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Volume X 1975–1978

PROCEEDINGS OF
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Volume X 1975-1978

by

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INTRODUCTION

In the past three academic years, the program of the Seminar has been concentrated, besides the international and interstate aspect of the water resources problem, on the basic data collection and geodetic survey, including its interrelation with hydrology. The ninth volume is dedicated not only to oceanography and saline water but also to water resources data collecting and to some pollution problems. The entire tenth volume is devoted to the various problems of the geodetic and land surveying in connection with mapping, tidal water, basic data collection and especially the needs of the New York City-Philadelphia area. The eleventh volume is again pollution oriented handling not only water quality but also research in water resources.

Since 1975, the "Annual Meeting in Washington, D.C." has been held in the Cosmos Club with the U. S. Geological Survey as host, where each year a review of the world situation in water resources planning is the topic.

The most important activities of the Seminar besides its regular meetings were as follows:

On Sept. 21-25, 1975 in Reston, Va. the International Symposium on Computer-Assisted Cartography was held and one paper was delivered on water resources oriented data bank.

In the Spring and Fall of 1976, two different teams of Scientists from Hungary, sponsored by the UN, visited the "Land Oriented Reference Data System (LORDS)" of N. J. Bureau of Geology and Topography to learn more about the water resources data bank. This system has been in operation since 1974 with the assistance of the Seminar. The visits were feasibility studies as to how to apply the system also in Hungary.

In the Summer of 1976, the members of the Seminar were asked to write entries for the international "Encyclopedia on Earth Sciences, Vol. XVIII - Geohydrology and Water Resources" as they did in 1972 by contributing 20% of the articles in the "Encyclopedia on Earth Sciences, Vol. IV-A - Geochemistry and Environmental Science". (See introduction to Proceedings, Vol. V). To date, our members committed themselves or wrote over sixty entries (25% of the volume).

On February 14-15, 1977, a "Seminar on Issues before the United Nations Water Conference" was organized in New York City with the assistance of the Seminar to prepare fifty-five participants from fifty-four countries for the United Nations Water Conference in Mar del Plata, Argentina in March 14-25, 1977. Five of our members delivered lectures to assist the United Nations in their effort.

In June 1977, the representative of Arizona State University visited the N. J. Bureau of Geology and Topography to inspect the previously mentioned data bank and its applicability to Arizona.

On August 15-19, 1977 in Baden-Baden, F. R. Germany, three members at the Conference of the International Association for Hydraulic Research, and one member at the University, Ghent, Belgium delivered papers on water resources oriented data bank systems. Researchers from Belgium and Netherland were especially interested in the presentation at Ghent University because they are working on a similar system after they had received information about the data bank in 1974, and they wanted further details on how the system improved since then. The paper has been delivered as a supplement to the report of 1974 at the request of the University Ghent.

On August 23-25, 1977, the Geodetic Survey and Cadastre Offices of the State of Lower Saxony and the Geodetic Institute of Techn. University, Brunswick, both in F. R. Germany, were visited to gain information about the water resources mapping based on geodetic survey.

From Sept. 14, 1977 to Nov. 22, 1977, the National Academy of Sciences initiated international research exchange programs between the United States and Yugoslavia and also between the United States and Hungary. The Chairman of the Seminar was nominated as a fellow of the National Academy of Sciences to exchange ideas about water resources oriented data bank including hydrology of smaller watersheds and karst hydrology. The program generated ten lectures in Yugoslavia and six presentations in Hungary at various universities and national Academies of Science. As a further result of the trip, there were eleven articles prepared in English by Hungarian and Yugoslavian scientists for publication. In the joint program, five articles were delivered in English, German, Hungarian and Yugoslavian by members of the Seminar for publication in scientific journals of those countries. During this visit, the Water Data Banks in Zagreb, Yugoslavia and that of VITUKI in Budapest, Hungary were visited. Both centers had been informed in 1975 about the environmental data bank (LORDS) of N.J. Bureau of Geology and Topography. Finally, an exchange of scholars with fellowships, publications, and a joint research program in soil mechanics and geohydrology were initiated with the involvement of three universities in the United States (Columbia, Rutgers and Fairleigh Dickinson) and the Hungarian Academy of Sciences, Yugoslavian Academy of Arts and Sciences including Techn. Universities in Budapest, Miskolc in Hungary, and the Universities Zagreb and Sarajevo in Yugoslavia.

In the Spring of 1978, two teams again visited the operating data bank of New Jersey in Trenton to check operational procedure and details of the system in order to organize similar information centers. The first team came in May of 1978 from Techn. University Stuttgart, F. R. Germany and the second team came in June of 1978 from the Techn. University Lisbon, Portugal.

Finally, the editors of the Proceedings wish to express their appreciation to all members contributing articles and lectures for the past three years. The publications were made possible only by the generous help and cooperation of the U.S. Department of the Interior-Geological Survey and the State of New Jersey, Department of Environmental Protection.

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GEODETIC CONTROL NETWORK -- FOUNDATION OF THE CADASTRE

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GEODETIC CONTROL NETWORK -- FOUNDATION OF THE CADASTRE (1976)

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ABSTRACT

Almost all kinds of collected data are somehow spatially related in some way to the surface of the Earth. Geographic coding, or geocoding, has become the means by which the integration of spatial data is possible. In the last 20 years, as technology has provided increased computer storage and computing capabilities, the support for and the use of coordinates in geocoding have grown proportionally. Coordinates consequently have emerged as the preferred system of geocoding. Such geocoding of our land records is advocated by the author for the orderly development of our country--a developing Nation.

INTRODUCTION

Geographic coding had its genesis in ancient times with the emergence of cartography and geography. As early as 200 B.C., Eratosthenes had calculated the circumference of the Earth within 200 miles and had devised a grid of latitude and longitude lines on which he plotted the locations of seas, lands, mountains, rivers, and towns with coordinate references. In the first century A.D., the Greek geographer Strabo compiled a multivolume reference, Geographia, which prior to the sixteenth century, represented the most ambitious attempt to catalogue and locate all of the place names in the known world.

For any centuries, geographers, cartographers, explorers, and surveyors continued to devise geographic reference systems whereby areas were subdivided and coded for the purposes of classification, administration, or data collection.

GEOCODING CLASSIFICATIONS

The geocoding systems can be classified as geoidentifying systems or geodefining systems. Geoidentifiers either do not define the position as a place name or imply position, and must be used in conjunction with a map, for example, the arbitrary block number. In contrast, geodefiners make direct reference to the position on the Earth's surface.

The codes can be classified as nominal, ordinal, or cardinal. Nominal coding uses names, words, and alphanumeric codes, such as place names and block numbers. A metes-and-bounds property description is a nominal coding scheme for a land parcel. It is also an implicit geoidentifying system which necessitates that it be used in conjunction with large-scale topographic and planimetric maps.

Ordinal geocodes imply relative position in a spatial system. Two examples are geocodes using a local coordinate system, and geocodes using the public land survey system. In these codes the relative accuracy within the local systems greatly exceeds the relative accuracies between the systems.

The final category in the hierarchy is identified as cardinal geocodes. These directly indicate absolute position in a National or world system. The most frequently utilized cardinal geocodes are geodetic latitudes and longitudes. In reviewing the summary of geocoding systems and regions, found at the end of the paper, two other characteristics are worth noting--the granularity or resolution and the flexibility. The granularity varies theoretically from an infinite number of nondimensional points to several large regional areas. Geocodes possess flexibility to aggregate and disaggregate the coded data.

LINKING GEOCODING SYSTEMS

The weaknesses of these geocoding methods have caused the following disadvantages:

(1) Only a single purpose can be served using job specific formats and codes.

(2) Discipline-dependent definitions are used that are not universally or mathematically accepted, e.g., place.

(3) Many users consider the resolution inadequate, i.e., geographically too coarse.

(4) Method is inflexible; e.g., the percent-of-cell data collection method does not permit redefinition of cell or block size without recollecting the data.

(5) Nonmathematical systems do not adapt well to computerization.

(6) Nonmathematical systems lack the mathematical redundancy and, hence, code reliability.

The fundamental similarities are:

(1) All data are tied to the Earth's surface with some location scheme.

(2) They deal with land and information about land.

The basic pieces of information regarding land are its space location and geometric definitions. The least common denominator of geocoding regions is the land parcel, and communication between geocoding systems is through coordinates.

The Atlanta Conference on land parcel identification systems held January 20-22, 1972, had as its theme, Compatible Land Identifiers-Problems, Prospects, and Payoffs (CLIPPP). The CLIPPP Conference was organized by the American Bar Association to locate a linking mechanism between many systems dealing with real property that would be compatible with all the various types of land title and land use records.

The following six concerned organizations participated in the development of the Atlanta Conference:

Amercan Bar Association (ABA)
American Congress on Surveying and Mapping (ACSM)
American Land Title Association (ALTA)
National Association of County Recorders and Clerks (NACRC)
Urban Information Systems Interagency Committee (USAC)

It was the consensus of this Conference that the linking mechanism should be the State plane coordinates of the visual center, often referred to as a paracentroid of the parcel.

NEED FOR CENTRALIZATION

At the parcel level, the coordinate will be used as only one means of identifying and manipulating computer records. A family of parcel record identifers will be necessary. Many jurisdictions are presently adopting an arbitrary numbering system so that public records (tax, deeds, permits, liens, plans or zoning) can be placed immediately in computer-readable form. These identifiers are geographically oriented, but the parcels are not positioned. The need for precision exists, but the separate local offices cannot independently establish the necessary mapping "system." The emphasis is placed on the total system. This would include the logistics for update, and not jut a one-time mapping project. Under the present system the source of data for large-scale map updates should be the Deeds Registry Office. The surveying and mapping system necessary to initially guarantee county-wide use of relative parcel position (nation-wide would be obtained at no additional cost) can only be economically justified by many users. With more users, the benefits of more precise mapping would produce an attractive cost/benefit ratio. To accomplish this surveying and mapping "system," a State surveying organization will be necessary. Its duties might include:

(1) Provide the standard property map series at scales in the range of 1:500 through 1:10,000, dependent on land value.

(2) Design and administer the standard means of map update from data available at the Deeds Registry Office.

(3) Perform the actual data processing, storage, and retrieval of various land records for local jurisidictions desiring to enter into such an agreement with the State.

(4) Administer and coordinate the services, and prorate the cost to local jurisdictions.

The National Geodetic Survey of the National Ocean Survey provides the vertical and horizontal coordinate networks in the United States, but it is the State's responsibility to provide the control densification and large-scale mapping. Perhaps if more States had a surveying and mapping office the need for so many different geocoding systems may not have become necessary.

TOWARD THE CADASTRE

As a result of many national and international conferences during the past decade, coordinates are used as point identifiers and the records of individual land parcels are used as building blocks for area identifiers. The pairing of these two least-common denominators, coordinates and parcels, now forms the foundation for the information systems of many disciplines. The need for coordinates, the need for parcel-level information, and the need for land title reform, stated in 1974 by Chief Justice Warren Berger, "The point I seek to make is that the basic system of real estate titles and transfers and related matters concerning financing and purchase of homes cry out for reexamination and simplification," form a natural triad.

These needs are leading to the realization in the United States that the time has arrived to develop a register of land holdings. In Europe this register is called a cadastre. Webster defines a cadastre as "an official register of the quantity, value, and ownership of real estate, used in apportioning taxes." This apportionment of taxes is referred to by the Europeans as the fiscal cadastre. Similarily, a juridical cadastre and an environmental cadastre are also maintained by them.

The function of a juridical cadastre is the registration of the following:

deeds or titles depending on local legislation
easements
covenants
subdivisions or mergers
appeasements
new private and public service works
liens
lease holds
privileges of governments

In the United States, the primary statutory laws affecting the record of rights are at the state level. With the passing of the Uniform Land Transaction Act, the trend probably will be toward Federal regulation which will prescribe the means for land registration in local jurisdictions.

The environmental data files of the future will provide:

(1) geographic land-use information

(2) information that will greatly assist the planning function and cope with urban land expansion.

(3) long-range developement plans, and

(4) information necessary to meet the needs of the people within the local communities in such vital areas as housing, education, health services, and recreation.

The same information that is used on the local level can be aggregated and analyzed by State and regional agencies to provide the necessary State-wide policy. The data will also assist Federal agencies with their national growth policies which are of prime importance due to the diminishing of natural resources. The potential of these files will be for land resource management. Without the needed environmental data, there cannot be intelligent use of the resources and without intelligent use, man cannot stabilize the necessary balance between himself and these resources. For this reason, these cadastre files will be of value to the national interest.

GEOGRAPHIC CADASTRE

A responsibility of the geographic cadastre--providing the necessary geometry--is one of the functions of the National Geodetic Survey. The geometry, the set of points (nodes and vertices) connected by a network of curved or straight lines (links), defined by coordinates, will provide the integration and communication between the other cadastres. It will describe what is being dealt with--land.

The geographic data base will contain distances, angles, and positions (observed and computed) that will have the capability of describing and positioning parcel boundaries. The input will come from ground surveys, photogrammetric surveys, and deed descriptions.

The distinguishing characteristic of the geographic cadastre is that all observables utilized in cadastral operations are related to one integrated data base. These include all types of bearings (grid, true and magnetic) as well as measures for distances (chains, paces, etc.) that are found in parcel descriptions.

For the same reasons that numbers are essential for so many applications in today's society, names have become unsatisfactory for indexing our land records. So many different organizations exercise authority over a region that a more uniform parcel coding method must be developed that lends itself to computer searches.

DEED VERSUS TITLE REGISTRATION SYSTEM

The possibility also exists that the automation and modernization of land-related records may be accomplished by establishment of a title registration system, following the torrens model. In the torrens system, the title to the land is registered. This differs from the deed recording system where only the evidence (the instrument of conveyance) of title is recorded. Under torrens, the land is conveyed by registration of the title in the appropriate local government office. The merits of the torrens system are:

(1) Certification of guaranteed title by governmental authority and establishment of insurance funds for payment of loss suffered from unjustified certification.

(2) The economy of land transfer subsequent to initial registration.

(3) Certification of titles following resolution of competing claims.

In the United States, the torrens system has lacked support and title insurance continues to be favored. Its slow growth contrasts sharply with the tremendous expansion of competing private businesses, such as title insurance and abstracting. Seven of the ten provinces in Canada are on the torrens system. There are presently twelve States that have implemented voluntary torrens registration.

There are two parts to the torrens system: title registration and map registration. Basically map registration requires a new survey, a survey review, a map check, and the adjoining maps checked for agreement. The torrens system is often referred to as the system of guaranteed boundaries, whereas a deeds registry system is referred to as the system of general boundary, or often referred to as the system of uncertain boundary. The torrens system integrates survey accuracy with title history.

If this title registration method is elected, then precise boundary location will also become the function of an established surveying and mapping office. The assignment of coordinates to vertices is being advocated by the American Bar Association to accomplish this precise boundary delineation.

SUMMARY

In conclusion, the cadastre will be a public information system in the most general sense. As such our first concern must be to evaluate the nature and quality of the information required by society and to develop the most economic and effective solution. NGS believes a solution may be the integration of this information.

(1) Graphic Data

Maps
Aerial Photography
Plats and Plans

(2) Numeric Data

Assessors Records
Ground Survey Data

(3) Descriptive Data

Deeds
Other rights and restrictions, as previously described.

The cadastre, by maintaining the accuracy of these source documents, permits addressing very accurate data needs (e.g., for detailed engineering) as well as grouped data needs (e.g., for regional planning). This solution not only makes more needed data available, but provides it at one location. In addition all necessary data are aggregated to produce indefeasible title and description of land.

An entire new approach needs to be undertaken into the logistics and administration of land-related data at the local level, keeping in mind the utilization of these data by the State and Federal Governments. It will be necessary to identify those functions which belong to the States and those which are the responsibility of the Federal Government. Costs for implementing and maintaining these programs will be another area requiring mutual agreement. New legislation may be needed to provide the legal base for this modernization of land records.

