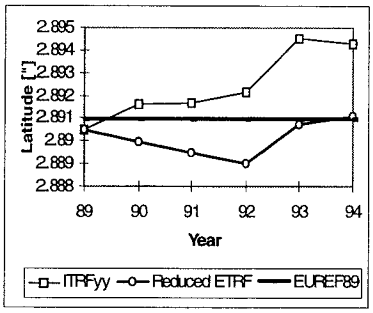


Fig. 3-6a. The original EUREF89 latitude of *Metsähovi* and the ETRF89 latitude reduced from ITRF-yy.

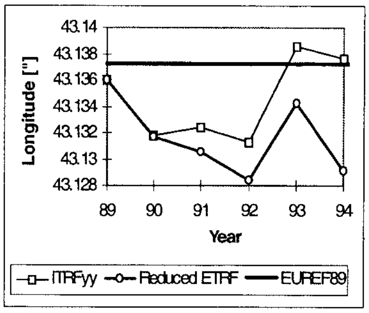


Fig. 3-6b. The original EUREF89 longitude of *Metsähovi* and the ETRF89 longitude reduced from ITRF-yy.

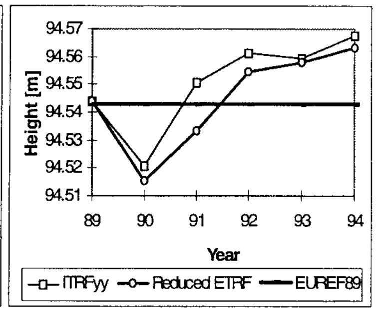


Fig 3-6c. The original EUREF89 height of *Metsähovi* and the ETRF89 height reduced from ITRF-yy.

The coordinates of three EUREF89 stations in Finland have been used as initial points for the determination of several hundreds of GPS points by the National Land Survey, the Finnish Maritime Administration and the FGI since the EUREF89 campaign in 1992. If the coordinates of the principal EUREF89 point will be changed by a new densification the whole reference system and all coordinates connected to it will be changed. The new EUREF densification could be transformed to the original EUREF89 system by a 7-parameter similarity transformation. By doing so, however, we introduce the errors of the original EUREF89 coordinates to our new GPS network, which is, as we believe, significantly more accurate than the EUREF89 network. The RMS of the standard deviations of the coordinates in Finland was in the EUREF89 campaign ±15 mm, ±15 mm and ±7 mm in north, east and height component, respectively. The corresponding figures in the present EUREF densification are ±2 mm, ±2 mm and ±6 mm.

The adjustment of the coordinates was made with ADDNEQ program. If the adjustment comprises only one fixed point no problems exist; the coordinates of the fixed point are handled as they are given in the file of *a priori* coordinates. We wanted, however, to keep all 12 coordinates of the permanent GPS stations fixed in order to maintain the hierarchical structure of the network. In this case one has two possibilities to define the fixed points: either to mark the points fixed or define the *a priori* standard deviations of the fixed coordinates as small as possible, which is ±0.1 mm. Both ways lead to same result: the program adjusts (i.e. corrects) also the coordinates of the fixed stations. In our case the coordinates of the fixed stations did not change much; the largest changes were 3-4 mm. The way to handle the coordinates of the fixed stations might be mathematically correct, but it ruins the hierarchical structure of the network. Let us think that one has to measure a new point in the network and connects it to two old points. In order to maintain the consistency of the coordinate system the adjustment has to be carried out so that the fixed coordinates do not change at all. If the new adjustment changes the coordinates of the fixed stations, who knows in which coordinate system the new coordinates are.

To get the new set of coordinates as close as possible to the original EUREF89 system and at the same time maintain the internal accuracy of the GPS solution we transformed the coordinates from the ITRF96 to the EUREF system in the following way: the rectangular coordinate differences between the two systems were computed in *Metsähovi*. The differences were then added to all coordinates resulting from the campaign. We refer to the reference system defined by the coordinates of the densification points as EUREF-FIN. To compute the coordinate differences in *Metsähovi* we used here the coordinates resulting from the ADDNEQ adjustment.

	EUREF89 [m]	ITRF96 [m]	Differences	[m]
X	2892571.072	2892570.939	ΔX	0.133
Y	1311843.306	1311843.337	ΔY	-0.031
Z	5512633.939	5512634.047	ΔZ	-0.108

When the corrections above are applied to the GPS coordinates at the two other EUREF89 stations the consistency of the EUREF-FIN and EUREF89 systems can be seen. The deviations of the horizontal coordinates are within ±50 mm, but the height deviates -76 mm at the other EUREF89 station.

Table 3-5. The deviation of the EUREF-FIN coordinates from the EUREF89 solution at three common stations in mm.

Station	ΔX mm	ΔY mm	ΔZ mm	ΔN mm	ΔE mm	Δh mm
348	0	0	0	0	0	0
165	-37	36	30	27	50	18
275	-54	-63	-52	52	-31	-76

4 Coordinate comparisons and transformations

Main part of the densification points belong to the first order network Finland. The triangulation network has been adjusted several times which resulted in various coordinates for the stations. The most important of these adjustments were the national ED50 adjustment (KORHONEN 1967), the international ED87 adjustment (EHRNSPERGER 1989) and the national ED87 adjustment (JOKELA 1994). The EUREF89 system was realized in Finland during the EUREF89 GPS campaign when 4 triangulation stations were observed (GURTNER *et al.* 1992). Three stations included in the original EUREF89 network were observed in this campaign, too. The first EUREF densification was made in 1992, when 22 triangulation stations were connected to the EUREF89 stations by GPS observations (OLLIKAINEN 1995). The repeated determinations offer various possibilities for coordinate comparisons and derivation of the transformation parameters between EUREF-FIN system and the terrestrial coordinates systems. The number of stations included in different campaigns is shown in Table 4-1.

Table 4-1. Number of the densification stations included in the previous determinations.

Campaign	Nr. of common stations	Reference
ED50	73	KORHONEN 1967
ED87 International	77	EHRNSPERGER 1989
ED87 National	89	JOKELA 1994
EUREF89	3	GURTNER *et al.* 1992
GPS Finland'92	15	OLLIKAINEN 1995

4.1 Comparison of the coordinates with the previous GPS determinations

Three of the stations observed in this campaign were included in the original EUREF89 campaign (GURTNER *et al.* 1992). The coordinate difference at one fixed station, i.e. 348 Metsähovi, between new solution and the original EUREF89 solution was used to convert the new coordinates back to the EUREF89 system. That is why we can compare the new coordinates with the EUREF89 solution only at two stations. The differences between EUREF89 and EUREF-FIN coordinates were discussed already in the previous chapter.

The first EUREF89 densification was carried out in 1992, when 22 first order triangulation stations were connected to three EUREF89 stations by GPS observations (OLLIKAINEN 1995). In this campaign 15 of the stations in the 1992 campaign were reobserved. The coordinate differences between the EUREF-FIN solution and the GPS Finland'92 campaign are shown in Table 4-2. The overall agreement between these solution is better than 10 cm. The largest differences are 65 mm, 64 mm and 90 mm in the north, east and height component respectively. The differences in the horizontal coordinates are shown in Fig. 4-1, in which the systematical shift of the coordinates may be noticed.

Site	ΔX mm	ΔY mm	ΔZ mm	ΔN mm	ΔE mm	Δh mm
42	-46	12	-33	14	32	-46
63	-71	-8	-68	27	27	-90
69	-67	20	-66	15	51	-81
84	-64	19	-24	32	46	-45
158	-39	24	8	25	39	-4
165	-37	36	30	27	50	18
171	-75	33	-6	44	64	-29
212	-64	8	-19	37	36	-42
229	-85	19	-44	44	53	-68
246	-74	13	-54	31	46	-74
275	-54	-63	-52	52	-31	-76
301	-83	-15	-24	65	24	-54
309	-89	18	-51	46	56	-76
314	-104	13	-44	65	56	-74
Mean:	-68	9	-32	37	39	-53
St.dev.	19	25	28	16	23	31

Table 4-2. The coordinate differences between the EUREF-FIN and the GPS Finland'92 solutions in mm.

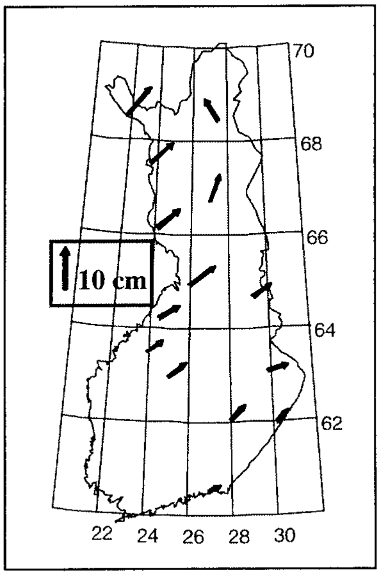


Fig. 4-1. The change of the horizontal coordinates from GPS Finland'92 campaign to the definitive solution of EUREF-FIN.

5.2 Comparison of the GPS coordinates with coordinates of the first order triangulation

The main part of the observation stations were first order triangulation points. The comparison of the coordinates resulting in the adjustment of the triangulation network with those obtained by GPS measurement shows some interesting features in the triangulation coordinates. The Finnish first order triangulation network, which was observed during 1922-1987, has been adjusted several times. Only some most important adjustments were used here for the comparisons, i.e. the national adjustment in 1966 (KORHONEN 1967), the international adjustment in 1989 (EHRNSPERGER 1989) and the national adjustment in 1994 (JOKELA 1994).The adjustment made in 1966 was computed in the ED50 system and the two latter adjustments in the ED87 system. The national adjustments were computed in the European

The renovation of the Cadastral control-network

By Søren West-Nielsen, Ole Eiersted and S. Stampe Villadsen, Kort & Matrikelstyrelsen, Denmark

Abstract

From 1990 - 1997 the Cadastral contol-network with 330.000 points was readjusted, as a basis for the digital Cadastral map covering the whole area of Denmark. This paper deals with the problems in the the renovation process, the outcome, and the future for the cadastral network.

Introduction

It is an assumption that data, collected for different purposes to be used together in a GIS, are coordinated in the same coordinate system. Physically the coordinate system is accessible in the field on coordinated fixpoints. The quality of this coordination is decisive for the subsequent use of the points for cadastral and tecnical purposes.

The Danish horizontal reference network consists of the primary triangulation network ("GI-network" of app. 21.400 points) and of the detail fixpoint network established for cadastral use ("MV-network" of app. 330.000 points).

A. The purpose of the renovation of the MV-network

Since the 1930'ies the main part of all new established cadastral border lines have been connected to the MV fixpoint network. Therefore it is possible to each measured cadastral point to calculate coordinates, which can be used if the cadastral point shall be reestablished or it can be used in connection with construction of the cadastral map in analogous or digital form.

The MV fixpoint network has therefore been used as the basis for establishing the digital cadastral map in Kort & Matrikelstyrelsen (KMS) both in a pilot experiment carried out on the island of Funen in the period 1985-1990 and in the nationwide conversion of the cadastral map to digital format in the period 1990-1997.

However, the MV network could not immediately satisfy the demands for the quality in the digital cadastral maps, mainly because of the in-homogenity in the MV network. It was a known fact, that most of the new MV points established in connection with the privat chartered surveyors elaboration of registrations only were measured - and coordinated from allready existing MV points. This in-homogeneity expressed itself when introducing new measuring techincques in the 1970'ies.

A precondition for using the MV fixpoint network as a basis for establishing the digital cadastral network was therefore that the MV fixpoint network should be improved.

B. Methods, aims and results for improving the MV fixpoint network

The pilot experiment on Funen (1985-1990)

The first attemt to improve the MV network on Funen was based only on aerotriangulation and transformation procedures.

Every GI point and every third MV point were found in the field, signaled and coordinated by

Fig.1: Map showing the location of the Cadastral Control Network, along roads and in developed areas

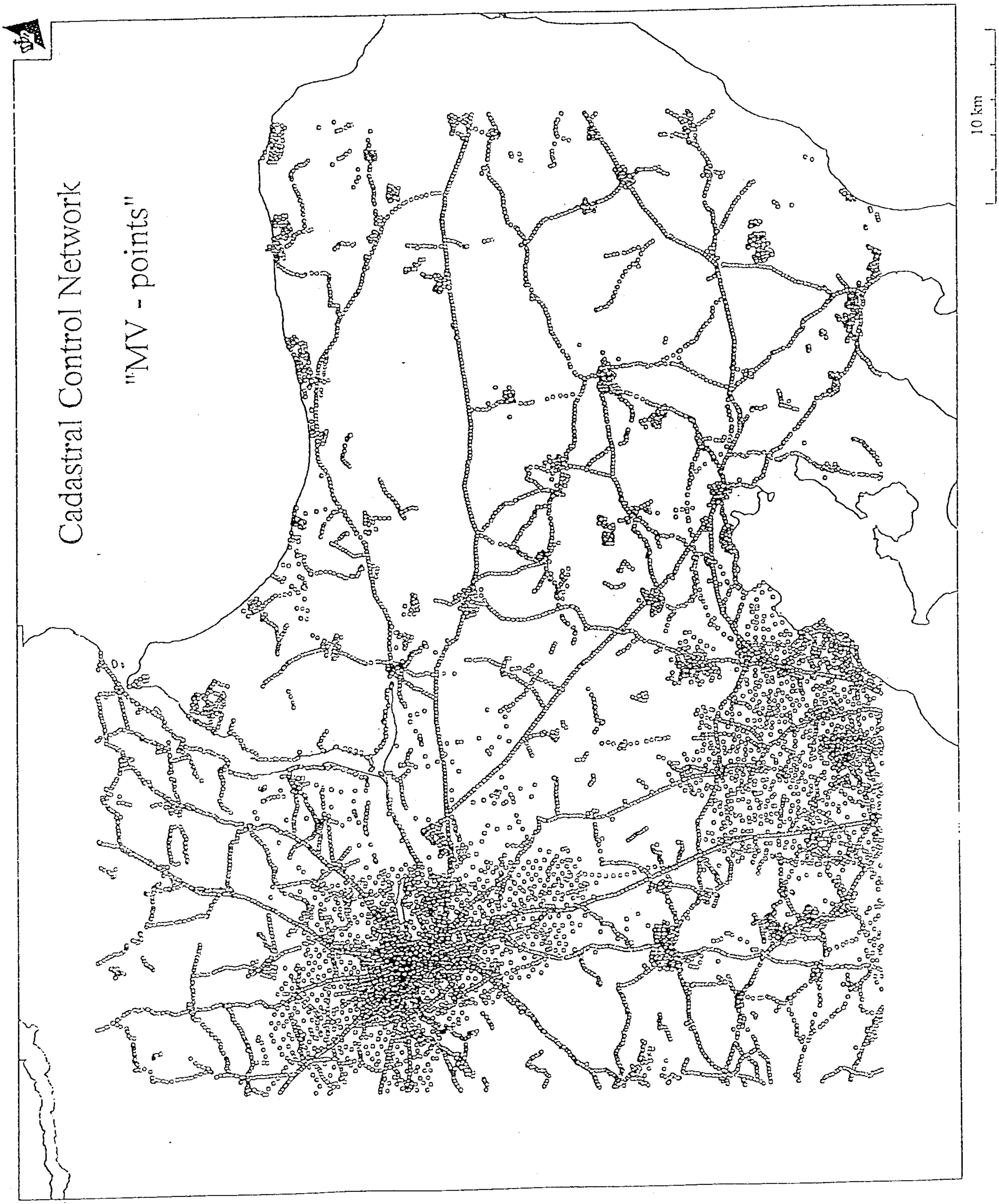
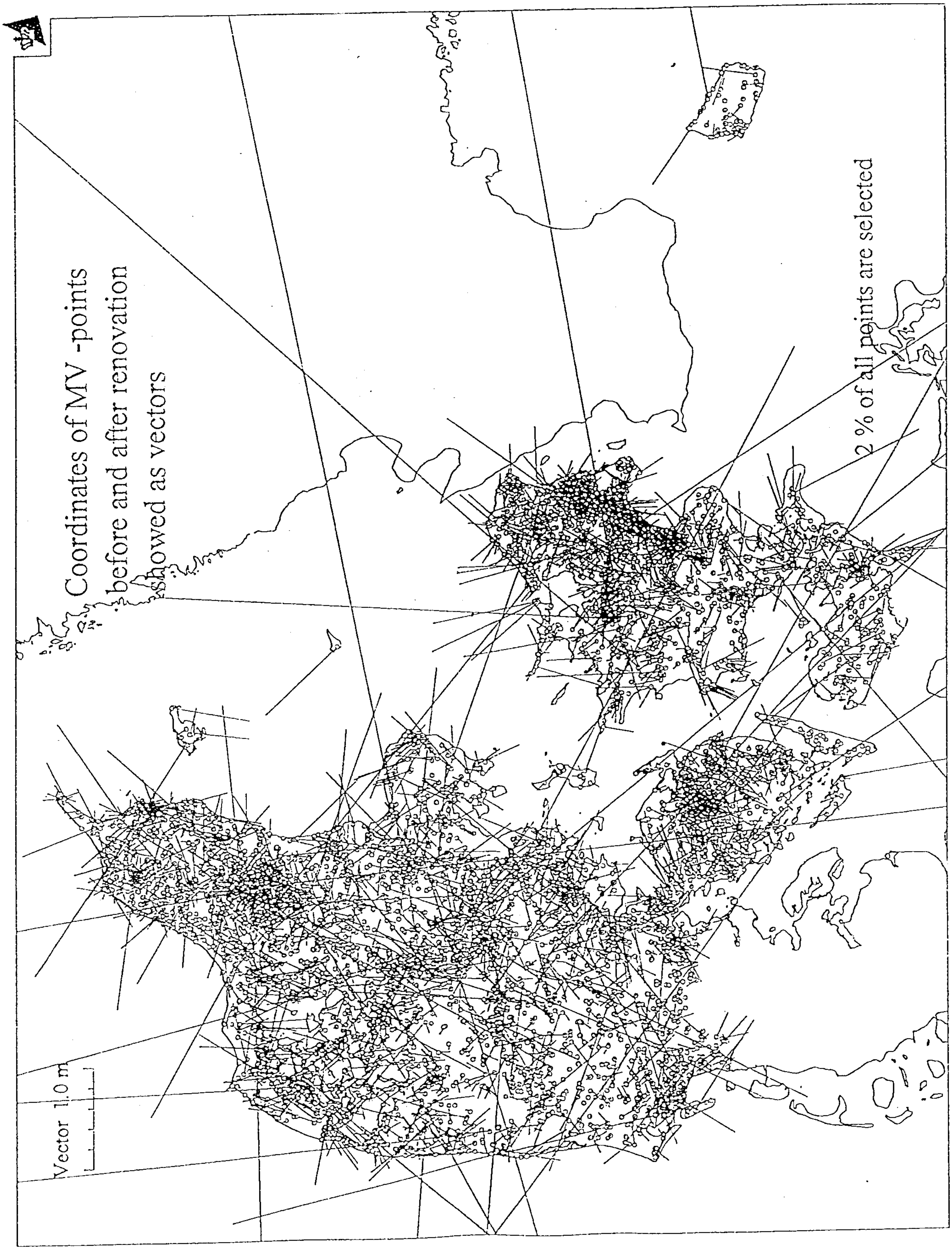