APPENDIX I. SUMMARY OF GEOCODING SYSTEMS AND REGIONS

National Bureau of Standards Federal Information Processing (FIP) Codes

FIP State Codes (50) - 2-digit alphabetical code
FIP County or Equivalent (City) Code (3141) - 3-digit alphabetical code within states
FIP Standard Metropolitan Statistical Areas (SMSA) Codes (267) -
SMSA is a region of counties with one city of at least 50,000 - 4-digit code
FIP Congressional District Codes (435) - changes with each decennial census - 2-digit code within states

General Services Administration (GSA) City Coding System

4-digit number assigned to any "populated area having a recognized entity and geographical boundary" (about 33,000 locations)

IBM City Code and Population Code

5463 cities with a population of 2500

Dun and Bradstreet City Code

44,000 locations in United States plus the many outside the U.S.

American National Standards Institute (ANSI) "Place" Code

ANSI is about to publish "place" codes which will eventually become a FIP publication about 130,000 in U.S. - not necessarily populated - 5-digit code

The Bureau of Census uses the following divisions of the U.S.

4 Regions - no codes
9 Divisions - no codes
121 Economic subregions (ESRs) - 3-digit numeric code
509 State Economic Areas (SEAs) - 2-digit numberic code (for non-SMSA SEAs)

Approximately 28,000 Minor Civil Divisions (MCDs) - 3-digit numeric within states
Approximately 8,000 Census County Divisions (CCDs) 3-digit numeric within states
Approximately 18,500 Incorporated Places - 4-digit numeric 78,269 governmental units made up of:

23,885	special districts (perform a single function as fire, sewerage, etc.)
15,781	local school districts
18,517	municipalities
16,991	townships
3,044	counties
50	states
1	federal
78,269	TOTAL

2 (at present) Standard Consolidated Areas (SCSs) 1-digit alphabetic code
Unknown number of Wards:
252 Urbanized areas - 4-digit numeric code
150 Central Business Districts (CBDs) - defined as the sum of tracts

Approximately 2,000 Major Retail Centers (MRCs)

Approximately 32,000 tracts within SMSAs and 2,600 tracts outside SMSAs - 4-digit numeric code (unique only within SMSA) plus 2-digit numeric suffix used in tract splits

Approximately 250,000 Enumeration Districts (EDs) - 4-digit numberic code within counties plus 1-digit numeric suffix used to indicate splits

Unknown number of block groups - 1-digit numeric code unique within tracts

Unknown number of blocks - 3-digit numeric code unique within tracts

Miscellaneous Districts used for specific applications defined as the result of various census:

Foreign trade statistical areas
Water locations
Industrial water usage regions
Fishing regions
Petroleum regions
Lumber industry regions
Regional marketing areas
Oil and gas districts
Standard location areas for Office of Civil Defense
Production areas and market areas

GSAs National Location Code (NLC)

7 digits - Region, State, Area, County - RSAC Code (See Below)
4 digits - Standard Location Area - SLA Code (See Below)
13 digits - Geodetic latitude and longitude to nearest second

24 total alpha-numeric digits

RSAC Code

1 digit - Standard Federal Region (10 regions)
1 digit - State within region (or state-equivalent)
1 digit - SMSA or non-SMSA SEAs
3 digits - first 3 digits of SMSA number of 3-digit SEA number
1 digit - County identifier within the SMSA or non-SMSA SEA

7 digits TOTAL

SLA Code

4 digits - tract number in tracted counties, or
- ward number in cities over 25,000 if available, or
- ED cluster if ward number not available

Standard Point Location Code (SPLC)

Designed for transportation by American Trucking Association (ATA) and Association of Americn Railroads (AAR), and consists of nested geogrphical units

1 digit - region - clusters of states
1 digit - section - state or portion of state
1 digit - county group - clusters of counties
1 digit - county or portion thereof
1 digit - 100 square-mile units
1 digit - 10 square mile units

The AAR lists 60,000 locations
The ATA lists 110,000 locations

Approximately 800 manufacturers and carriers have subscribed to the SPLC system and a few havew altered code to meet their individual needs.

PICADAD System - developed for Census of Transortation

PI stands for Place Identification
CA stands for Characteristics and Area
DAD stands for Distance and Direction

One file of 37,000 places which are "keyed" to a file of the 5,700 positioned points used for transportation analysis.

Federal Highway Administration Network - Department of Transporation (DOT)

DOT has generated a computer file of coordinates of population centroids for each county and a network of 11,500 links between the centroids. A node-link network has also been developed for highways, rails, waterways, airways, and pipeline. Also 533 transportation zones hve been delineated and a coordinate of the centroid assigned. Zones are defied with the boundaries of the 267 SMSAs where they exist.

Zone Improvement Plan (ZIP) - implemented by the U.S. Postal Service

1-digit - identifies one of 10 regions
2-digits - identifies sections within regions. Sections were determined by transportation patterns - one central post office per section
2-digits - identifies a group of city blocks or small rural area served by a single post office

Water Resources Council Geographic Areas

21 regions
205 subregions
350 accounting units (proposed)
1600 county groups (proposed)

Office of Managment and Budget (OMB)

10 Standard Federal Regions were established in 1969

Economic Development Administration (EDA)

3 regions for administration
120 district within these regions have had their development programs approved by EDA.

Bureau of Economic Analysis (BEA)

173 city centered geographic economic areas have been delineated.

Interstate Commerce Commission

5 major freight rate territories

Institute for Defense Analyses

Uses county centroids for node analysis involving population vs. nuclear attack targets vs. transporation routes

American Telephone and Telegraph

Computes utility mileages from coordinates based on the Donald Elliptic Projection

Railway Express

Overlays arbitrary grid on U.S. for zoning

Global Reference Code

This code identifes a location within a two-minute square of latitude and longitude within six levels of grids covering the globe. It uses 12 digits for the grid-in-grid system.

Linear Geographic Code

This code identifies a location within a 36-second square of longitude and latitude within a hierarchy of five levels of grids covering the globe. It uses 10 digits.

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THE ROLE OF ERTS AND SKYLAB INFORMATION IN THE MEXICAN WATER PLAN

by

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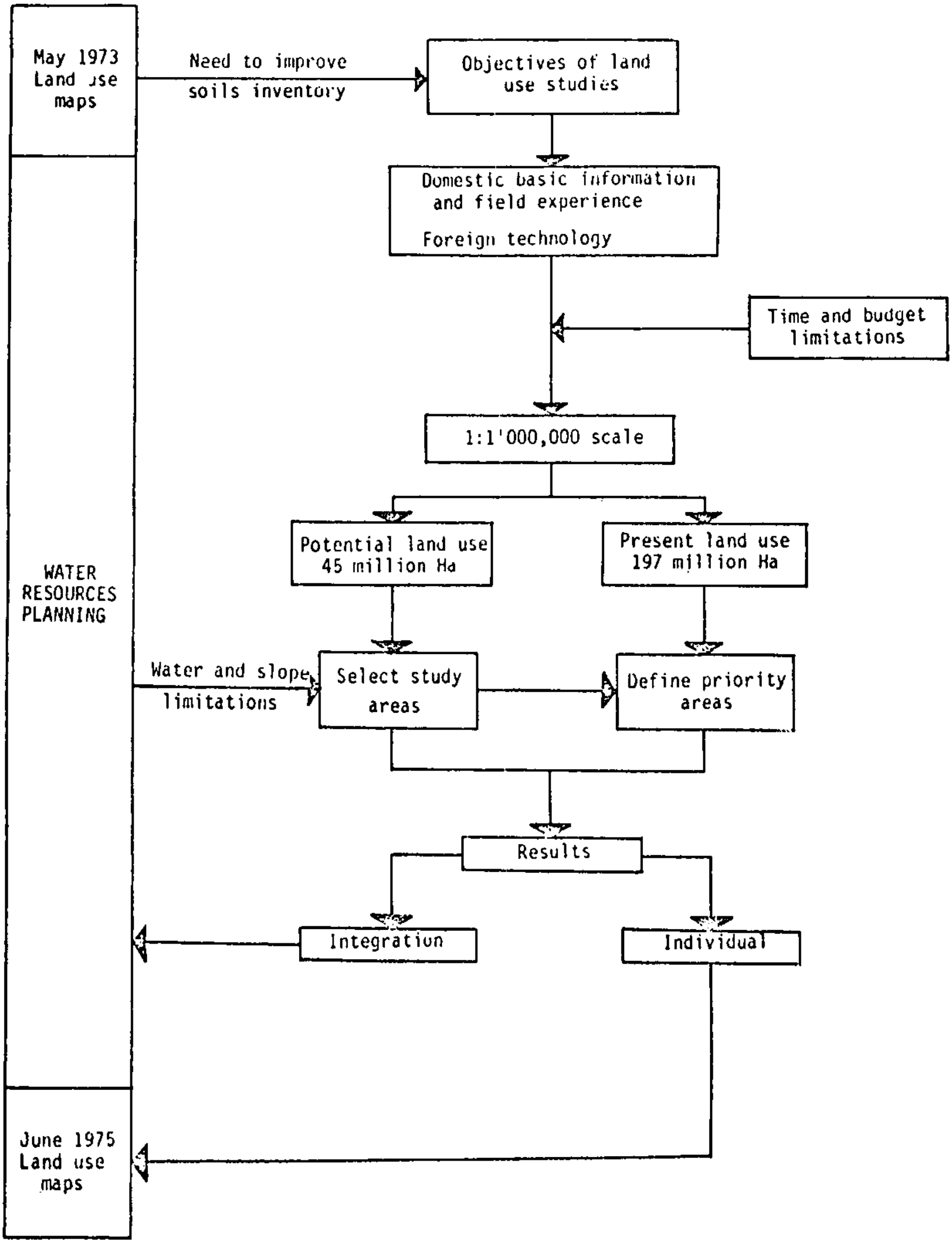
Stages of Mexican soil studies as an input to water resources planning



ABSTRACT

ERTS and SKYLAB satellites were useful to the Mexican Water Plan (MWP) in providing basic data regarding present and potential land use. These data were not available on a national level and could not have been obtained in a short time by any other means.

Funds were provided by the Mexican government and by the United Nations. The World Bank was the executing agency for UNDP fund and assisted also in selecting and hiring foreign personnel.

Studies of present land use covered the entire country (197 million hectares). Land uses were mapped according to the first level classification of nine categories recommended by the U. S. Geological Survey. In addition 6.3 million hectares of land with advanced erosion were detected. Work was executed at a rate of 8 million hectares per month; reliability was 90% and the cost only 0.1 cents/hectare.

Potential land use was studied for 45 million hectares at a rate of 4 million hectares per month and a cost of 0.33 cents/hectare. Soil units were delineated according to the FAO classification at a scale of 1:1 million; interpretive maps were prepared to show potential agricultural productivity, carrying capacity for cattle, water erosion risk, and slope.

INTRODUCTION

The Mexican Water Plan (MWP) had been instituted by late 1972 with the main objective of developing systematic water resources planning procedures for the country. The Mexican Government had made an agreement with the United Nations Development Program for the use of foreign expertise when needed. At the same time, it made a commitment to share the WMP's experiences with other countries. One fourth of the 4 million dollar project was provided by the UN, and the rest by the Mexican Government. The World Bank provided assistance in selecting and hiring the foreign experts. IPESA, a Mexican consulting firm conducted the study.

The socio-economic development creates increasing water demands, mainly for agriculture, domestic and industrial water supply, power, etc.

Studies already made indicate that national demands are less than available water resources. On a regional basis, however, the demands will exceed the run off during this decade.

In order to decide where and when to open new lands for cultivation, we need inventories of present land use and soil capacity for the entire country.

METHODS

The MWP staff defined objectives and sketched procedures for a 33 month long study of present and potential land use. Two seminars were conducted in Mexico City to refine procedures for using ERTS-1 imagery.1/ Highly experienced U. S. Soil Conservation Service scientist and two leading U. S. experts in ERTS imagery participated in the seminars. A short training course on image interpretation for present land use was given by one of the remote sensing experts. The other ERTS expert, the soil scientist, and a soil scientist from another Mexican Consulting firm conducted two pilot studies for investigating land use potential.

The foreign experts made six one week trips to Mexico during the following year, functioning as a review team. The leadership and responsibility for the study were held by Mexican Soil scientists.

Soil inventories were one of the main inputs to both present and potential land use studies. Water requirements for irrigated agriculture accounts for more than 95% of the country's total water consumption. Therefore, soil inventories are as important as water inventories for successful water resources planning.

When MWP project was started, only the FAO soils map was available for the whole country. Unfortunately the FAO soils map is too general to be useful for water resources planning. Many local soil studies covered small areas. Present and potential land use maps at a scale of 1:50,000 are being developed by CETENAL (Commission de Estudios del Territorio Nacional). CETENAL maps are excellent but cover less than a third of the country at the present time.

Present land use was surveyed for the entire country and a map prepared at a scale of 1:1 million. Areas where potential land use studies would later be carried out were given first priority. The entire country was mapped in a two year period at a cost of two hundred thousand (U. S. dollars, i.e. 0.1 cents/hectare).

The potential land use study was started six months after the present land use study because present land use information was needed as input. Areas were selected from regions with less than 10% slope where water is still available for agricultural developement. More than one fifth of the country was studied in a year and a map produced at a scale of 1:1 million.

The cost was roughly 150 thousand dollars; i.e. 0.33 cents/hectare. As a by-product of the water resources planning, land use studies have improved the Mexican soil resources inventories. The development of techniques for the integration of present and potential land use studies contributes significantly to the regional water resources planning process.

A summary of results of both studies, as well as their integration is being prepared. Finally, to accomplish the commitment to the United Nations, outline of a step-by-step handbook for land use studies using ERTS imagery, is under preparation.

1/ Garduno, H. Garcia Lagos, R., Garcia Simo, F. y Perez Gavilan, D., UTILIZACION DE LAS IMAGENES DEL SATELITE ERTS-1 EN LA PLANEACION DE LOS RECURSOS HIDRAULICOS, Primer Congreso Panamericano y Tercero Nacional de Fotogrametria, Fotointerpretacion y Geodesia, Mexico, 1974.

PRESENT LAND USE STUDY

The objectives of this study were:

1) To survey present land use for the whole country. Special emphasis was placed on irrigated agricultural land and on rainfed areas.

2) To provide basic information to determine potential land use.

False infrared color transparencies using channels 4, 5, and 7 - from ERTS were used. The U. S. Geological Survey First Classification level recommendations1/ were followed with slight modifications to interpret the 200 images that cover the country. Since agricultural land was the main interest of the study, images taken during the dry and rainy seasons were used. This made it possible to discriminate irrigated from rainfed agriculture.

The project included intenstive low altitude flights, ground-truth trips, and comparison with detailed 1:500,000 CETENAL maps. Table I - - shows the reliability for each land use.

In southeastern tropical Mexico Skylab IR color photographs were used where cloud-free ERTS images were not available. Since only visual interpretations techniques were used, those areas densely covered by vegetation were especially difficult to interpret. Future efforts will also utilize methods which take advantage of computer-aided spectral analysis.

TABLE I. RELIABILITY OF THE PRESENT LAND USE STUDY RESULTS

Land Use	Code	Minimum mapping Unit	More common erroneous interpretations	Reliability %
Irrigated cropland	(1)	50 Ha	(2) and (3)	95
Flat rainfed cropland	(2)	100 Ha	(1) and (3)	90
Steep rainfed cropland	(3)	150 Ha	(1) and (2)	85
Range and grassland	(4)	250 Ha	(3) and (7)	85
Woodland (conifer and hardwood)	(5)	250 Ha	(6) and (7)	90
Tropical forestland	(6)	250 Ha	(5) and (7)	85
Shrub/Scrab land	(7)	250 Ha	(5) and (6)	85
Barren land	(8)	300 Ha	(4)	85
Wetland	(9)	200 Ha	(1)	95
Water bodies	(a)	50 Ha		98
Urban areas	(u)	100 Ha		85
Erosion	(e)	300 Ha		95
Average				90

1/U.S. Geological Survey, A LAND CLASSIFICATION SYSTEM FOR USE WITH REMOTE-SENSOR DATA, Geological Survey Circular 671, Washington, D.C. 1973

Note 1. The two uses more intensively field checked were irrigated and rainfed cropland.