Fig.2 Map showing the change of coordinates after the renovation. The "mean" change in position is about 18 cm.



From 1990- 1994 most of the nessecery new observations to the MV points was made by employers at KMS, first with traditional terrestical metods, later on by GPS.
From 1994 most of the field work was done by private surveyors, using their knowledge to their working area, especially in finding the points in the field. GPS measurements too was made by private surveying compagnies. In 1994 only 2 or 3 compagnies were able to handle the GPS tecnique. In 1997 this number comes close to 40. By getting the surveying job done outside KMS, we have achieved, a quicker distribution of the GPS tecnique, and a higher quality standard in private surveying practice.

From the private licensed surveyer who are the daily users of the MV-network, there is only positive reporting. The days with larger discrepancies in the network is over, and they dare to use the points to more than cadastral work. The network has become reliable.

E. Economi, productivity

In total, the nationwide project (exclusive the pilot project on Funen covering about 10% procent of Denmark) is done with 96 man years work, about 30% less than expected when the project started.. In addition to this number, we have spend about 7.2 mio dkr. mostly on private surveyors, their findings and surveying of the MV-points.

The productivity is increased dramiticly. In numbers, - measured in percent of the total number of points in the country -,one man/woman has adjusted in one year:

	1990	1991	1992	1993	1994	1995	1996	1997
Productivity	0,61%	0,58%	0,77%	0,87%	0,95%	1,36%	1,30%	2,18%

There are three main reason for the increase in the productivity.

- Higher professionalisme, more shortcut in the adjustment process
- In 1995 we changed the the production line from user-controlled iniative to production-controlled initiative. Before that time we were not allowed to finalize the ajustment of a municipality before a contract of bying the cadastral map were made. The change gave us an opportunity to handle more than one municipality in an ajustment.
- In 1996 the new edp-software was introduced, wich gave much quicker answer and shorter calculation time. Th introduction can also be seen in the decrease in productivity in 1996, were the new programme system had its introduction time.

F. Summary.

With the renovation of the MV-network we have achived the most important goal, to create the basis for establishing the digital cadastral map. In the same time we have achieved a possibility through creating of observation- and coordinate database to raise the quality of the MV-network and the digital cadastral map. New measurements to the existing MV-points can easily be used together with the old observations.

The MV-network and the money spend in it to establishment, point sketches, measurement etc. is activated and secured for the future.

The old dream, to put all kind of maps, i.e. tecnical-, topograpical- and cadastral map together is a reality. All the maps are now based on the same homogenius coordinate system.

With all kinds of maps made on base of the same homogenios coordinate system, it is easy to transform the maps to a better and more user-friendly coordinate system than the old danish system from 1934. A better and coming system is the EUREF89.

Modern survey tecnique reduces the need for fixpoints, including the very densed MV-network. Though there will still be a need for a good part of the points, e.g. good coordinated points distributed equally all over Denmark. It will allways be an advantage to be able to start a new measurement from a known point instead of starting all over again.

A New Three Dimensional Reference Network in Denmark

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Introduction

When GPS was introduced as a tool for surveying and geodesy the need for a new reference network and a new reference frame became evident in Denmark.

Observations from GPS are three dimensional vectors, which is in disagreement with the traditional triangulation and levelling techniques that were used to establish the existing two dimensional horizontal network and the one dimensional vertical network. In order to obtain 3D coordinates from a GPS survey it is necessary to have access to reference stations that are coordinated in all three dimensions. With the existing networks that was not possible.

Furthermore the accuracy of the old networks is not good enough for the heigh accuracy that can be obtained with GPS especially over longer distances.

In order to obtain the full benefits of GPS for geodetic purposes it is not only necessary with a good reference network. Access to a reference frame concordant with the WGS84 reference frame used for the GPS satellites and control stations is also a requirement.

In order to fulfill the demands from the GPS users a new three dimensional reference network and a new reference frame had to be introduced in Denmark. Kort & Matrikelstyrelsen consequently decided to introduce the European Terrestrial Reference System (ETRS) and at the same time create a new reference network.

The Danish EUREF89 stations

In the original EUREF89 observation campaign in 1989 two stations were occupied in Denmark. During the national densification work that started in 1992 it became evident that the results from this campaign were inconsistent with GPS observations made under a new set of conditions. Firstly, observations could be made by a new generation of non-squaring receivers with more channels. Secondly the ionospheric disturbances were declining concordant with lower sun spot activity following Solar Cycle 22 which had a maximum in 1990. Finally the establishment of the IGS global network and precise orbit determination since 1992 reduced the orbital geometrical biases in GPS observations.

As in many other European countries a new realization of the European Reference Frame was decided, and a new observation campaign called EUREF-DK94 was performed in September 1994. Six stations in the old 1. order triangulation network were selected to represent an official extension to the EUREF89 network. Monumentation and location formulated the criteria for selection of the six stations that are more or less surrounding the country. The station locations can be seen in Figure 1.

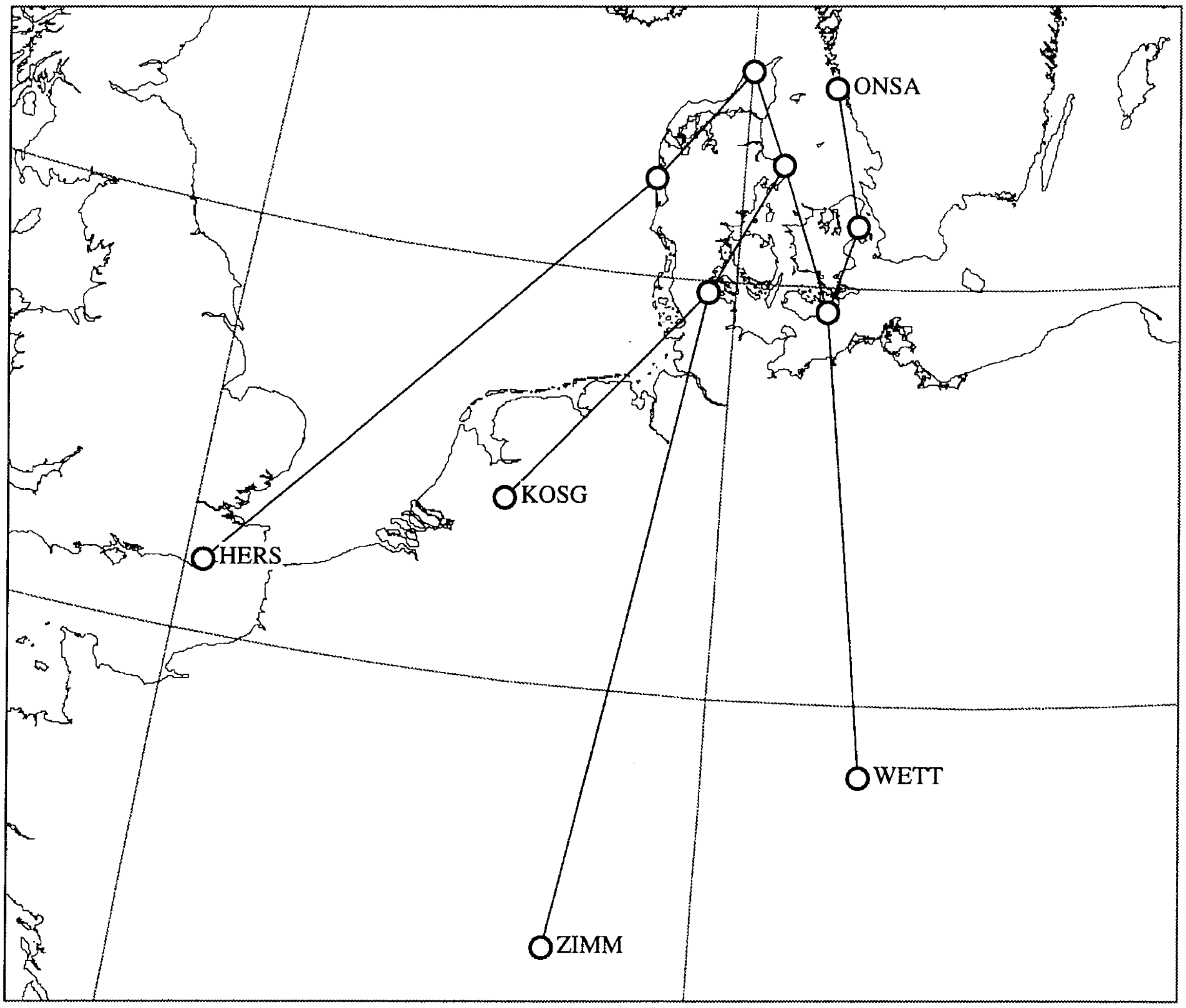

Figure 1. Location of Danish EUREF89 stations and connection to IGS stations.

The stations were observed with dual frequency P-code receivers for five consecutive days. Data was processed at the Astronomical Institute at the University of Berne in Switzerland using the Bernese GPS Processing Software version 3.6 (Frankhauser and Gurtner, 1995).

Observations from the IGS stations Onsala, Wettzell, Zimmerwald Kootweijk and Herst-monceux were included and their coordinates fixed at the epoch of observation in the ITRF92 reference frame.

Results from the EUREF-DK94 observation campaign show daily repeatabilities within 1-2 millimetre in north and east components and 4-9 millimetre in the height components.

The results from the EUREF-DK94 campaign were referenced to the ETRS89 by the formulas in (Boucher, C.,1993), and was at the EUREF symposium held in Helsinki in May 1995 accepted as an official extension to the EUREF89 network classified to an accuracy of 1 centimetre at the time of observation.

REFDK

During 1992 to 1997 a number of GPS observation campaigns were performed to density the six official EUREF89 stations introducing the ETRS89 in Denmark. This densification project is called REFDK (Referencenet Danmark).

The REFDK network consists of 94 stations covering the whole country including the larger islands. The location of the stations is shown in Figure 2.

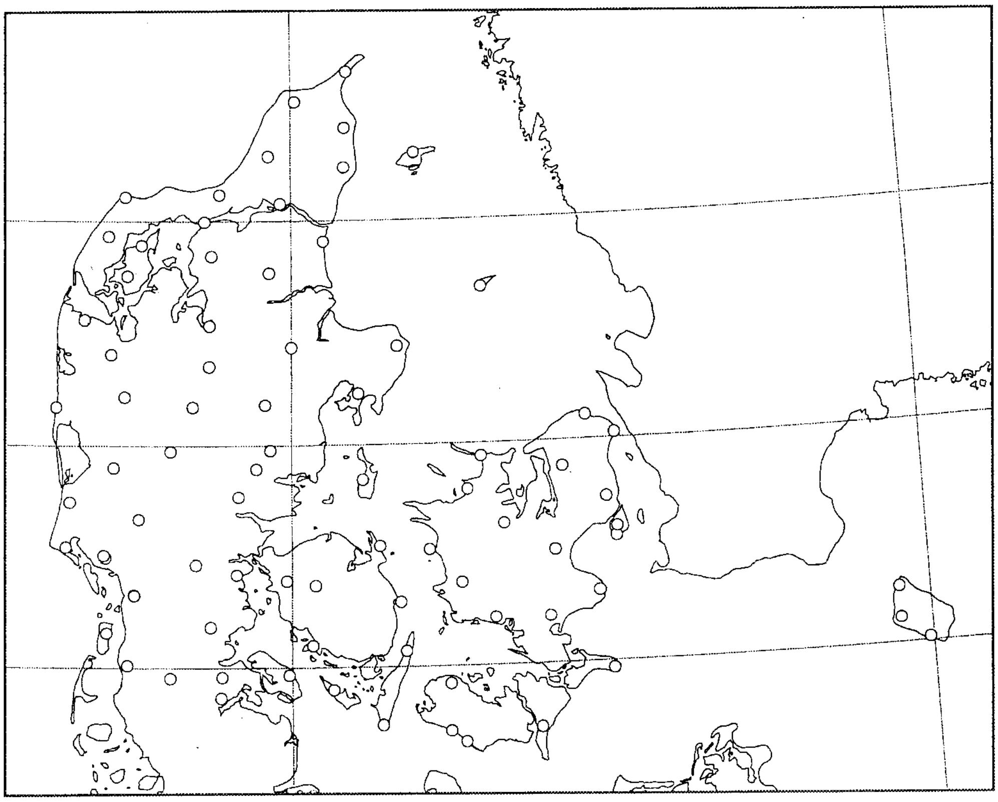

Figure 2. REFDK stations.

The stations in the network are primarily existing stations in the 1. and 2. order triangulation network. These stations are well monumented with granite pillars, and are often situated in open areas that are suitable for observations with GPS.

In the main GPS campaigns in 1992 and 1993 observations were performed during night in order to minimize the influence of the ionosphere. The minimum observation time is four hours.

Initially the processing of observations was performed using the Topas software, but in 1996 the campaigns were reprocessed using the Bernese software version 3.5. This processing is performed under the following conditions:

- Precise orbits from CODE[1] are used
- Observations below 20 degrees are cut-off
- Narrowlane ambiguities are resolved
- Troposphere parameters are determined for N-1 receivers for each 2 hours

Normal equations are saved for each session processed and finally the normal equations are reevaluated fixing the coordinates of the six official EUREF89 stations.

The internal accuracy of the processing is at the millimetre level. Independent tests indicate that the external accuracy of the network is at the 1-2 cm level in both the horizontal and vertical components.

With the 94 stations the REFDK network is considered sufficient for the introduction of the ETRS89 in Denmark. The surveying community in Denmark has asked for a more dense network in the ETRS89, and Kort & Matrikelstyrelsen is in the process of investigating the technical and economical possibilities for this.