Note 2. Water bodies and urban areas were easily identified.

RESULTS

The final results are now being printed in 17 1:1,000,000 scale land use maps (Fig. 1). Each map includes a detailed description of each use, taking into account regional differences. A grid formed by squares of one half degree latitude and one half degree longitude was superimposed onto the maps. Statistics were calculated for each land use on the basis of state, MWP regions and individual charts. The area showing advanced erosion was computed for each use. Fig. 2 shows a generalization of results for the whole country.

Comparison with census figures is difficult due to differences in land use definitions. It is interesting, however, to point out that while census data indicated that an area of 30.0 million Ha is severely eroded, the present land use study detected only 6.3 million Ha of land with advanced erosion.

DURATION, MANPOWER, AND COST

Figure 3 shows the schedule of activities for the present land use study. Once the procedure was defined and the interpreters trained, a set of straightforward steps was followed for each of the 17 land use maps. A major change in methodology was made when ground truth proved the need for a far greater effort than was at first anticipated.

Table II shows the manpower used in the study and Table III shows estimated costs. The reported total cost of $200,000.00 (U.S.) does not cover air checking or the expenses of foreign personnel.

TABLE II. MANPOWER REQUIRED FOR PRESENT LAND USE STUDY

Mexican personnel	Man months
Project manager	26
MWP project coordinator	3
Image interpreters	112
Chartographic support assistants	29
Assistants land use area estimation	90
Draftmen	64
	329
Foreign personnel	
Remote sensing expert	1

TABLE III. COST OF PRESENT LAND USE STUDY

Image and photographic material	8%
Image interpretation	35%
Chartographic support	7%
Ground truth	15%
Land use area estimation	22%
Drawing and reports	13%
	100%

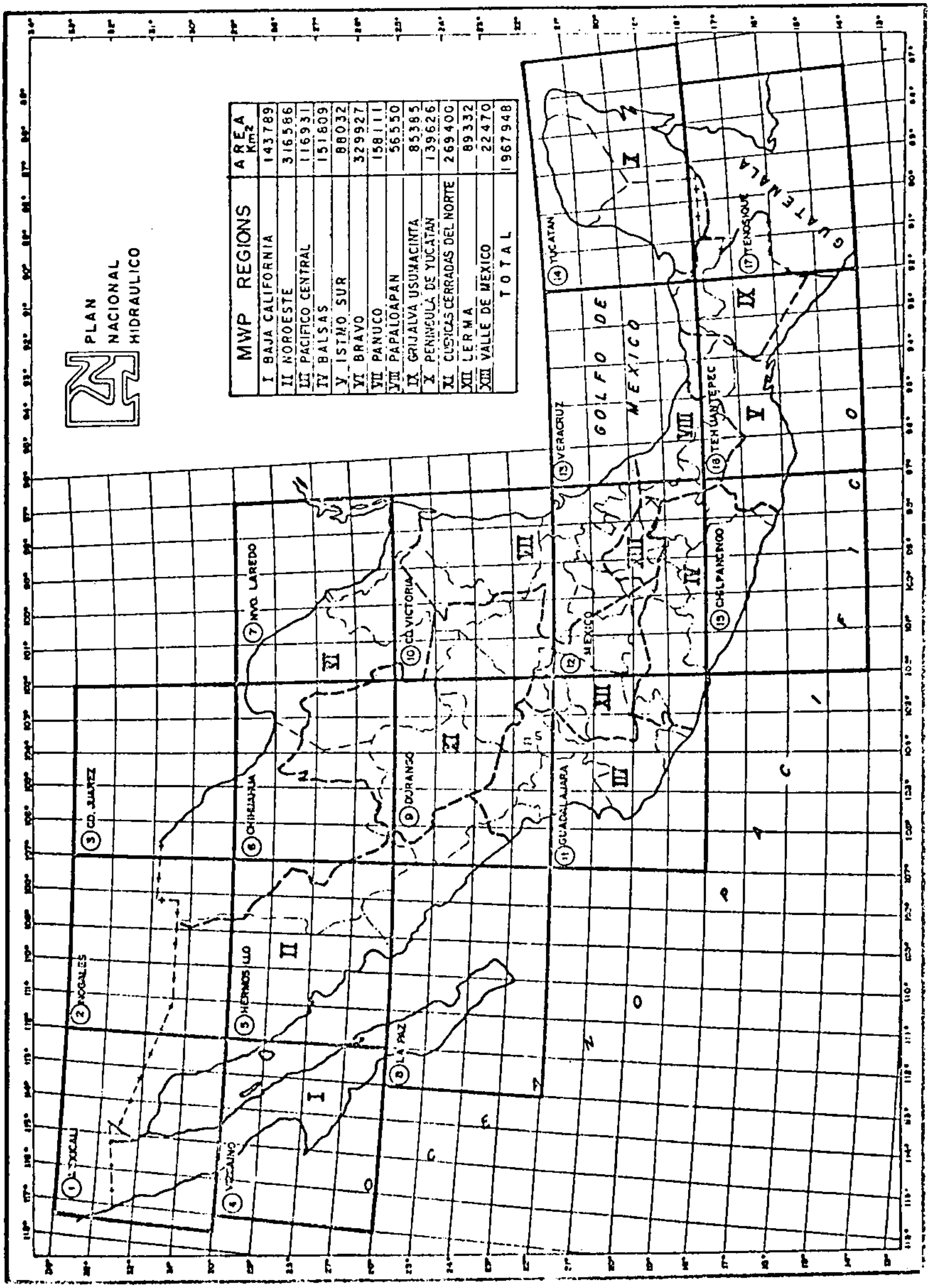
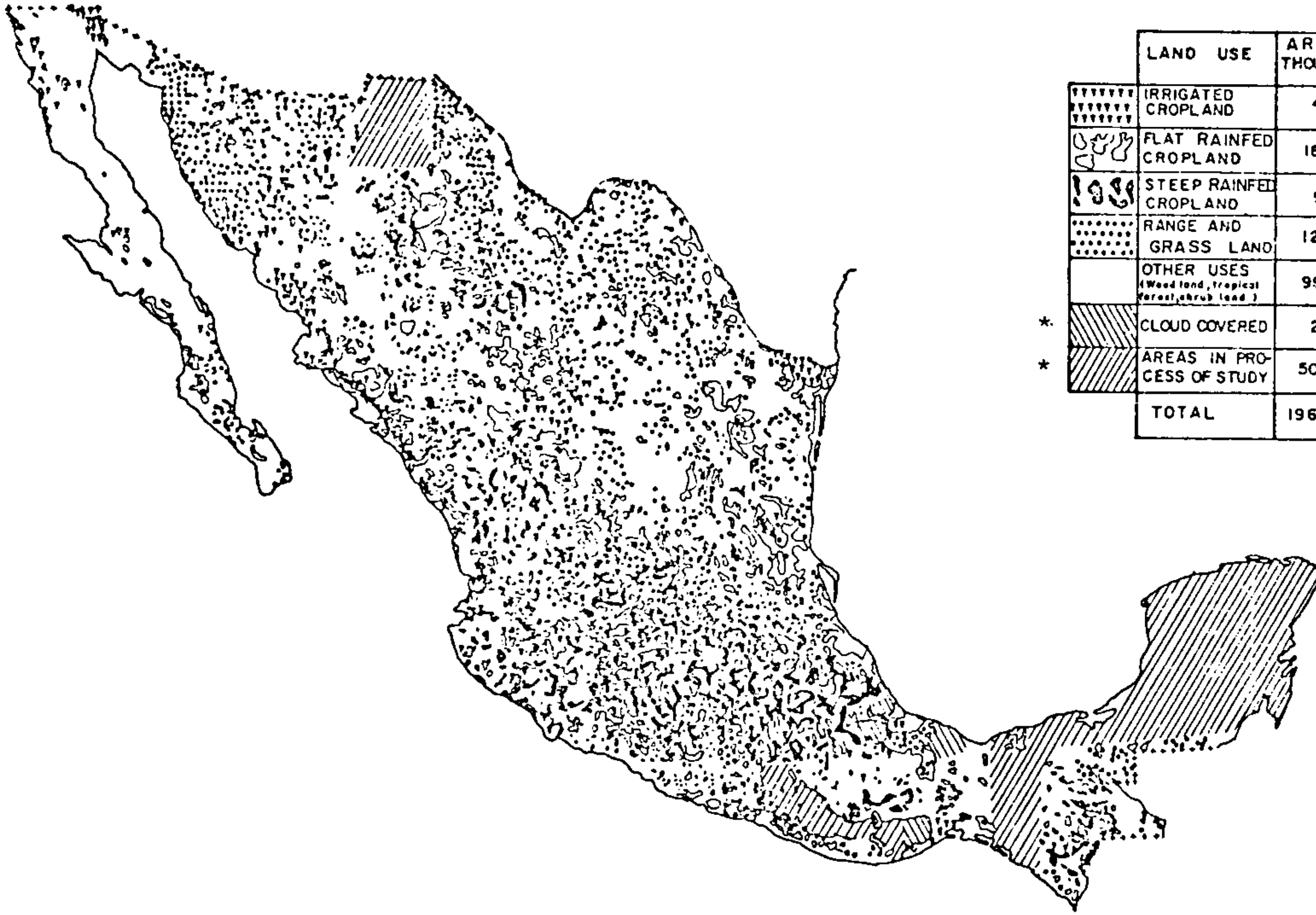


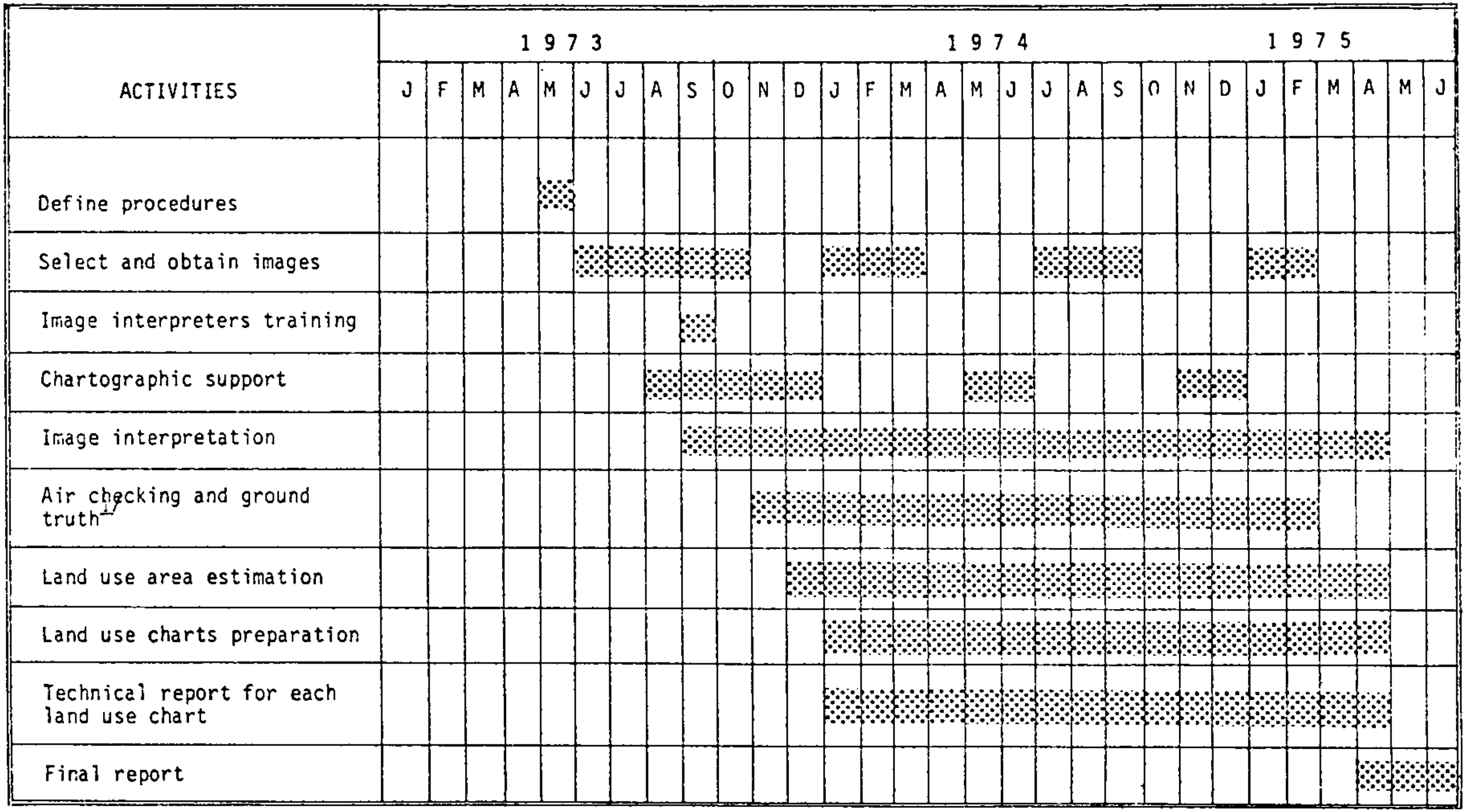
Fig. 1 Distribution of charts for the present land use study



	LAND USE	AREA IN THOUSAND Ha	% OF TOTAL TERRITORY
	IRRIGATED CROPLAND	4,178.91	2.12
	FLAT RAINFED CROPLAND	18,050.14	9.17
	STEEP RAINFED CROPLAND	9,935.88	5.05
	RANGE AND GRASS LAND	12,290.50	6.24
	OTHER USES (Wood land, tropical forest, shrub land)	99,470.0	50.07
*	CLOUD COVERED	2,875.36	1.47
*	AREAS IN PROCESS OF STUDY	50,913.13	25.88
	TOTAL	196,713.92	100.00

Fig. 2 Present land use study in Mexico

* Areas already finished
Oct. 1975



1/ A total distance of 13,500 Km was covered by plane. Ground truth covered 12,500 Km, sampling 4,000 points.

Fig. 3 Schedule of activities in the present land use study

POTENTIAL LAND USE STUDY

Description

The objective of the study was to assess, at an identification and not a project level, areas with high, medium or low potential agriculture and pasture productivity and water erosion risk.

Since the project also aimed at the development of methodology to be used, two pilot studies covering 6 million Ha were first carried out, one in a semi-arid and the other in a humid tropical area. The areas covered were of a reasonable size and the ecological conditions variable enough to assure that the methodology could be successfully applied in any other region of the country.

Interpretation of infrared false color and channel 5 images was made using transparencies and prints at 1:1,000,000 and 1:1,500,000 scales. Final results were produced at a 1:1,000,000 scale. Overlays with general delineations of present land use, geology, rainfall, FAO soil units and infrastructure were used to help image interpretation. More reconnaissance flights and ground truth trips with more intensive sampling were required than for the present land use study.