Coordinate conversion

Introduction of a new reference frame does not automatically imply that all existing maps, databases and registers with geographical information are converted to the new reference frame. Conversion of maps and data registers can be a time consuming and costly project, and therefore most of the mapping community keep their data in the old reference frames as long as possible. Any new data, collected in the new reference frame, will then be converted to the old reference frames, before being entered in the databases.

This implies a great need for coordinate conversion procedures between the new reference frame and the existing reference frames.

In Denmark mapping and registration of geographical data is traditionally carried out in either the ED50 datum combined with the UTM map projection or in the national Danish coordinate system, System 34. These coordinate systems are realised by the national GI network[2], and some of the observations forming the basis for the network are dating back to the 1930'ies. The old observations are influenced by the precision of the surveying techniques used at that time, and therefore the network appear to be biased when using it today in connection with modern survey techniques such as GPS.

This makes it fairly difficult to determine any analytical formulas for conversion of coordinates between the new and the existing coordinate systems, but an attempt was made with a 7 parameter datum transformation.

The 94 REFDK stations are all coordinated in both the ED50 and in ETRS89, and based on

1. Center for Orbit Determination in Europe

2. The GI - network is the Danish horizontal geodetic reference network consisting of approximately 25.000 stations.

the two sets of coordinates a 7 parameter datum transformation between ED50 and ETRS89 was determined. The formula and the parameters for the transformation are shown below.

$$\begin{bmatrix} x \\ y \\ z \end{bmatrix}_{ED50} = \frac{1}{s} \boldsymbol{R}^{-1} \left[\begin{bmatrix} x \\ y \\ z \end{bmatrix}_{ETRS89} - \boldsymbol{T} \right] \tag{1}$$

Where:

$$\boldsymbol{R} = \begin{bmatrix} \cos r_z & \sin r_z & 0 \\ -\sin r_z & \cos r_z & 0 \\ 0 & 0 & 1 \end{bmatrix} \begin{bmatrix} \cos r_y & 0 & -\sin r_y \\ 0 & 1 & 0 \\ \sin r_y & 0 & \cos r_y \end{bmatrix} \begin{bmatrix} 1 & 0 & 0 \\ 0 & \cos r_x & \sin r_x \\ 0 & -\sin r_x & \cos r_x \end{bmatrix} \tag{2}$$

The translations, T and the rotations, R are shown in the matrices below.

$$\boldsymbol{T} = \begin{bmatrix} -81{,}0703\ m \\ -89{,}3603\ m \\ -115{,}7526\ m \end{bmatrix}, \quad \begin{bmatrix} r_x \\ r_y \\ r_z \end{bmatrix} = \begin{bmatrix} -0{,}''48488 \\ -0{,}''02436 \\ -0{,}''41321 \end{bmatrix} \tag{3}$$

The scale factor, s $= 1{,}0 - 0{,}540645 * 10^{-6}$

The mean error of the transformation is 20 cm. This is not sufficient for the general surveying community, but the formula is however used as a temporary solution and is employed by users that are satisfied with this level of accuracy.

The investigations for an improved coordinate conversion procedure based on polynomials was initiated and a solution was developed by Karsten Engsager in Kort & Matrikelstyrelsen in 1998.

1300 stations from the GI network - that is coordinated in the UTM projection, datum ED50 - where selected for a recomputation. Based on existing observations coordinates were computed in the ETRS89. The coordinates in the two reference frames were used for developing a set of regular polynomials of degree 10 for a 2D coordinate conversion.

The following procedure must now be followed in order to perform the coordinate conversions:

ETRS89 coordinates are initially converted using formula (1). The result is a set of cartesian coordinates that are converted to a “technical coordinate system”. The technical coordinate system is a map projection defined as the UTM, but with a datum that is similar to ED50. The technical coordinate system is only used internally in the conversion procedure and can not be used for surveying or mapping purposes. By introducing the regular polynomials, the technical coordinates can finally be converted to the UTM projection, datum ED50.

The mean error of the procedure described above is 3 cm which is sufficient for most survey applications.

For geodetic purposes it would however be advantageous to have an even better conversion procedure. In Kort & Matrikelstyrelsen a recalculation of the entire GI network (approximately 25.000 stations) from ED50 to the ETRS89 is being carried out. The recalculation is based on existing observations between the stations. The new coordinates can be used to determine a more reliable coordinate conversion procedure between ETRS89 and the existing coordinate systems in Denmark.

Height conversion

When converting coordinates using the polynomial procedure described above the height information is not taken into consideration.

Heights determined using GPS are ellipsoidal heights above the reference ellipsoid and often surveyors need the ortometric height in the Danish height datum that is related to Mean Sea Level. Conversion of the ellipsoidal height to the ortometric height can be carried out using a GPS tailored geoid model.

NKG96 is the Nordic and Baltic geoid model, developed from the largest gravity data set ever used for a Nordic geoid model (Forsberg et al, 1997). From the NKG96 a Danish geoid model called DKGEOID98 has been generated and fitted to the Danish height datum by René Forsberg in Kort & Matrikelstyrelsen. DKGEOID98 has an absolute accuracy of 1 - 3 cm and can thereby be used for conversion of heights in connection with most GPS applications.

A new height datum will be implemented in Denmark in the near future. The Danish geoid model will subsequently be fitted to the new hight datum, enabling GPS height conversion to be carried out in an even better concordance with the actual Mean Sea Level.

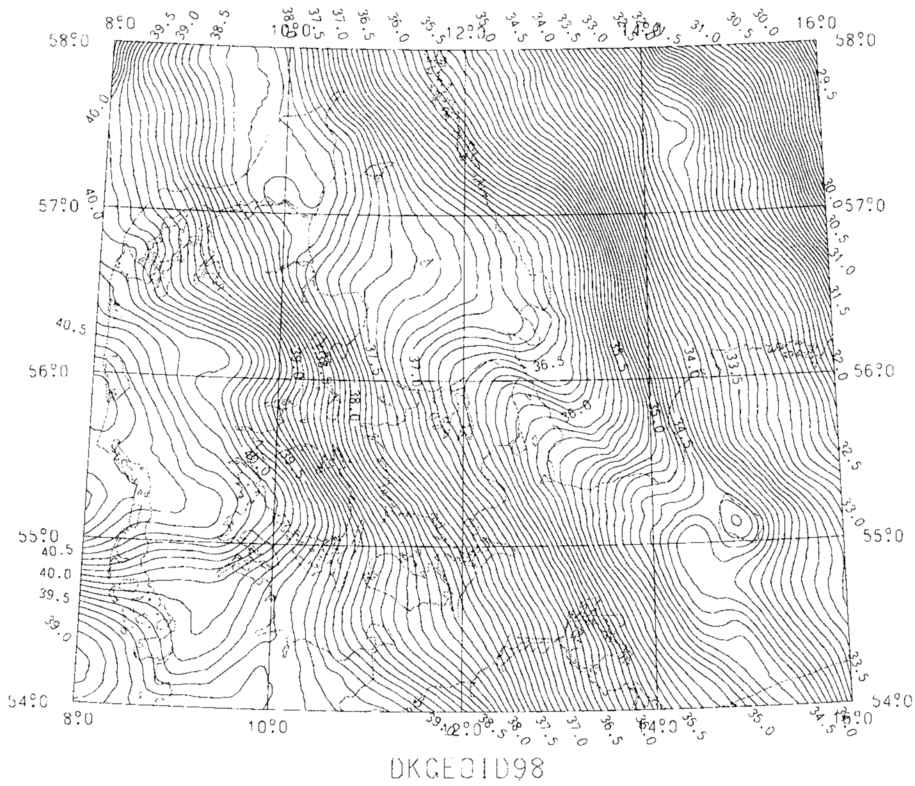

Figure 3. Contour interval 0.1 metre. Figure from René Forsberg, Kort & Matrikelstyrelsen.

Maintenance of REFDK

A reference network must be maintained in order to be reliable. An entire resurvey of the REFDK is nevertheless too expensive to carry out for instance every 10 years. As an experiment Kort & Matrikelstyrelsen defined a monitoring project called MREFDK (Monitoring REFDK).

The objective of MREFDK was to monitor selected nodal points in the precise levelling network by utilising precise GPS surveys relative to the Danish EUREF89 stations. The results of the survey should be used to indicate any local or regional deformations.

The first survey campaign has been carried out. 20 nodal points in the precise levelling network were selected and observed with GPS together with the six EUREF89 stations for four x 24 hours.