Initial soil units classification was taken from the 1:2,000,000 scale FAO map, but a far more detailed soil units map was produced with the aid of ERTS images, the overlays mentioned above, air reconnaissance and ground truth. The potential land use classification was made according to the Handbook 210 of the U.S. Soil Conservation Service.1/

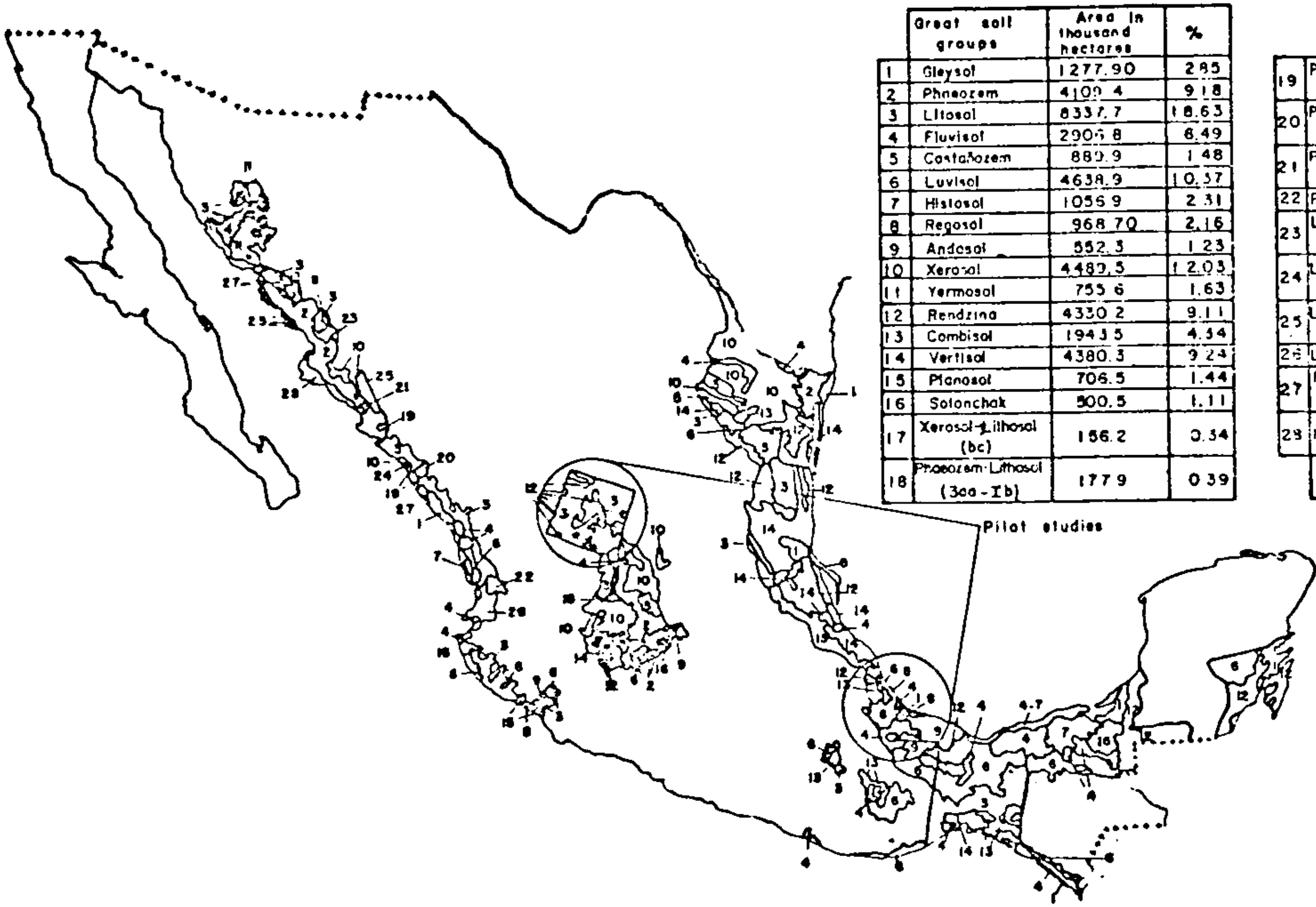
Finally, interpretive maps were produced showing potential agricultural and pasture use, slope classification, and water erosion risk. A reliability of 80% to 90% was determined by comparing results of the study with more detailed results of conventional soil surveys.

RESULTS

Figure 4 shows the soil unit delineations obtained following the FAO soil classification system. Image interpretation was the primary means of refining the soil unit boundaries and producing greater detail than shown on the original FAO map showed.

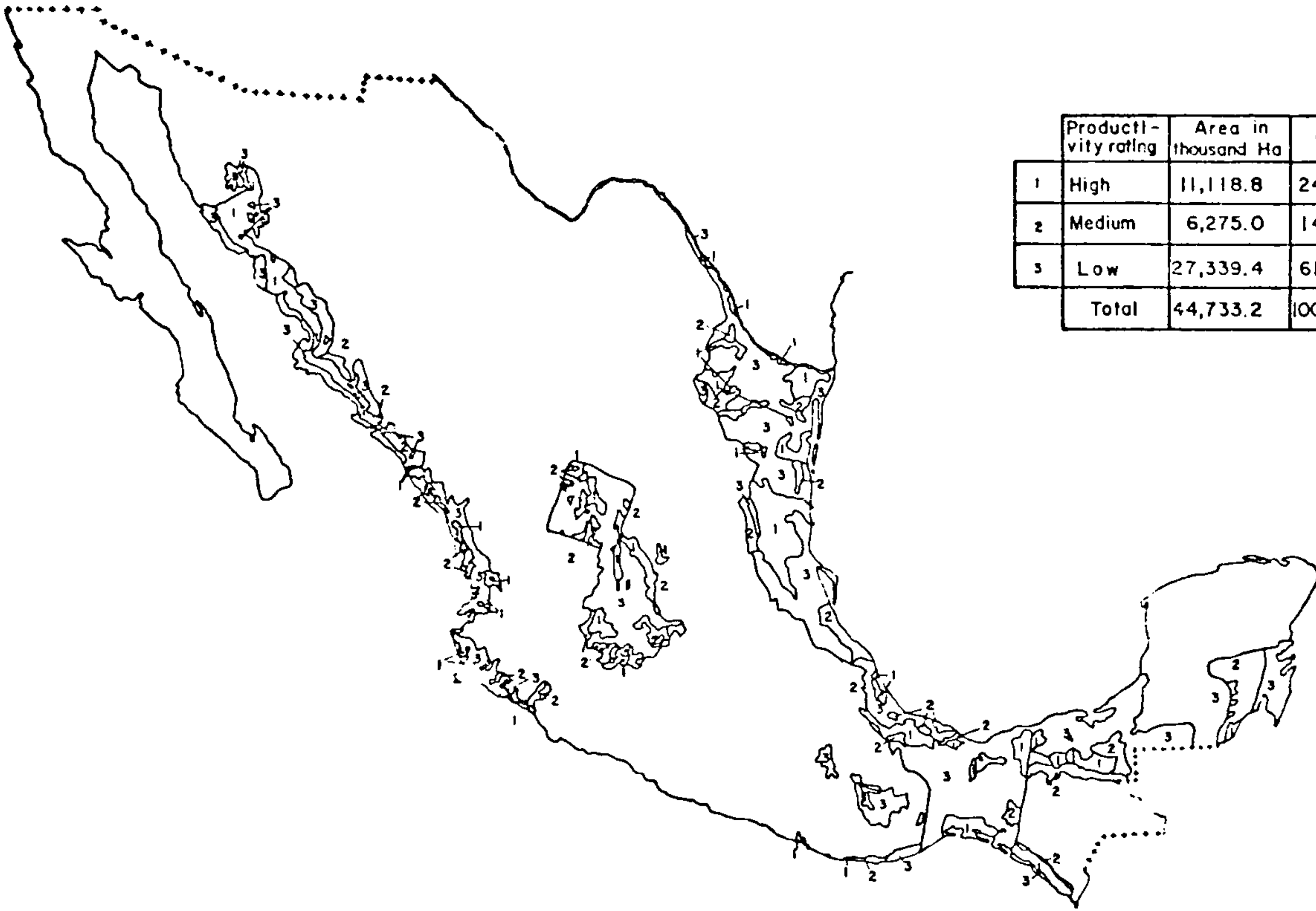
Potential land use maps were based on the FAO soil units, yield and production statistics, results from agriculture and livestock experimental stations, field observations, and personal experience of the soil scientists that carried out the study. Estimates were made for each soil unit of yields for the most important crops and for carrying capacity under grass for cattle. Figure 5 shows general results for agricultural productivity.

Only 17 million Ha were found to have high and moderate potential agricultural productivity. This figure seems low, taking into account that the study was carried out in selected areas according to slope. However, large areas in the southeast were classified as of low productivity due to



	Great soil groups	Area in thousand hectares	%
1	Gleysol	1277.90	2.85
2	Phaeozem	4109.4	9.18
3	Litosol	8337.7	18.63
4	Fluvisol	2905.8	6.49
5	Castañozem	889.9	1.48
6	Luvisol	4638.9	10.37
7	Histosol	1055.9	2.31
8	Regosol	968.70	2.16
9	Andosol	552.3	1.23
10	Xerosol	4489.5	12.03
11	Yermosol	755.6	1.63
12	Rendzina	4330.2	9.11
13	Cambisol	1943.5	4.34
14	Vertisol	4380.3	9.24
15	Planosol	706.5	1.44
16	Solonchak	500.5	1.11
17	Xerosol-Lithosol (bc)	156.2	0.34
18	Phaeozem-Lithosol (3aa-Ib)	177.9	0.39
19	Phaeozem-Lithosol (3a-(b)	177.0	0.39
20	Phaeozem-Lithosol (3a-(c)	151.0	0.33
21	Phaeozem-Lithosol (2a-(bc)	104.4	0.23
22	Phaeozem-Vertisol	134.6	0.30
23	Lithosol-Phaeozem (b-3a)	310.9	0.69
24	Lithosol-Phaeozem (c-3a)	225.0	0.50
25	Lithosol-Phaeozem (c-2b)	331.0	0.74
26	Lithosol-Luvisol	458.2	1.02
27	Fluvisol-Vertisol ---	57.0	0.12
28	Regosol-Solonchak	605.2	1.35
	Total	44733.2	100.00

Fig. 4 - Potential land use studies in Mexico
Great soil groups and associations
Preliminary results



	Productivity rating	Area in thousand Ha	%
1	High	11,118.8	24.85
2	Medium	6,275.0	14.04
3	Low	27,339.4	61.11
	Total	44,733.2	100.00

Fig. 5 - Potential soil productivity for agriculture

seasonal flooding. It is believed that with adequate flood protection and drainage measures, they could easily be considered as areas of potential high productivity since the soils are deep, flat, and fertile.

The potential range and grass productivity was determined only for 29.7 million Ha since estimates of carrying capacity per hectare were not available for the northwestern and central regions of the country. Figure 6 shows the general results. A large proportion of the area was found to have from medium to high productivity. Most of the land is located in the Gulf coastal plains and in the southeastern regions where most of the country's present livestock production is concentrated.

Finally, Figure 7 shows areas with different degree of water erosion risk. Even though the study areas lie mostly on flat lands, 69% of the 45 million Ha shows medium to high erosion risk. This fact points out the need for sound soil conservation programs and policies.

Figure 8 shows a schedule of activities for the potential land use study. Separate package studies were carried out in each selected area. An assembly-line procedure was not feasible because integration of image interpretation, basic data, field notes and personal experience requires that an enormous amount of information be borne in mind. One area had to be completed by a single team before starting studies on its next area were begun.

DURATION, MANPOWER, AND COST

Table IV summarizes manpower use. Table V shows an estimate of costs for the entire area of 45 million hectares. The reported total cost of U.S. $150,000.00 does not cover air checking nor expenses of foreign personnel.

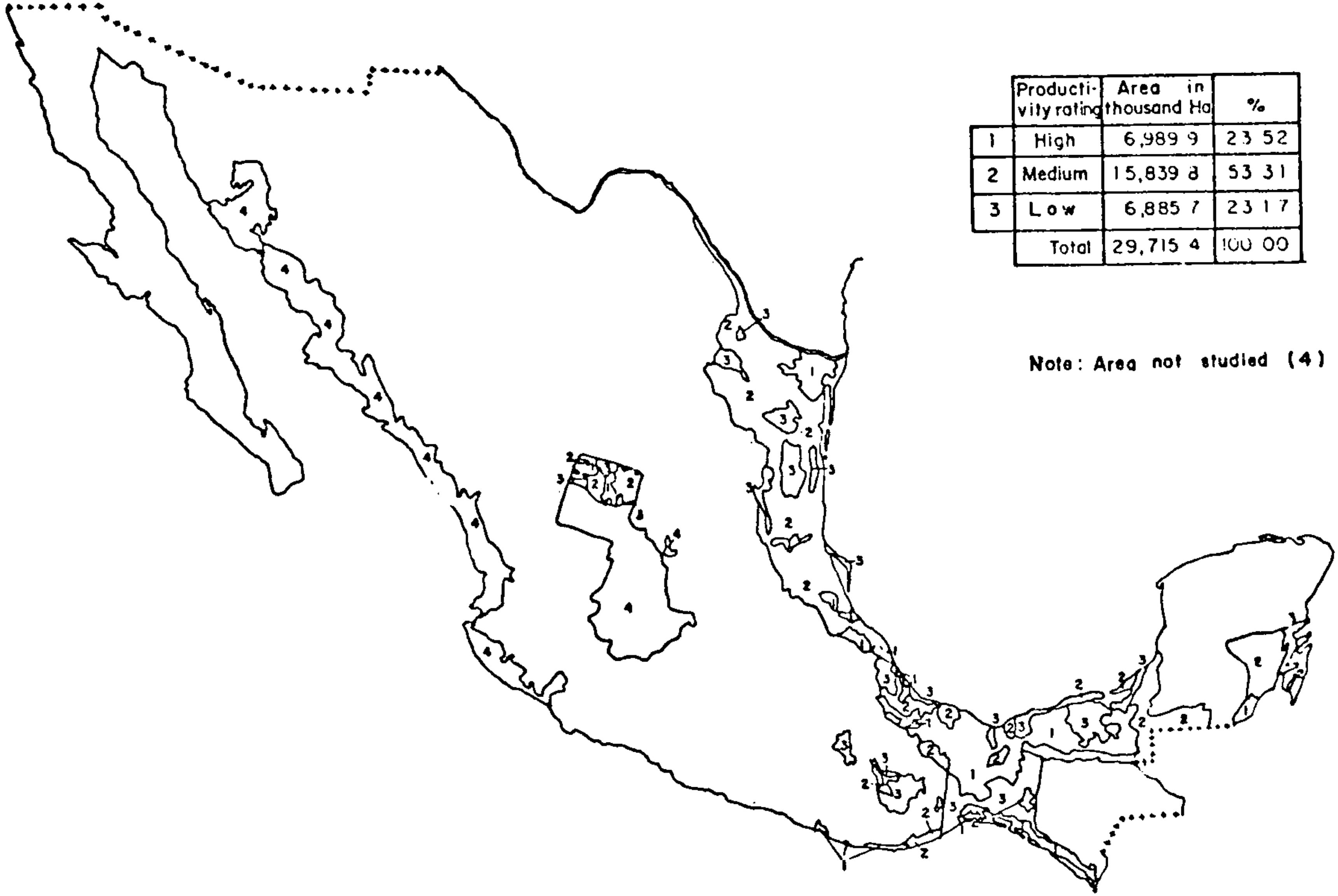
TABLE IV. MANPOWER REQUIRED FOR THE POTENTIAL LAND USE STUDY

Mexican personnel	Man months
2 Team leaders	22
2 Soil scientists	22
2 Image interpreters	22
2 Draftsmen	20
MWP	2
	88

TABLE V. COST OF POTENTIAL LAND USE STUDY

Overlay material and image prints	2%
Office work: preliminary delineation, source material analysis and final integration to obtain soil unit maps and interpretative maps	56%
Field reconnaissance trips	26%
Drawing and reports	16%
	100%

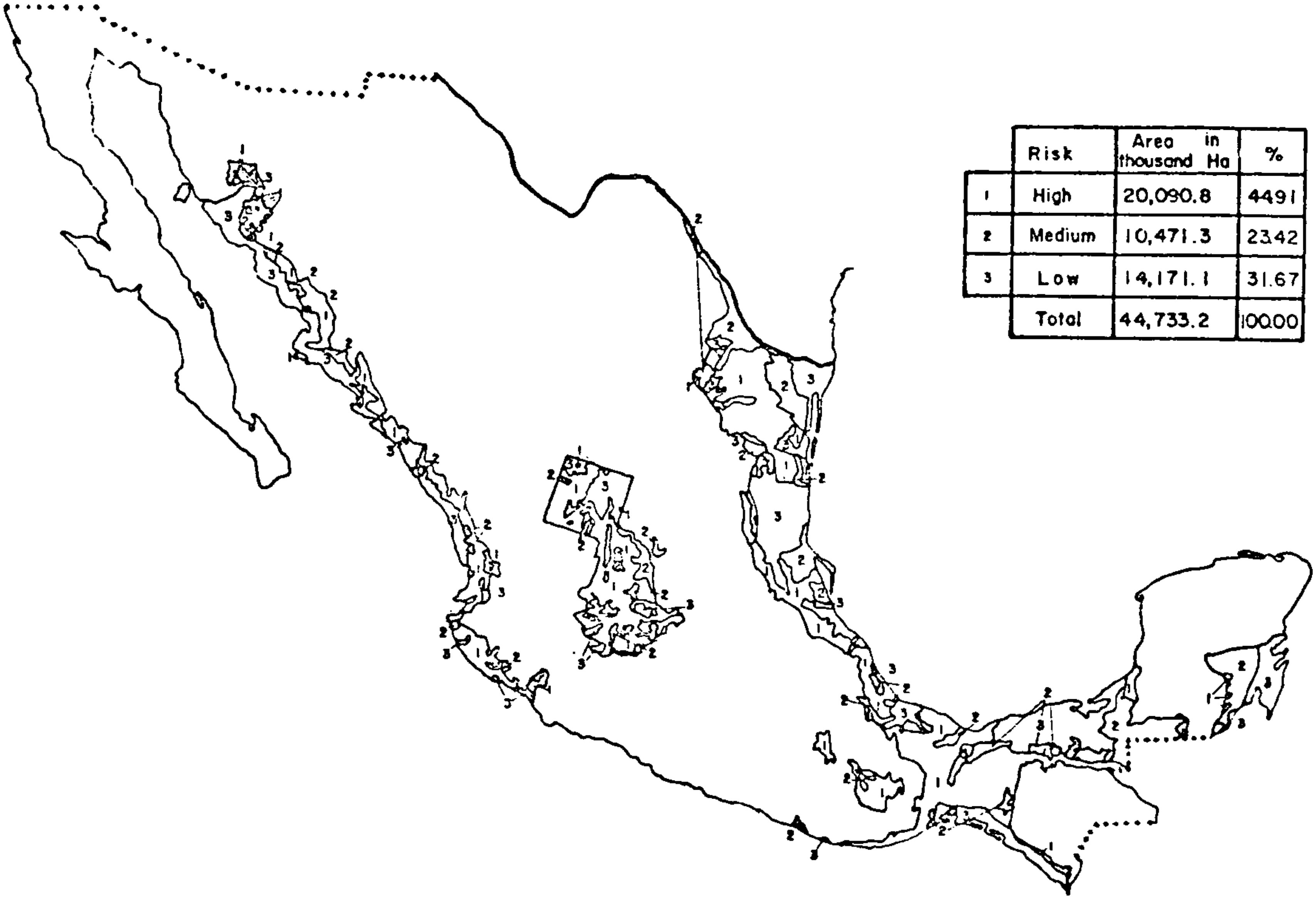
1/ Klingebiel, A.A., and Montgomery, P.H. LAND CAPABILITY CLASSIFICATION. Agriculture Handbook No. 210, Soil Conservation Service, U.S. Department of Agriculture, 1973.



	Productivity rating	Area in thousand Ha	%
1	High	6,989 9	23 52
2	Medium	15,839 8	53 31
3	Low	6,885 7	23 17
	Total	29,715 4	100 00

Note: Area not studied (4)

Fig. 6 - Potential soil productivity for grass



	Risk	Area in thousand Ha	%
1	High	20,090.8	44.91
2	Medium	10,471.3	23.42
3	Low	14,171.1	31.67
	Total	44,733.2	100.00

Fig. 7 - Water erosion risk

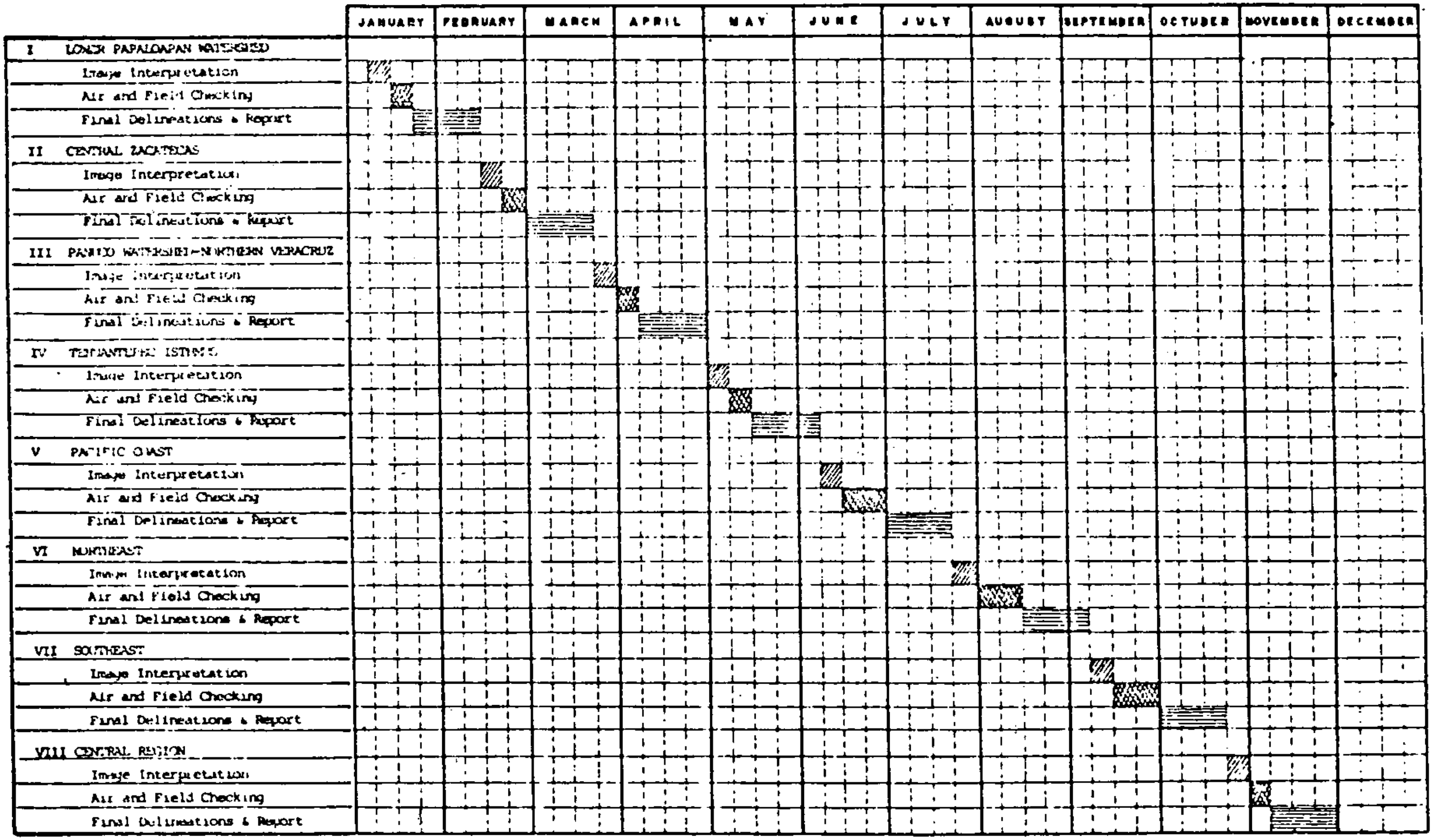


Fig. 8 Schedule of activities for the potential land use study

C O N C L U S I O N S

The comparison of present and potential land use maps shows that 82% of the land now being farmed and rated as having high potential agricultural productivity is presently under irrigated or rainfed flat land agriculture. However, there are 535 thousand Ha of low productivity areas presently being irrigated and 3.6 million Ha of low productivity areas under rainfed agriculture. On the other hand, 3.9 million Ha with high potential agricultural productivity remain unfarmed.

It is also important to point out that 41% of the 16.6 million Ha presently being used within the area studied is in high danger of being eroded. Since most irrigated land lies in areas with low risk of erosion most of this area has rainfed agriculture. This, as well as the fact that 15.6 million Ha with mediuim to high risk of water erosion are not being farmed, had been anticipated.

There are large areas in the Gulf coastal plains with low potential agricultural productivity and medium to high risk of water erosion. These areas should be mainly devoted to pasture land care taken not to overgraze them.

The integration of both present and potential land use maps is extremely useful in providing orientations for regional development of water resources in agricultural countries with scarce water supplies. It is also useful in the establishment of policies for land redistribution according to capability and water erosion risks.

In summary, the role of statellites in Water Resources Planning for Mexico was very important. A problem was identified, a classification scheme was developed, images were acquired; interpreters were selected, trained and went to work; ground data was collected and used for both training and verification; maps were made and detailed area statistics were compiled. The outstanding characteristics of this project is that precise information is available regarding the <u>accuracy</u> of the final map products (approximately 85-90%, overall), the <u>manpower</u> expended to perform the work (approximately 14 men for 18 months) - and the <u>costs</u> associated with all aspects of the work (approximately 10 U.S. cents per square kilometer).

RECOMMENDATIONS FOR FURTHER WORK

Despite the fact that the objectives are being successfully met on the Mexican ERTS-1 Land Use Inventory Project, only a small portion of the information contained in the multitemporal/multispectral satellite data is being used.

Recent advances in the areas of color compositing, digital image processing, correcting geometric distortion, ground coordinate positioning (to an accuracy within one half of a picture element), digitally overlaying multitemporal/multispectral data, band ratioing, man machine interaction, computerized image classification and multistage sampling, allow for the extraction of much of this additional information that remains hidden in the satellite data.

The next steps in using LANDSAT data in Mexico will include study of dynamic changes in land use, computer aided spectral scanning, more detailed studies of areas of interest, etc.

A C K N O W L E D G E M E N T S

Much of the information in this paper has previously been presented in the Earth Resources Survey Symposium at Houston, Texas, with the name "Present and Potential Land use Mapping in Mexico," by Hector Garduno, Ricardo Garcia Lagos, and Fernando Garcia Simo.

Messrs. Gerardo Cruickshank, Undersecretary of Planning at the Water Resources Ministry and Fernando Gonzalez Villareal, General Coordinator of the Mexican Water Plan supported and oriented the land use studies. Dr. Donald Lauer in charge of training programs at the EROS Data Center of the U.S. Geological Survey, helped in defining the present land use study methodology, trained the image interpreters and recently made an appraisal of the final results. Dr. Albert Klingebiel, one of the more experienced U.S. soil conservation scientists, recently retired from the Soil Conservation Service and Dr. Victor Myers, Director of the Remote Sensing Institute at the University of South Dakota, helped in defining the potential land use study methodology and trained Mexican soil scientists on the field. They were also responsible for assistance during the air and field reconnaissance and the reviewing of manuscripts.

Roberto Vital from MWP was in charge of preparing source material, map generalizations and integration of present and potential land use. The EROS Data Center provided excellent service in processing and sending the images that were required for the study.

GEODETIC SURVEY ACTIVITIES IN NEW JERSEY

by

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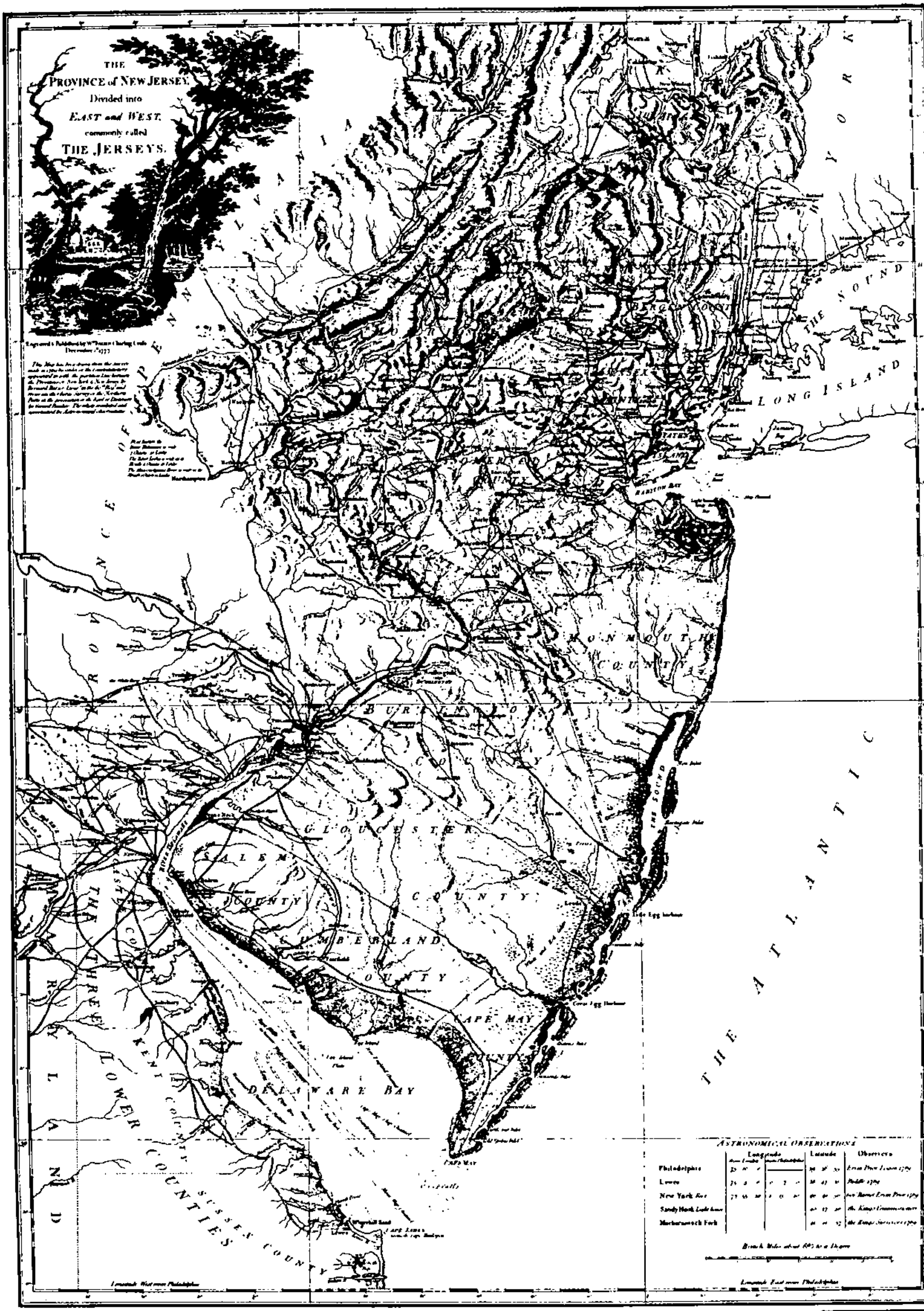

Fig. 1. Map published in 1777 of the Province of New Jersey, divided into East and West Jersey, based on Survey of Lt. Bernard Ratzer in 1769.

HISTORICAL DEVELOPMENT FROM 1807 TO 1941

The New Jersey Geodetic Survey has a long history.* It began in 1807 with the U.S. Coast Survey, headed by Ferdinand Hassler. Because of its central position along the eastern coast of the United States, New Jersey was chosen for the location of the first base line for the coastal triangulation arcs. Hassler started his triangulation and surveying in Englewood and Sandy Hook, N.J. He surveyed until his death in the fall of 1843 when he was fatally injured while protecting his instruments against a storm in the Delaware Bay.

The following geodetic monuments are presently in the process of being nominated for the national Register of Historic Places:

Springfield Triangulation Point--1817 (installed by F. Hassler)
Sandy Hook Lighthouse--1817 (destroyed but reestablished in 1836 by F. Hassler)

Both monuments are among the first 11 monuments established in the United States in 1817 and are still part of the N.J. Geodetic Network.

The New Jersey Geodetic Control Survey (NJGCS) was organized in 1854 by Gen. Egbert L. Viele, who became the first topographic engineer of the state. This Survey has been the oldest state organization for geodetic surveying in the Western Hemisphere. It began with plane tables and triangulation in Sussex County. In three years the surveying of Cape May, Monmouth, and Sussex Counties wwas completed. The services were extended to the whole state in the 1870's. All 21 counties were surveyed and mapped on a scale of 1:21,120 by 1887. Based on this survey, 17 "Atlas Sheets" (1:63,360) were prepared. This series of Atlas Sheets was altered and updated several times and is still in use. The latest revision was aided by the 1972 aerial surveys.