Processing of this first survey campaign is not yet finished but data from IGS stations will be incorporated in the processing, and thereby it will be possible to see whether the EUREF89 stations are moving relative to the IGS stations.

When the survey campaign is repeated within a number of years it will hopefully be possible to see whether the precise levelling points are moving relative to the EUREF89 stations or relative to the results of the previous survey campaign. A closer investigation can then be carried out with more intensive surveys in the appropriate regions, occupying both precise levelling points and REFDK points.

The best way of indicating movements in the surface topography would be to establish a dense network of permanent GPS stations and continuously monitor any movements through time series of data from the stations. Establishment of for instance 20 permanent stations is however a costly project, and the MREFDK project can be used as a cheaper alternative a random test project. Since no results from the project are available yet it is not possible to conclude whether the project can fulfill the purpose.

Conclusion

The use of GPS for surveying and geodesy requires a reference system that is in the best possible concordance with the global system the GPS satellites and orbits are represented in. The best way of securing this geometrical agreement is to utilise a reference system determined directly in the reference frame of the GPS system.

Kort & Matrikelstyrelsen has with the REFDK provided the necessary reference network making it possible to perform GPS surveys directly in the ETRS89. The REFDK network has also founded the basis for development of a coordinate conversion procedure making it possible to present the GPS survey results in the existing Danish reference systems.

The need for a homogenous and accurate reference network is based on the increasing need for integration and harmonisation of geographical information also across borders, which will imply that more and more data will be determined in the ETRS89 in the years to come. In the future all the Danish topographical and nautical mapping performed by Kort & Matrikelstyrelsen will be carried out based on the ETRS89.

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National Survey and Cadastre
Denmark
1998

Testing of a priory mean errors of Geodetic Observations

Abstract

Groups of observations may have different a priory mean errors and different weight functions. This holds especially for different kinds of observations. The degrees of freedom for the test is determined after calculations of the **local redundancy** associated to each observation. Some results are demonstrated.

Introduction.

The testing a priory mean errors of observations has been described earlier see, e.g. Milbert or Kubik. The present description is made because the **local redundancy** is calculated as a natural element in the adjustment process. Furthermore do you often have a good estimate of the a priory mean errors, and therefor you only want to test that two or more groups of observations are having the same **estimate of mean error of the weight unit.** You may then decide to change the a priory mean error of one ore more groups.

The weight functions used at National Survey and Cadastre are given, and the result of an adjustment is given.

Adjustment.

To each kind of observations exists a functional relation between the coordinates ***G*** of the stations (or station) and the observations. This function is seldom linear. Hence it is necessary to give a set of coordinates to the stations and to linearize the function by taking the partial first derivatives of the function with respect to the coordinates, and use the updates ***dG*** to the coordinates as unknowns in the observation equations ***A***. The symmetric normal equations ***N*** are established from the observation equations ***A***. The normal equations ***N*** are Cholesky reduced to get an upper triangular matrix ***U***. The right-hand side of the equations are Cholesky reduced accordingly. The Cholesky reduced equations are finally back solved. (See figure 1).

OBS FUNCTION	:	$\mathrm{F}(G) = \textit{Observation} + d$
where d *is the error of the* ***Observation***		
LINEARISATION	:	$\frac{d\mathrm{F}(G)}{dG}\, dG = \textit{Observation} - \mathrm{F}(G) + d$
	$\Leftrightarrow$	$A \quad dG = V$
ITERATIONS	:	$G_i = G_{i-1} + dG_i$
OBSERVATION EQ	:	$A_i dG_i = V_i$, *weight* P_i
NORMAL EQ	:	$N_i dG = A_i^T P_i\, A_i\, dG_i = A_i^T P_i\, V_i$
UPPER TRIANGULAR MATRIX	:	$U_i, \quad U_i^T\, U_i = N_i$
CHOLESKY REDUCTION	:	$(U_i^{-1})^T\, N_i\, dG_i = (U_i^{-1})^T\, A_i^T P_i\, V_i$
	$\Leftrightarrow$	$U_i\, dG_i = W_i$
BACK SOLUTION	:	$dG_i = U_i^{-1}\, W_i$

Figure 1. Symbolic description.

The coefficients of the observation equations A_i must due to the non linearity be recomputed in each iteration with the last updated coordinates G_{i-1}.

Redundancy.

The (overall) redundancy of an adjustment is the number of over determinations which equals the degrees of freedom F of the variance of the weight unit of the observations m_c.

$$m_c = \sqrt{\left(V^T P\, V - W^T W\right) / F}$$

The degrees of freedom F is "the number of observation equations" minus "the number of unknowns" (including orientation parameters) and minus "the number of weight reductions", when the normal matrix N is of full rank.

The local redundancy number of observation no. i is defined: one minus the diagonal element h_{ii} number i of the prediction matrix H.

Prediction matrix :	$H = A\, N^{-1}\, A^T\, P$
Local redundancy no. i :	$r_i = 1 - h_{ii}$

Figure 2. Local redundancy.

It is noted, that the only elements of the prediction matrix H used, are the elements of the diagonal h_{ii}. It is therefore possible to calculate these numbers from the inverse of the normal matrix and the observations equations by a sequential run through the observations.

Each observation has the same number of local redundancies associated as the number of observation equations which are established.

The local redundancy may be calculated from either the right, the left or the central prediction matrix (Milbert).

Mean error for groups of observations.

When more groups of observations contribute to the adjustment, it is usually done under the **hypothesis** that **the groups are having the same mean error of the weight unit of the observations**. The square of mean error m_{cj}^2 of the weight unit of the observations of the group G_j is χ^2/F_j distributed, where F_j is the degrees of freedom of the group G_j.

It is calculated:

$$m_{cj}^2 = \left(\sum_{k \in G_j} V_k P_k V_k \right) / F_j$$

The test of two groups of observations G_j, G_k is having the same mean error of the weight unit of the observations is $v^2(F_j, F_k)$ - distributed:

$$test_{jk} = m_{cj}^2 / m_{ck}^2$$

The degrees of freedom F_j of the group G_j is calculated from the local redundancies r of the group:

$$F_j = \sum_{k \in G_j} r_k$$

It should be noted that these formulas for calculation of the mean errors of the weight unit of the observations are only valid when the iterations of the adjustment has run to the end (i.e. $W \approx 0$); - and the tests should not be made as long as you have any weight reduced observations (Engsager, 1997).

Geodetic observations.

An observation in a typical Danish geodetic network may be one of the following kinds:
- a set of directions (angle measurements)
- a distance measurement
- a GPS vector
- a levelled height difference
- a height difference coming from motorized zenith distance measurement (MTL)
- a set of mono comparator measurements (photogrammetry)
- a stereographic model in two dimensions
- a height model measurement from photogrammetry
- a control height model

The distances may either be station to station distance (geometric geodesy (gg)) or be the distance reduced to zero level height (reduced geodesy (rg)). The adjustment program will on basis of the knowledge of the geoid heights reduce the distances to the ellipsoid.

The levelled height differences are metric. When adjusting in geopotential units, the gravity information is put in separately.

The MTL metric height differences are calculated on computers designed for the purpose during the measurement of zenith distance and distance. (Becker, 1988)

The mono comparator measurements are the actual readings of the photographic plates. These observations are treated as in a bundle adjustment. (Schwidefsky/Achermann, 1976).

The stereographic model is coming from older types of photogrammetric stereographical instruments, which reads out the coordinates in a local coordinate system from two photogrammetric plates.

The height model measurements are the coordinates of points having the known height or the same unknown height. This might be the shoreline of seas or lakes.

The control height model is simply introduced to have some height information to stations with no accurate height information. For a group of connected stations the height model may be taken from the topographic maps with a relatively big mean error.

The more kinds of observations which go into an adjustment the more important it is to have good a priori estimates of the mean errors of the observation. These a priori mean errors are used to give the observations their weight.

Weights for geodetic observations.

To any observation is given an a priori mean error composed of two elements:

- distant dependent mean error ($\boldsymbol{m}_d$)
- centring mean error ($\boldsymbol{m}_c$).

The distant dependent mean error $\boldsymbol{m}_d$ is zero in some cases.

The centring mean error is used to signal that a station marking may be unstable over some decades, and to give a mean error of re-centring over the station marking; - for levelling the mean error of the zero point of the levelling rod and the single reading accuracy. A reasonable chosen value for the centring mean error may also prevent that the weight $\boldsymbol{P} = \sigma^{-2}$ becomes huge at a small distance S (so that a so-called weight singularity might occur, (Meissl, 1980)).

When S is the distance between the station and the object, and *"sets"* is the number of repetitions of the observation, the mean error of an observation is given by:

Direction: *EDM distance :*	$\sigma = \sqrt{\dfrac{m_d^2 + m_c^2/S^2}{sets}}$
Taped distance :	$\sigma = \sqrt{\dfrac{m_d^2/S + m_c^2/S^2}{sets}}$
GPS vectors (scalefactor to variance -covariance):	$\sigma = \sqrt{m_d^2 S^2 + m_c^2}$
Levelling observations:	$\sigma = \sqrt{\dfrac{m_d^2 S + m_c^2}{sets}}$
Zenith distance observations:	$\sigma = \sqrt{\dfrac{m_k^2 S^2 + m_d^2 + m_c^2/S^2}{sets}}$
Motorized zenith distance observations:	$\sigma = \sqrt{\dfrac{m_d^2 \Sigma s_i^2 + m_c^2}{sets}}$
Mono comparator measurements (Photogrammetry):	$\sigma = m_c$
Stereographic models 2 dim. (Photogrammetry):	$\sigma = m_c$
Height models measurements (Photogrammetry):	$\sigma = m_c$
Control height models :	$\sigma = m_c$

Figure 3. Mean errors of observations.

Where the m_k is the mean error of the refractive index k, s_i is a single sight length, and ***sets*** often is one. The weight of a distance observation may look peculiar; - but the observation equation is established by logarithmic differentiation of the observation function (see figure 4). The coefficients in an observation equation for distances are then the same as the coefficients in an observation equation for directions rotated 90°

OBS FUNCTION : $F(G) = S + d$

LOG LINEARISATION : $\dfrac{dF(G)}{S\ dG}\, dG = \dfrac{S - F(G) + d}{S}$

$\Leftrightarrow \quad A \quad dG = V$

Figure 4. Observation equation for distance measurements.

The weight for a GPS vector is calculated using the variance-covariance matrix V_c :

$$\mathbf{P = \sigma^{-2} (det\ V_c)^{1/3}\ V_c^{-1}}$$

Typical mean errors are shown in figure 5. The unit ***sx*** means sexagesimal arc seconds, ***ppm*** means part per 10^6 and the levelling unit called ***ne*** means mm/ $km^{1/2}$ and expresses the mean error in mm per square root of measured distance in km. The unit of m_d for taped distance observations should actually be ppm*m^{½}.

	m_d	m_c	m_k
Direction:	*1.5 sx*	*2.0 cm*	
EDM distance:	*3.0 ppm*	*1.0 cm*	
Taped distance:	*150 ppm*	*2.0 cm*	
GPS vectors:	*0.8 ppm*	*0.8 cm*	
Precissionlevelling observations:	*0.7 ne*	*0.01 mm*	
Levelling observations:	*1.2 ne*	*0.5 mm*	
Zenith distance observations:	*7.5 sx*	*5.0 cm*	*0.02*
Motorized zenith distance observations:	*8.0 ppm*	*0.5 mm*	
Mono comparator measurements:		*2.0 μm*	
Stereographic models 2 dim.crd.:		*2.0 cm*	
Height models measurements:		*5.0 cm*	
Control height models dim.:		*20.0 cm*	

Figure 5. Example of a priori mean error.

Concluding remarks.

The results of an adjustment is a little influenced of unbalanced a priory weights.

The error estimates and functions of the error estimates are strongly influenced of unbalanced a priory weights.

It is in the example given below demonstrated that any kind of observation may contribute to the adjustment, when the weights which are determined from the a priory mean errors are balanced (i.e. the test factor is near 1.0). It is therefore unnecessary to work with classes of geodetic networks as for example a first order network established by satellite observations only, a second order network and so on. In dense geodetic networks is so much information about the first order network to be found, so that any dubious first order observation may pass unsuspected in a pure first order adjustment; - but it might be spotted as dubious in an adjustment including all available observations from the networks of lower order.

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Example of an adjustment.

In figure ex. 1 is given details of the input data to the adjustment. There is a total of 4017 stations in the adjustment. 16 of these stations are kept fixed in the two coordinates in the plane with the input UTM coordinates in the EUREF89 Datum; - 3861 are unknown in the plane; - 198 stations have fixed height; - and 1984 are unknown in the vertical. It is seen that a station need not be fixed in all three coordinates, and that a station need not be unknown in all three coordinates. The stations fixed in the vertical may actually have location coordinates with a mean error of 8 metres; - and no observations present to give better coordinates in the plane.

ADJUSTMENT-DATA	
Stations	4017
Fixed in plane	16
New in plane	3861
Fixed in vertical	198
New in vertical	1984
Orientation unknowns	6423
Obs-blocks	12893
Max obj. pr. block	15
Directions	23505
Distances	3416
Vectors	1380
Control heights	166
Control heights blocks	54
Levelled differences	142
Zenith distances	1877
Redundancies	17117

ex. 1. Adjustment data.

The number of orientation parameters (Orientation unknowns) is 6423. The total number of rounds of observations is 12893 (Obs-blocks) having a maximum of 15 objects.

The observations which contribute are 23505 directions, 3416 distances, 1380 vectors, 166 vertical controls in 54 blocks, 142 levelled height differences, and 1877 zenith distances.

This gives a count of 17117 redundancies in the adjustment.

The 166 vertical controls are heights read from topographic maps to 54 groups of local height networks which are not connected to levelled height stations. These local height networks are composed by zenith distance observations which have been measured simultaneously with the measuring of distances between the stations. The purposes of the zenith distance observations have not been to determine the height of the station but to reduce the distance measurement to zero level.

In figure ex. 3 is given an extract of the results of the adjustment. The mean error of the weight unit of the observations is 1,34.

There have been 12 iterations before the stopping condition was fulfilled. The count of redundancy is 17117 and the sum of local redundancies gives 17117 as well, so here is no signal of missing observations for determination of the unknowns in the adjustment. The number of weight reductions is 0, - as expected because the observation files have been cleaned for errors and the factor k for weight reduction is 5 (see Engsager, 1997).

ADJUSTMENT SURVEY

Mean error	1.34
ENP number	5.09
Max binary loss	11
Iterations	12
Redundancy	17117
Local Redundancy sum	17117
Weight reduced obs.	0
Reduction limit	5.00

DISTRIBUTION OF VARIANCE-ESTIMATES

type		var	free	eq.	me	test	v2_025	v2_975
dir		1.80	12397.17	23505	1.34			
dst		1.83	1556.41	3416	1.35	0.98	0.94	1.07
vt3		1.79	2405.20	4140	1.34	1.01	0.95	1.05
niv		1.83	82.93	142	1.35	0.99	0.79	1.32
zds		1.80	563.59	1877	1.34	1.00	0.91	1.11
cb_h		1.92	111.69	166	1.39	0.94	0.81	1.27
dir	gp 1	1.80	12349.90	23435	1.34			
dir	gp 2	1.38	11.74	25	1.17	1.31		
dir	gp 3	1.58	35.53	45	1.26	1.14	0.71	1.55
dst	gp 4	1.83	1147.53	2242	1.35	0.99	0.93	1.08
dst	gp 5	1.54	26.66	30	1.24	1.17		
dst	gp 6	1.88	381.23	1143	1.37	0.96	0.89	1.13
dst	gp 7	3.22	0.99	1	pvv written			
vt3	gp 8	1.82	2076.42	3702	1.35	0.99	0.95	1.06
vt3	gp 9	1.64	328.78	438	1.28	1.10	0.88	1.14

ex. 2. Some adjustment result data.

Then follows a list of test numbers of groups of observations: the variance σ_i^2 (var) of the group, the degrees of freedom F_i (free), the number of observation equations (eq.), the mean error m_{ci} (me) of the group, the test number t_i (test) which is the variance σ_1^2 of the first group divided by the variance σ_i^2 of the group, and at last is given the 2,5% (v2_025) and the 97,5% (v2_975) fractiles of the $v^2(F_1, F_i)$ distribution.

The test number t_i must be in the interval from the 2,5% fractile to the 97,5% fractile of the $v^2(F_1, F_i)$ distribution to accept the hypothesis of the first group and group number i is having the same mean error of the weight unit of the observations.