The present geodetic network of New Jersey, together with the state plane coordinate system, was established by Chapter 116, P.L. 1935, on March 25, 1935. In accordance with the Law, 35 geodetic crews, under the supervision of Prof. Philip Kissam, Princeton University, installed 13,500 monuments by December 1941. The crews were then called to military duty in World War II. Since then the NJGCS has had only a skeleton crew for maintenance and filing.

THE GEODETIC SURVEY FROM 1941 TO PRESENT

In the past 37 years the network of monuments deteriorated from 13,500 to 7,500. This reflects an average annual destruction of 160-185 monuments due to construction and vandalism. A full crew concentrating on the reinstallation of monuments is capable of installing not more than 100 monuments on a yearly basis. During the same period, the population of the state grew from 4,160,165 (554 persons per sq. mi.) to 7,510,000 (1,003 persons per sq. mi.). In accordance with the nationwide standards in the geodetic survey, which are based on population density, and due to the increased real estate values, New Jersey's population density called for 16,000 monuments by 1940 and 29,956 by 1977 or four monuments per sq. mi.

* Based on article: Halasi-Kun, G.J., "The Geodetic Survey in New Jersey," ACSM Bulletin, American Congress on Surveying and Mapping, Vol. 52, Nov. 1977

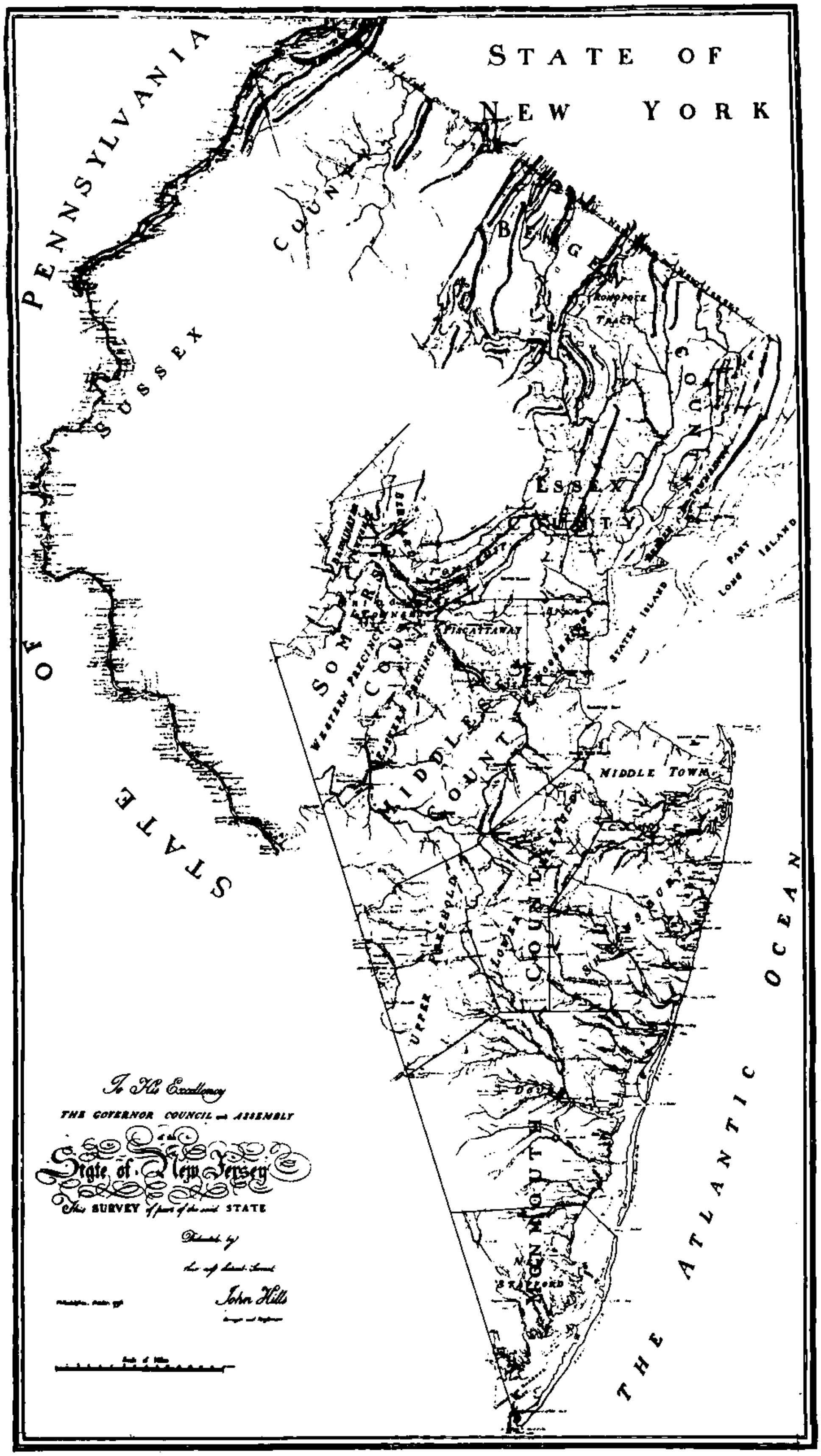

Fig. 2. State of New Jersey during the American Revolution, John Hills, Surveyor, October 1796.

It became imperative to revitalize the NJGCS because further deterioration would have made the system inoperative. Several attempts were made to get financial help from the Federal Highway Administration through the N.J. Dept. of Transportation. These attempts were made in accordance with Circular A-16 of the Executive Office of the President, U.S. Bureau of the Budget, Washington, D.C., May 6, 1967, and the Federal Highway Administration's Federal Aid Program Manual, Vol. 6, Chap. 3, Sec. 2 Subsec. 1, Aug. 5, 1974. All efforts to obtain financial help failed. Similarly, the cooperative program of the U.S. National Geodetic Survey, NOS, NOAA, for expansion of the network did not materialize due to the lack of the state's matching funds. After these possibilities proved to be unworkable, the individual county governments within the state were approached with the suggestion that a cooperative program in geodetic survey be established for each county on an individual basis with the following guidelines:

a) The county secures a geodetic crew consisting of a party chief, deputy, and two engineering aides. The personnel is paid for by the country.

b) The crew operates under the supervision and instruction of the NJGCS to survey and maintain GCS monuments in accordance with the needs of the county.

c) The necessary vehicle and instruments are specified by the state and purchased by the county.

d) When establishing new monuments, the county furnishes the necessary concrete mix and the state provides the disks.

e) All computations are made with the help of the NJGCS's computerized program and the original records are kept with the state GCS and a copy with the county engineer.

The above program was initiated with Burlington County in 1975 and presently operates in Burlington and Somerset Counties. Six counties are now on a "waiting list" to begin cooperative surveying in the near future, pending additional state funds and an acceptable agreement with the counties.

LAND DEVELOPMENT AND THE CADASTRE

In accordance with the Land Development Review Resolution of Burlington County, June 30, 1975, all "Preliminary Subdivision Plat Details" for land development located within the distances as shown below, shall have at least two bench marks on the National Geodetic Vertical Datum of 1929 or on NJGCS datum. All contours shall be based and referenced to these bench marks. The elevations of the monuments shall be shown on the plans filed with the county. The required precision shall be 0.035 Run in Miles.

Subdivisions	Distance to Nearest Vertical Control
5 lots to 10 acres	2 miles
over 10 acres to 25 acres	6 miles
over 25 acres	12 miles

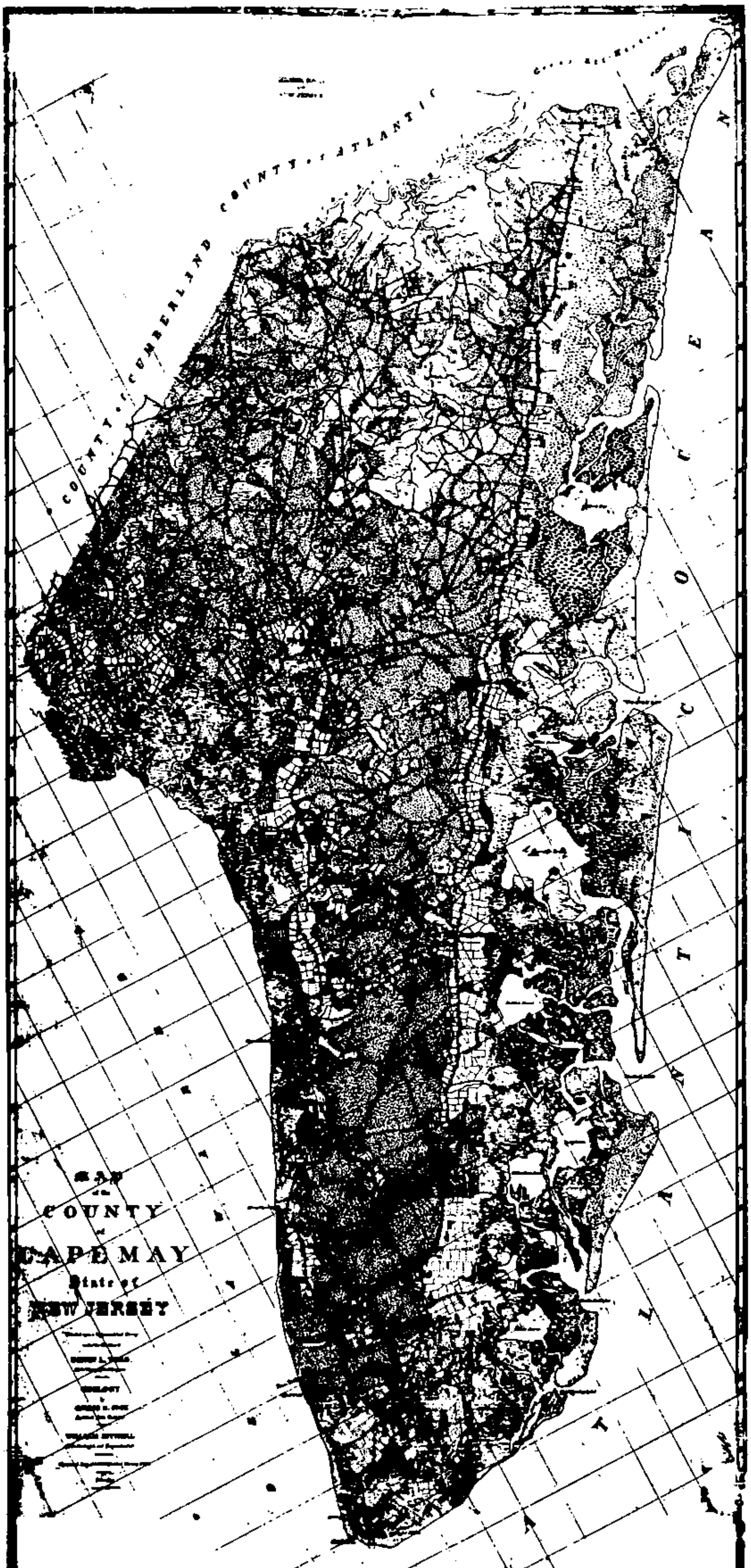

Fig. 3. Map of the County of Cape May, based on Geodetic Survey of Gen. Egberg L. Viele, State Topographical Engineer in 1855-1856.

The property lines of the above land developments shall be tied into the N.J. Plane coordinate system and into the N.J. geodetic network if they are located within the distances as shown in the table below. The coordinates of all monuments shall be shown on the final plats filed in the county clerk's office.

Subdivisions	Distance to Nearest Horizontal Control
5 lots to 10 acres	2 miles
over 10 acres to 50 acres	6 miles
over 50 acres	12 miles

Residential land developments of four lots or less, agricultural subdivisions and court ordered subdivisions may not be required to use geodetic control survey monuments.

In case of "Site Plans," the resolution calls for even stricter requirements as follows:

Land Development:

Site Plan	Distance to Nearest Vertical Control
less than 2 acres	2 miles
2 acres to 10 acres	6 miles
over 10 acres	12 miles

Property Lines:

Site Plan	Distance to Nearest Horizontal Control
2 to 10 acres	2 miles
over 10 acres to 50 acres	6 miles
over 50 acres	12 miles

Site plans of less than two acres may not be required to use geodetic control survey monuments.

The specified distances conform with the present density of the geodetic network. Similar resolutions were adopted by five other counties within the state since 1975. It can be assumed that in the near future this, or a similar resolution, will be adopted by every county. These resolutions are decisive steps toward the cadastre system and will be used to refine community tax maps throughout New Jersey.

TIDAL BENCH MARK OF THE COASTLINE

The sea level along the New Jersey coastline is marked by ninety-three geodetic monuments and their elevations are based on the 1929 sea level datum which is the present basis for all geodetic calculations in the United States and Canada. The 1929 General Adjustment was computed by holding sea level fixed as observed at twenty-one tide stations in the United States and five in Canada. The resulting datum from these observations is referred to as the Sea Level Datum of 1929.

For accuracy of local sea level datum and tidal water elevation, two hundred forty local tidal bench marks have been installed in 1975-1978. These monuments give only the local sea level and tidal information and they are not tied into the National Geodetic Survey or NJGCS network at present.

MAPPING ACTIVITY OF THE N.J. BUREAU OF GEOLOGY AND TOPOGRAPHY-GEODETIC SURVEY

The mapping activity of the Geodetic Survey through 1975 was previously outlined in "Maps of New Jersey--in Summary," ACSM Bulletin, No. 50, Aug. 1975.

Since 1975, the following activity has taken place:

Topographic Maps:

Atlas Sheet #24 (Hunterdon Co. and part of Morris Co. area), scale 1:63,360 or 1 in. equals 1 mi. This is one of the official maps of the state of New Jersey within the topographic series. A revision was completed based on the aerial survey of March 1972 by Mark Hurd (1:24,000). (Prepared for printing).

Land Use Series:

Land Use Overlay Sheets #21 to #37. Scale 1:63,360 or 1 in. equals 1 mi. A survey of land uses throughout New Jersey compiled from EROS images, September 1973, and from aerial survey of March 1972 by Mark Hurd. The land use is classified in accordance with the Geological Survey Professional Paper 964: "A Land Use and Land Cover Classification System for Use with Remote Sensor Data," by J.R. Anderson, et al., Washington, D.C., 1975. (On sale).

Population Series:

Population Overlay #21, #22, and #25. Scale 1:63,360 or 1 in. equals 1 mi. A survey of population density in persons per sq. mi. along with the delineation of urbanized areas (population over 2,000 per sq. mi.) compiled with the 1970 Bureau of Census data. (On sale).

Drainage Basin-flood Prone Area Series:

Drainage Basin Overlay Sheets #21, #22, and #25. Scale 1:63,360 or 1 in. equals 1 mi. A survey of drainage basins in accordance with the N.J. Bureau of Geology and Topography Stream Maps and a delineation of flood-prone areas compiled with 1:24,000 USGS Flood Prone Area Maps, 1972-1976. (On sale).

Water Service Areas:

Water Service Areas and Public Water Supply Overlays. Scale 1:63,360 or 1 in. equals 1 mi. The overlay shows the individual water supply systems categorized by size and service area indicating the main supply lines together with the intake. Overlays compiled in 1976. (On sale).

Sewage and Sanitary Landfills:

Sewage Service Areas, Sewage Treatment Plants and Sanitary Landfills Overlays. Scale 1:63,360 or 1 in. equals 1 mi. The overlay shows the sewage authorities with the served area, main sewage lines, sewage treatments including capacity and sanitary landfills in operation or abandoned ones. Overlays compiled in 1976. (On sale).

Utility Map Series-Scale 1:250,000:

Electrical Services Map. A survey of electrical services throughout New Jersey delineating the service areas of all electric utility companies, all electric transmission lines of at least 115 kilowatts and accompanying transmission substations. Generating stations are also depicted and symbolized according to the type of power used for generation and whether the station is planned or operational. An accompanying leaflet outlines the capacities of transmission substations and generating stations within New Jersey. (On sale).

Water Service Areas with the operating water supply company, Sewage Service Areas, Sewage Treatment and Sanitary Landfills for the entire state.

Oil Pipelines, refineries and tank farms of New Jersey.

Gas Pipelines (high pressure) and gate stations.

IMPROVEMENT OF SERVICES

In order to improve services, eight separate computer programs were introduced at the State Computer Center in Trenton, N.J. A standby program at a Computer Center at some New Jersey college is also planned. These programs were established in order to: *

a) compute state plane coordinates from geodetic positions and vice versa;

b) compute the azimuth and distance between two stations, given their geodetic positions;

c) adjust a traverse survey network using the method of least-squares, the method of condition equations, and the state plane coordinates;

d) compute Universal Transverse Mercator coordinates for any geodetic position on the earth;

e) adjust a vertical survey network using the method of least-squares, and the method of observation equations;

f) compute a geodetic position, given Universal Transverse Mercator coordinates and zone number;

g) compute latitude and longitude from the state rectangular coordinate system and vice-versa; and

*(a) to (f) programs developed by the National Geodetic Survey;

(g) program prepared by N.J. Bureau of Geology & Topography (Mr. Daniel Dombroski) and (h) program planned by N.J. Dept. of Community Affairs.

h) compute municipal boundaries based on 1:126,720 map (1 in. to 2 mi.) from latitude and longitude to the state plane coordinate system. (The program was developed by the N.J. Dept. of Community Affairs for the Clean Water Act--208 Program in connection with MASA's "Landsat" satellite data with the assistance of NJGCS).

Cooperative programs with the National Geodetic Survey (NGS) were instituted as follows:

a) Establishment of EDM (electronic distance measuring) calibration base lines in 1977 in locations:

1) Sussex County area, financed by the Robinson Aerial Survey, Inc.;

2) Morris County area, financed by the Engineers and Surveyors Assoc., Inc.--Bergen-Passaic Unit;

3) Somerset County area, financed by the County of Somerset;

4) Burlington County area, financed by the Surveyor's Association of West Jersey;

5) Cumberland County area, financed by the Surveyor's Association of West Jersey;

6) Mercer County College, financed by the Professional Land Surveyors Association of New Jersey; and

7) Middlesex-Union Counties area financed by the N.J. Society of Professional Engineers-Land Surveyors Practice Section

In all seven locations, the base line is installed by the personnel of NJGCS, the calibration and adjustment is done by National Geodetic Survey, and the calibration is financed by the above-mentioned organizations. Each adjustment costs $500 and is paid to NGS.

b) Workshops with NGS were arranged to improve and update the knowledge and skills of the personnel of NJGCS.

The review of records of all geodetic monuments and the placement of all sketches and decriptions of the geodetic stations on microfiche improved the geodetic services. This program was completed. The placement of all records on a computerized "Geodetic Survey Station Inventory System" is now under consideration.

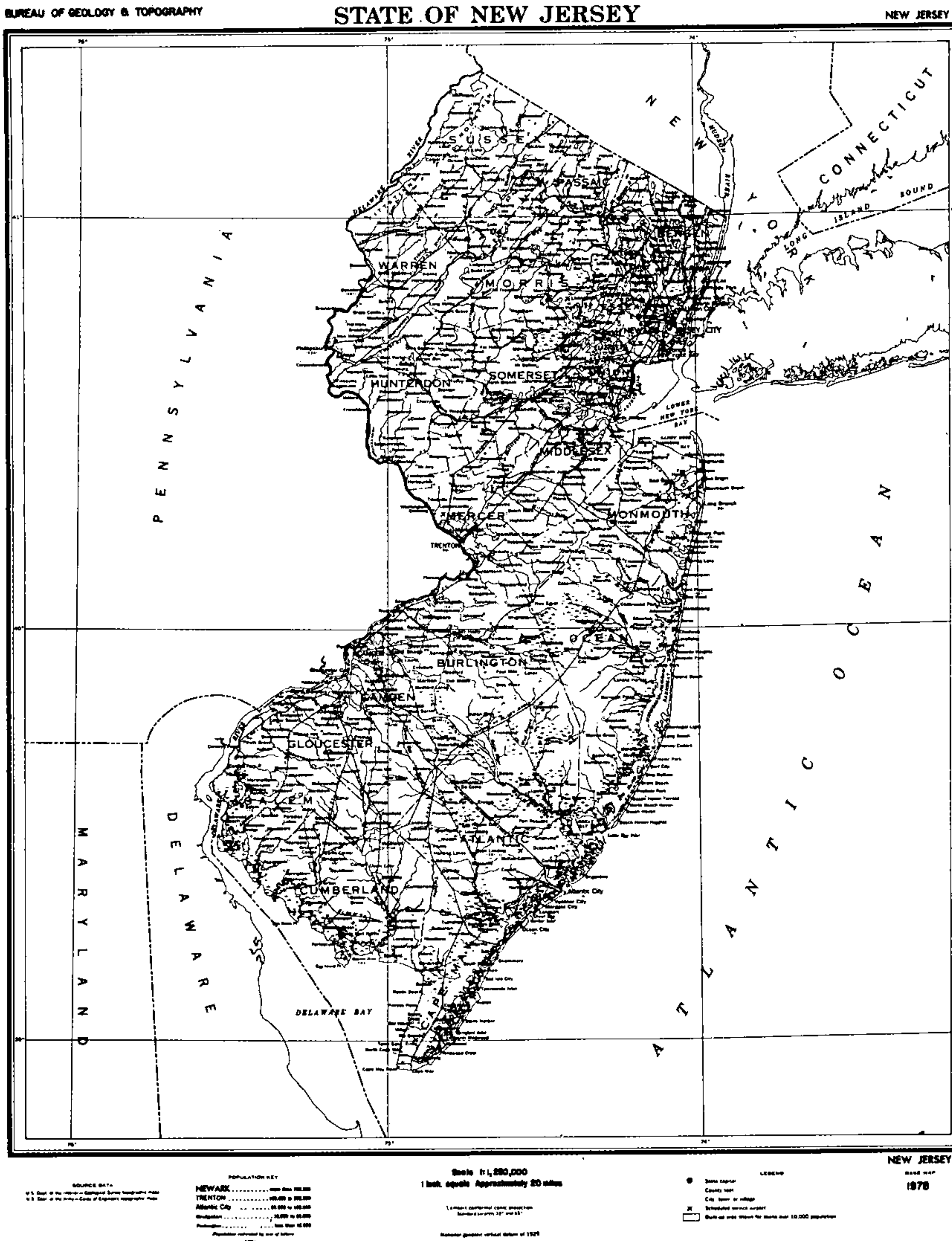

Fig. 4. Map of the State of New Jersey in 1978.

Finally, funds have been requested for fiscal year 1979-1980 which it is hoped will be approved so that NJGCS's crews can be brought up to strength in order to provide the wherewithall to continue training in cooperation with the existing county geodetic crews and those counties which seek to develop their own crews for the maintenance of the N.J. Geodetic Control Monument System.

This program would be developed in four increments. The first increment would expand the existing state geodetic crew with vehicles, instruments, and personnel so that they can properly supervise the county crews. The second increment would establish a second state geodetic crew so that monuments can be more rapidly installed or replaced in the system. The third and fourth increment would provide support personnel and equipment so that the work of the two state crews and the six county crews could be more quickly put into the operating geodetic system.

The extended budget would secure further development of the network. The annual installation of 800 new monuments is planned.

SCHOOLING OF SURVEYORS AND EXCHANGE OF INFORMATION IN THE GEODETIC SURVEY

To update the surveyors' education, a policy was adopted by the N.J. Bureau of Geology and Topography (NJGCS) to encourage every chief of survey party to obtain a bachelor of science degree in land surveying, geodesy, photogrammetry, engineering, or technical services. For practicing land surveyors, a B.S. in technical services branch surveying was initiated at the Thomas A. Edison College, Princeton, N.J., in spring 1978. At present, over 30 land surveyor students are enrolled. Similar programs are considered on a daily full-time basis or evening-course basis at Rutgers University, N.J. Institute of Technology, and Pennsylvania State University. For engineering aides, two years of surveying courses are recommended as a minimum requirement (associate degree in surveying or similar curriculum).

In 1976, the N.J. Bureau of Geology and Topography established contact with the Geodetic Survey of the State of Lower Saxony, Federal Republic of Germany, to exchange "information on geodetic survey services." This program proved to be beneficial and was extended to include the Dept. of Geodesy and Surveying of the Technical University Hannover. Several meetings were held with the representatives of the Commonwealths of Massachusetts and Pennsylvania to discuss the problems of reorganization of geodetic survey services and to initiate a cadastre system. The meetings with both states proved to be extremely valuable since they enabled an exchange of ideas and information.

SPECIFICATIONS AND ACCURACY STANDARDS OF GEODETIC CONTROL SURVEY

The government of the United States makes nationwide surveys of various kinds which must be referenced to national datums. The N.J. Geodetic Control Survey is part of the nationwide geodetic network and the used NJGCS datum is identical with the National Geodetic Survey datum. This is necessary to provide basic information for the conduct of public business at all levels of government, for planning and carrying out national, state, and local projects for programs relating to the development and utilization of natural resources, for national defense, and for development of the country. Requirements for geodetic control surveys are most critical where intensive development is taking place; included are offshore areas where the surveys are used in the development and exploitation of the marine resources and in the delineation of state boundaries.

In close cooperation, a framework of horizontal and vertical control survey was established to ensure coherent products. Geodetic surveys of large areas are affected by, and must take into account the curvature of the earth, astronomic observations, and gravity determinations. High precision geodetic surveys are of two types: horizontal and vertical. Horizontal control is established by triangulation, trilateration, and traverse procedures. Vertical control is provided by leveling of a high order of accuracy. Both types of control establish permanently marked and properly described stations. The national and state networks of these control marks were established with first or second order accuracy. Additional surveys can be incorporated into the national and state network on condition that they meet the specification for geodetic survey set for first and second order accuracy. Monuments surveyed with third order accuracy are not part of the network.

The accepted monuments are recorded at the National Geodetic Survey in Rockville, Md. and at the N.J. Geodetic Control Survey of the Bureau of Geology and Topography in Trenton, N.J. (See tables 1-4 for Accuracy Standards, pages C-19 to C-25.)

ORGANIZATION OF STATE GEODETIC SURVEYS

By January 1977 the following states in the U.S. had their own geodetic survey services in accordance with the National Geodetic Survey's report:

a) Organized as independent agencies:

Hawaii	-- Dept. of Accounting & General Services
Maryland	-- Bureau of Control Survey
Massachusetts	-- Geodetic Survey (in reorganization)
Missouri	-- State Land Survey Authority
New Jersey	-- Bureau of Geology & Topography
No. Carolina	-- Dept. of Conservation & Development, Geodetic Survey Division
Pennsylvania	-- (in organization)

b) Organized within the states' Dept. of Transportation:

As Surveyors:

Connecticut	-- Geodetic Survey
Louisiana	-- within Survey Division
Nevada	-- within Survey Division
New York	-- within Survey Division

As Photogrammetry:

Minnesota	-- Photogrammetric Unit
Oregon	-- Photogrammetry Section

c) Under consideration to establish geodetic control:

Alaska	-- Dept. of Natural Resources
Arizona	-- Dept. of Transportation
California	-- Dept. of Water Resources; or Dept. of Oil & Gas; or Dept. of Transportation
Illinois	-- Dept. of Public Works
Indiana	-- Dept. of Natural Resources
Kansas	-- Dept. of Transportation
Maine	-- Dept. of Transportation
New Hampshire	-- Dept. of Transportation
Ohio	-- Dept. of Natural Resources
Oklahoma	-- Dept. of Transportation
Tennessee	-- Tennessee Valley Authority
Vermont	-- Dept. of Transportation-Aerial Eng. Division
Virginia	-- Dept. of Transportation-Aerial Eng. Division
Washington	-- Dept. of Natural Resources
West Virginia	-- Dept. of Transportation
Wisconsin	-- Dept. of Natural Resources
Wyoming	-- Dept. of Transportation

INSTITUTIONS OFFERING DEGREES IN SURVEYING OR CLOSELY RELATED FIELDS IN THE U.S. AND CANADA (Based on a compilation of the ACSM in 1978)

Until recently schooling of surveyors in the U.S. had no regular pattern. Some engineering education with or without a college degree, and some practice under the supervision of an established surveyor, is usually all that is required to qualify for the Land Surveyors license except in the State of Michigan. In Michigan, a B.S. in Surveying is required for a L.S. license.

Surveying courses have been offered either by 2-year colleges (such as Mercer County and Middlessex Community Colleges) or by many schools as part of the engineering degree (N.J. Institute of Technology).

As of Spring 1978, the following schools were offering a degree or degrees related to surveying and geodesy in the United States:

Arizona State University (Tempe, AZ) --
B.S. in Cartography
Berkeley University of California (Berkeley, CA) --
M.S. and Ph.D. in Photogrammetry and Surveying
California State University (Fresno, CA) --
B.S. in Surveying & Photogrammetry
Clark University (Worcester, MA) --
M.S. and Ph.D. in Cartography
Clarkson College (Potsdam, NY) --
B.S. in Surveying (option)
Cornell University (Ithaca, NY) --
B.S. in Surveying (option); M.S. and Ph.D. in Surveying, Geodesy, Photogrammetry or Remote Sensing
Metropolitan State College of Denver (Denver, (CO) --
B.S. in Surveying
East Central University (Ada, OK) --
B.A. in Cartography
T. A. Edison College (Princeton, NJ) --
B.S. in Surveying (option)
Ferris State College (Big Rapids, MI) --

B.S. in Surveying
University of Illinois (Urbana, IL) --
M.S. and Ph.D. in Photogrammetry
Iowa State University (Ames, IA) --
B.S. in Surveying and Mapping; M.S. and Ph.D. in Surveying
University of Kansas (Lawrence, KS) --
M.S. and Ph.D. in Cartography
Kent State University (Kent, OH) --
M.S. in Cartography
University of Maine (Orono, ME) --
B.S. in Survey Engineering
University of Maryland (College Park, MD) --
M.S. in Cartography
University of Michigan (Ann Arbor, MI) --
M.S. and Ph.D. in Cartography
University of Minnesota (Minneapolis, MN) --
M.S. and Ph.D. in Cartography
State University of New York (Syracuse, NY) --
B.S. in Surveying; M.S. and Ph.D. in Surveying, Geodesy, Photogrammetry and Remote Sensing
Ohio State University (Columbus, OH) --
B.S. in Surveying; M.S. and Ph.D. in Surveying, Geodetic Science; Photogrammetry and Cartography
Oregon State University (Corvallis, OR) --
B.S. in Surveying; M.S. in Surveying, Geodesy, Photogrammetry, Remote Sensing
Penn State University (University Park, PA) --
M.S. and Ph.D. in Cartography
Purdue University (Lafayette, IN) --
B.S. in Surveying, M.S. and Ph.D. in Surveying, Photogrammetry, Geodesy, Remote Sensing
Southern Illinois University (Carbondale, IL) --
M.S. and Ph.D. in Cartography
Virginia Polytechnic Institute and State University (Blacksburg, VA) --
B.S. in Surveying
Washington University (St. Louis, MO) --
B.S. in Geodetic Science
University of Washington (Seattle, WA) --
M.S. and Ph.D. in Geodesy, Photogrammetry, Remote Sensing, Cartography
George Washington University (Washington, D.C.) --
B.S. in Geodetic Science
University of Wisconsin (Madison, WI) --
B.S. in Surveying (option); M.S. and Ph.D. in Surveying Photogrammetry, Remote Sensing

In 1976 Canada established a Land Recordation System (Cadastre) based on European experience. Cadastre and Land Recordation Systems have long been established in Europe as an essential to property ownership recordation. Several hundred colleges in Europe are available to educate surveyors and grant college degrees in surveying. The Canadian Cadastre necessitated to organize surveying curricula and at present in Eastern Canada the most important colleges with such a program are as follows:

Laval University (Quebec) --

B.S. in Surveying; M.S. and Ph.D. in Photogrammetry, Remote Sensing
McMaster University (Hamilton, Ont.) --
M.S. and Ph.D. in Geodesy, Photogrammetry, Remote Sensing
University of New Brunswick (Fredericton, N.B.) --
B.S. in Surveying; M.S. and Ph.D. in Surveying, Geodesy, Photogrammetry, Remote Sensing
University of Toronto (Erindale, Ont.) --
B.S. in Surveying; M.S. and Ph.D. in Photogrammetry, Remote Sensing

MAJOR SURVEYING PROGRAMS AND ACTIVITIES WITHIN THE STATE

Besides the previously described activities of the NJGCS, there are several surveying programs in progress that are conducted by private surveying companies. These projects are summarized as follows:

ADR--Aerial Data Reduction Assoc., Inc., Pennsauken, N.J., is surveying for flood-insurance purposes in Passaic and Bergen Counties for HUD. Additional projects include: Surveying Earle Ammunition Depot, Monmouth Co., and Lakehurst Naval Air Base Station, Ocean Co., for the U.S. Dept. of the Navy.