A line is given for each group of observations: directions (dir), distances (dst), vectors (vt3), levelled hight differences (niv), zenith distances (zds) and control blocks with heights (cb_h).

There after follow some lines where some of the kinds of observations have been split into two or more subgroups dependent upon observing technique and accuracy (age) of measurement.

The test number for in this case the direction subgroups number 2 and 3 (dir gp 2, dir gp 3) against the total group (dir) has been corrected, because the sub groups are contained in the main group. The biggest subgroup is not tested (here the dir gp 1). No fractiles to the v^2 distribution are given for the main group number one and for groups with degrees of freedom less than 30. The variance is not calculated for groups with degrees of freedom less than one, but the pvv sum is listed (***pvv written***).

This result shows that all different kinds and groups of observations are accepted to have the same a posteriori mean error of the weight unit of the observations. The result has been reached during a series of adjustments where the a priori mean errors have been corrected. The resulting a priori mean error settings and the variances are shown in ex. 3. The groups are the same as in ex. 2, but only the subgroups can be listed for the direction groups, the distance groups and the vector groups. Concerning the units ***sx***, ***ppm*** and ***ne*** see above.

type	description	m_d	m_c	m_k	var
niv		10 ne	1.5 mm		1.82
zds		8.5 sx	7.0 cm	0.02	1.80
cb_h			75 cm		1.92
dir gp 1		1.75 sx	2.0 cm		1.80
dir gp 2	cadastral survey	1.50 sx	1.0 cm		1.38
dir gp 3	before 1900	3.00 sx	2.0 cm		1.58
dst gp 4	EDM	3.0 ppm	1.0 cm		1.83
dst gp 5	micro wave EDM	8.0 ppm	2.0 cm		1.54
dst gp 6	tape	5.0 ppm	0.9 cm		1.87
dst gp 7	tape cadastral	200.0 ppm	3.0 cm		3.21
vt3 gp 8	proposed V_c	0.75 ppm	0.75 cm		1.79
vt3 gp 9	associated V_c	0.85 ppm	0.85 cm		1.64

ex. 3. A priori mean errors.

The niv is the group of levelled height differences.

The group of zenith distances (zds) is measured simultaneously with the EDM distances. The zenith distance observations should be used in the reduction of the EDM distances to zero level.

The cb_h is the group of control height blocks which are needed to avoid that some smaller groups of connected stations are singular in the vertical, because they are not connected to any levelled station. The control heights are readings from topographic maps, which explains the relatively high a priori mean error (m_c = 75 cm).

The dir gp 1 is the group of directions of the geodetic survey control network. It is the biggest group.

The dir gp 2 is a small group of directions of the cadastral network.

The dir gp 3 is the group of directions of the geodetic survey control network measured before year 1900.

The dst gp 4 is the group of distances of the geodetic survey control network measured by the electro optic instruments (geodimeter).

The dst gp 5 is the group of distances of the geodetic first and the second order network measured by the microwave length instruments of the Tellurometer type. The measurements are dated from the years 1959 to 1965.

The dst gp 6 is the group of shorter distances measured by steel tapes.

The dst gp 7 is the group of distances of the cadastral network measured by tapes.

The vt3 gp 8 is the group of GPS vectors associated with the proposed emergency variance-covariance matrix. (Engsager, 1997)

The vt3 gp 9 is the group of GPS vectors associated with the original variance-covariance matrix determined by the GPS reduction program.

Submitted for publication

IMPROVING A HORIZONTAL DATUM WITHOUT CHANGING THE COORDINATES

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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 meters. To prevent these errors from having a bad influence on the horizontal fit, the transformation parameters should be determined with no 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[G370] 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 determined by 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 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

$$\mathbf{X}_L = \begin{bmatrix} X_L \\ Y_L \\ Z_L \end{bmatrix} = \begin{bmatrix} (N_L^* + H + N_L)\cos\varphi_L \cos\lambda_L \\ (N_L^* + H + N_L)\cos\varphi_L \sin\lambda_L \\ (N_L^*(1-e_L^2) + H + N_L)\sin\varphi_L \end{bmatrix}$$

$$N_L^* = a_L / \sqrt{1 - e_L^2 \sin^2\varphi_L}$$

$$\mathbf{X}_G = \begin{bmatrix} X_G \\ Y_G \\ Z_G \end{bmatrix}$$

Doing the second best thing is better than doing nothing. Therefore, the approach is to compute 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 ellipsoid although the adjustment of the triangulation network was originally performed on the Hayford ellipsoid. In a second step, the coordinates were converted to Bessel's ellipsoid by a 2D Helmert transformation. These circumstances later gave rise to difficulties to define how the Bessel ellipsoid should be positioned relative to the earth's surface in connection with the RT 90 system. The decision was to have a fitting as close as possible to the old geoidal heights which had been in use for the last two decades for the reduction of EDM distances to the Bessel ellipsoid. Taking the vertical component as the sum of the geoidal height defined in this way and the height above the geoid, and the RT 90 coordinates as the horizontal component, a (2+1)-dimensional reference system was introduced under the name of RR 92.

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 estimated according to the above mentioned criterion. 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 the 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

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

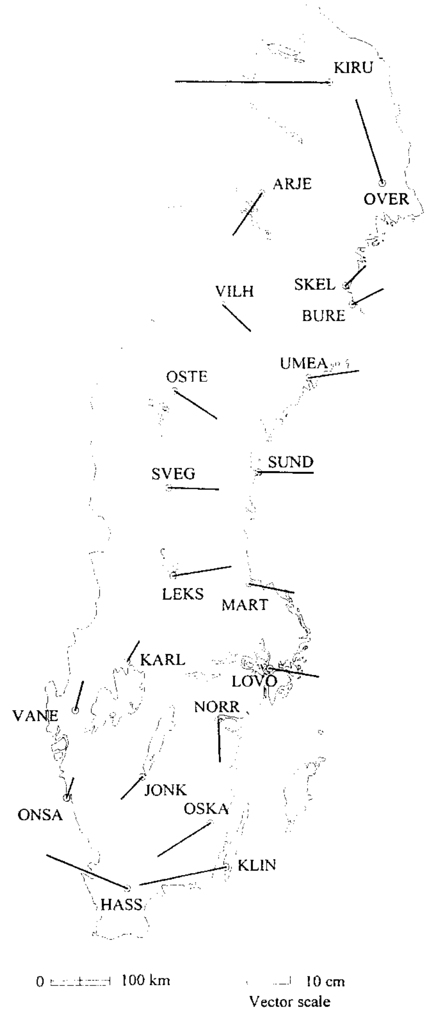

Fig. 1. Conventional fit, horizontal residuals

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.

As can be seen from fig. 2 and table 2 a considerable reduction of the horizontal residuals has 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.

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.

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

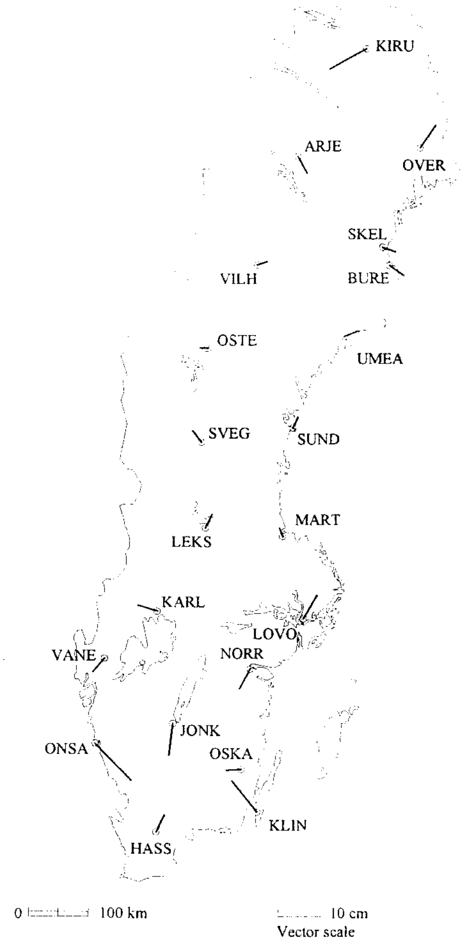

Fig. 2. Optimal fit, horizontal residuals

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 procedure, one step being the similarity transformation. For the average user, this step will probably produce satisfying results. For the advanced user with high accuracy demands, a second interpolation step can be offered.

When computing the 7 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 is 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.

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