American Geodetic Service Co., New York City, has concentrated on HUD flood-insurance surveying in Hudson Co. and similar surveying of flood-prone areas for the U.S. Geological Survey in Kentucky, Massachusetts, and New Hampshire.

Coast Survey Ltd., Herndon, Va., and Norman Porter Assoc., New York City, are installing 240 tidal bench marks under the supervision of the National Ocean Survey for the N.J. Dept. of Environmental Protection's Tidal Wetlands project. Ninety-three tidal gauging stations were previously installed and are part of the NJGCS network. The above-mentioned 240 bench marks are contracted exclusively in the Wetlands. They are not in accordance with the National Geodetic Survey Standards and are not tied in with the geodetic network. An additional project is under consideration that will incorporate these bench marks into the NJGCS network.

George de Benedicty Assoc., Jefferson, N.J., has a contract with the Triborough Bridge & Tunnel Authorities, New York City, to perform first-order settlement control measurements on all bridges across the East River and across the Hudson between North Jersey and the New York City area. Further activity includes a flood-prone area survey in North Jersey for HUD: surveying for major subdivision and residential planning and ground monument control for aerial survey use in Morris County.

Emilius Assoc., Oak Ridge, N.J., is surveying for an HUD flood-insurance program in Middlesex and Burlington Counties.

Pandullo Quirk Assoc., Wayne, N.J., has completed the tidal boundary mapping in Hackensack Meadowlands for testimony in the N.J. Sports and Exposition Authority v. Borough of East Rutherford court case. Tide monitoring by tidal gauging stations for the National Ocean Survey in Cape Cod, Mass., Nantucket Island, Mass., and Long Island, N.Y., and an extensive survey for the Cape May Regional Sewer authority are additional important contracts.

John G. Reutter Assoc., Camden, N.J., are mapping and surveying the eastern and northwestern sections of the Delaware-Raritan Canal in Mercer, Somerset, Hunterdon, and Middlesex Counties.

Robinson Aerial Survey, Inc., Newton, N.J., is involved in the preparation of the 1:1200 and 1:600 scale maps for the municipalities of Northern Jersey by photogrammetry. Another project is the aerial photography and analytical aerial triangulation for the Delaware Valley Planning Commission's pilot project, RMLR. The project's objectives include the development of a multipurpose cadastre.

Thomas Tyler Moore Assoc., Inc., Trenton, N.J., is surveying the southwestern portion of the Delaware-Raritan Canal in Mercer County.

Taylor-Wiseman & Taylor, Moorestown, N.J., completed a horizontal and vertical survey with second-order accuracy on the ConRail system between Trenton and New York for the "North-East Corridor" project. The position of the nuclear generating station "Atlantic" and the right-of-way of the cables to the shore were also determined and were made part of the N.J. Geodetic Network. Another important project is the development of a "Cinetheodolite System" at NAFEC, Pomona, Atlantic Co., where a first-order horizontal and vertical control for aviation was established under the supervision of Prof. Kissam, Princeton University. Finally, 65 miles of electric transmission line rights-of-way were surveyed between Camden and New Brunswick.

The Municipal Engineer of Parsippany-Troy Hills Township, with the assistance of George de Benedicty Assoc., is installing a geodetic network for the township area as a base for future cadastre system. The system will have at least four monuments per square mile surveyed with second order accuracy to meet the standards of the National Geodetic Survey and that of the State of New Jersey Geodetic Control Survey.

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Table 1.--Standards for the Classification of Geodetic Control and Principal Recommended Uses

Horizontal Control

Classification	*First-Order*	*Second-Order*		*Third-Order*	
		Class I	*Class II*	*Class I*	*Class II*
Relative accuracy between directly connected adjacent points (at least)	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000
Recommended uses	Primary National Network. Metropolitan Area Surveys. Scientific Studies	Area control which strengthens the National Network. Subsidiary metropolitan control.	Area control which contributes to, but is supplemental to, the National Network.	General control surveys referenced to the National Network. Local control surveys.	

Vertical Control

Classification	*First-Order*		*Second-Order*		*Third-Order*
	Class I	*Class II*	*Class I*	*Class II*	
Relative accuracy between directly connected points or benchmarks (standard error)	0.5 mm $\sqrt{K}$	0.7 mm $\sqrt{K}$	1.0 mm $\sqrt{K}$	1.3 mm $\sqrt{K}$	2.0 mm $\sqrt{K}$
			(K is the distance in kilometers between points.)		
Recommended uses	Basic framework of the National Network and metropolitan area control. Regional crustal movement studies. Extensive engineering projects. Support for subsidiary surveys.		Secondary framework of the National Network and metropolitan area control. Local crustal movement studies. Large engineering projects. Tidal boundary reference. Support for lower order surveys.	Densification within the National Network. Rapid subsidence studies. Local engineering projects. Topographic mapping.	Small-scale topographic mapping. Establishing gradients in mountainous areas. Small engineering projects. May or may not be adjusted to the National Network.

Table 2.--Classification, Standards of Accuracy, and General Specifications for Horizontal Control

Triangulation

Classification	*First-Order*	*Second-Order* Class I	*Second-Order* Class II	*Third-Order* Class I	*Third-Order* Class II
Recommended spacing of principal stations	Network stations seldom less than 15 km. Metropolitan surveys 3 km to 8 km and others as required.	Principal stations seldom less than 10 km. Other surveys 1 km to 3 km or as required.	Principal stations seldom less than 5 km or as required.	As required	As required
Strength of figure					
R_1 between bases					
Desirable limit	20	60	80	100	125
Maximum limit	25	80	120	130	175
Single figure					
Desirable limit					
R_1	5	10	15	25	25
R_2	10	30	70	80	120
Maximum limit					
R_1	10	25	25	40	50
R_2	15	60	100	120	170
Base measurement					
Standard error [1]	1 part in 1,000,000	1 part in 900,000	1 part in 800,000	1 part in 500,000	1 part in 250,000
Horizontal directions [2]					
Instrument	0".2	0."2	0".2 } or { 1".0	1".0	1".0
Number of positions	16	16	8 } or { 12	4	2
Rejection limit from mean	4"	4"	5" } or { 5"	5"	5"
Triangle closure					
Average not to exceed	1".0	1".2	2".0	3".0	5".0
Maximum seldom to exceed	3".0	3".0	5".0	5".0	10".0
Side checks					
In side equation test, average correction to direction not to exceed	0".3	0".4	0".6	0".8	2"
Astro azimuths [3]					
Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of obs./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	0".6	0".8	3".0
Vertical angle observations [4]					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"

SPECIFICATIONS AND ACCURACY STANDARDS OF GEODETIC CONTROL SURVEY

Table 2.--Continued

	Triangulation				
Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Closure in length [8] (also position when applicable) after angle and side conditions have been satisfied, should not exceed	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000
	TRILATERATION				
Recommended spacing of principal stations	Network stations seldom less than 10 km. Other surveys seldom less than 3 km.	Principal stations seldom less than 10 km. Other surveys seldom less than 1 km.	Principal stations seldom less than 5 km For some surveys a spacing of 0.5 km between stations may be satisfactory.	Principal stations seldom less than 0.5 km.	Principal stations seldom less than 0.25 km.
Geometric configuration [6]					
Minimum angle contained within, not less than	25°	25°	20°	20°	15°
Length measurement					
Standard error [1]	1 part in 1,000,000	1 part in 750,000	1 part in 450,000	1 part in 250,000	1 part in 150,000
Vertical angle observations [4]					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of figures between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths [5]					
Spacing-figures	6-8	6-10	8-10	10-12	12-15
No. of obs./night	16	16	16	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	0".6	0".8	3".0
Closure in position [8]					
after geometric conditions have been satisfied should not exceed	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10,000	1 part in 5,000

See notes (1)-(8), p. 7.

SPECIFICATIONS AND ACCURACY STANDARDS OF GEODETIC CONTROL SURVEY

Table 2.--Continued

Traverse

Classification	*First-Order*	*Second-Order*		*Third-Order*	
		Class I	*Class II*	*Class I*	*Class II*
Recommended spacing of principal stations	Network stations 10-15 km Other surveys seldom less than 3 km.	Principal stations seldom less than 4 km except in metropolitan area surveys where the limitation is 0.3 km.	Principal stations seldom less than 2 km except in metropolitan area surveys where the limitation is 0.2 km.	Seldom less than 0.1 km in tertiary surveys in metropolitan area surveys. As required for other surveys.	
Horizontal directions or angles [*]					
Instrument	0".2	0".2 or 1".0	0".2 or 1".0	1".0	1".0
Number of observations	16	8 or 12*	6 or 8*	4	2
Rejection limit from mean	4"	4" or 5"	4" or 5"	5"	5"
Length measurements					
Standard error [1]	1 part in 600,000	1 part in 300,000	1 part in 120,000	1 part in 60,000	1 part in 30,000
Reciprocal vertical angle observations [*]					
Number of and spread between observations	3 D/R—10"	3 D/R—10"	2 D/R—10"	2 D/R—10"	2 D/R—20"
Number of stations between known elevations	4-6	6-8	8-10	10-15	15-20
Astro azimuths					
Number of courses between azimuth checks [7]	5-6	10-12	15-20	20-25	30-40
No. of obs./night	16	16	12	8	4
No. of nights	2	2	1	1	1
Standard error	0".45	0".45	1".5	3".0	8".0
Azimuth closure at azimuth check point not to exceed [*]	1".0 per station or 2" $\sqrt{N}$	1".5 per station or 3" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 2".0 per station or 3" $\sqrt{N}$ •	2".0 per station or 6" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 4".0 per station or 8" $\sqrt{N}$	3".0 per station or 10" $\sqrt{N}$. Metropolitan area surveys seldom to exceed 6".0 per station or 15" $\sqrt{N}$	8" per station or 30" $\sqrt{N}$
Position closure [*] [*] after azimuth adjustment	0.04m $\sqrt{K}$ or 1:100,000	0.08m $\sqrt{K}$ or 1:50,000	0.2m $\sqrt{K}$ or 1:20,000	0.4m $\sqrt{K}$ or 1:10,000	0.8m $\sqrt{K}$ or 1:5,000

* May be reduced to 8 and 4, respectively, in metropolitan areas.

Table 2--Continued

NOTES

NOTE (1)

The standard error is to be estimated by

$$\sigma_m = \sqrt{\frac{\Sigma v^2}{n(n-1)}}$$

where σ_m is the standard error of the mean, v is a residual (that is, the difference between a measured length and the mean of all measured lengths of a line), and n is the number of measurements.

The term "standard error" used here is computed under the assumption that all errors are strictly random in nature. The true or actual error is a quantity that cannot be obtained exactly. It is the difference between the true value and the measured value. By correcting each measurement for every known source of systematic error, however, one may approach the true error. It is mandatory for any practitioner using these tables to reduce to a minimum the effect of all systematic and constant errors so that real accuracy may be obtained. (See page 267 of Coast and Geodetic Survey Special Publication No. 247, "Manual of Geodetic Triangulation," Revised edition, 1959, for definition of "actual error.")

NOTE (2)

The figure for "Instrument" describes the theodolite recommended in terms of the smallest reading of the horizontal circle. A position is one measure, with the telescope both direct and reversed, of the horizontal direction from the initial station to each of the other stations. See FGCC "Detailed Specifications" for number of observations and rejection limits when using transits.

NOTE (3)

The standard error for astronomic azimuths is computed with all observations considered equal in weight (with 75 percent of the total number of observations required on a single night) after application of a 5-second rejection limit from the mean for First- and Second-Order observations.

NOTE (4)

See FGCC "Detailed Specifications" on "Elevation of Horizontal Control Points" for further details. These elevations are intended to suffice for computations, adjustments, and broad mapping and control projects, not necessarily for vertical network elevations.

NOTE (5)

Unless the survey is in the form of a loop closing on itself, the position closures would depend largely on the constraints or established control in the adjustment. The extent of constraints and the actual relationship of the surveys can be obtained through either a review of the computations, or a minimally constrained adjustment of all work involved. The proportional accuracy or closure (i.e. 1/100,000) can be obtained by computing the difference between the computed value and the fixed value, and dividing this quantity by the length of the loop connecting the two points.

NOTE (6)

See FGCC "Detailed Specifications" on "Trilateration" for further details.

NOTE (7)

The number of azimuth courses for First-Order traverses are between Laplace azimuths. For other survey accuracies, the number of courses may be between Laplace azimuths and/or adjusted azimuths.

NOTE (8)

The expressions for closing errors in traverses are given in two forms. The expression containing the square root is designed for longer lines where higher proportional accuracy is required.

The formula that gives the smallest permissible closure should be used.

N is the number of stations for carrying azimuth.
K is the distance in kilometers.

Table 3.--Classification, Standards of Accuracy, and General Specifications for Vertical Control

Classification	First-Order Class I, Class II	Second-Order Class I	Second-Order Class II	Third-Order
Principal uses Minimum standards; higher accuracies may be used for special purposes	Basic framework of the National Network and of metropolitan area control Extensive engineering projects Regional crustal movement investigations Determining geopotential values	Secondary control of the National Network and of metropolitan area control Large engineering projects Local crustal movement and subsidence investigations Support for lower-order control	Control densification, usually adjusted to the National Net. Local engineering projects Topographic mapping Studies of rapid subsidence Support for local surveys	Miscellaneous local control; may not be adjusted to the National Network. Small engineering projects Small-scale topo. mapping Drainage studies and gradient establishment in mountainous areas
Recommended spacing of lines National Network	Net A; 100 to 300 km *Class I* Net B; 50 to 100 km *Class II*	Secondary Net; 20 to 50 km	Area Control; 10 to 25 km	As needed
Metropolitan control;	2 to 8 km	0.5 to 1 km	As needed	As needed
other purposes	As needed	As needed	As needed	As needed
Spacing of marks along lines	1 to 3 km	1 to 3 km	Not more than 3 km	Not more than 3 km
*Gravity requirement**	0.20×10^{-3}gpu	....	....	....
Instrument standards	Automatic or tilting levels with parallel plate micrometers; invar scale rods	Automatic or tilting levels with optical micrometers or three-wire levels; invar scale rods	Geodetic levels and invar scale rods	Geodetic levels and rods
Field procedures	Double-run; forward and backward, each section	Double-run; forward and backward, each section	Double- or single-run	Double- or single-run
Section length	1 to 2 km	1 to 2 km	1 to 3 km for double-run	1 to 3 km for double-run
Maximum length of sight	50 m *Class I*; 60 m *Class II*	60 m	70 m	90 m
Field procedures† Max. difference in lengths Forward & backward sights per setup	2 m *Class I*; 5 m *Class II*	5 m	10 m	10 m
per section (cumulative)	4 m *Class I*; 10 m *Class II*	10 m	10 m	10 m
Max. length of line between connections	Net A; 300 km Net B; 100 km	50 km	50 km double-run 25 km single-run	25 km double-run 10 km single-run
Maximum closures‡ Section; fwd. and bkwd.	3 mm $\sqrt{K}$ *Class I*; 4 mm $\sqrt{K}$ *Class II*	6 mm $\sqrt{K}$	8 mm $\sqrt{K}$	12 mm $\sqrt{K}$
Loop or line	4 mm $\sqrt{K}$ *Class I*; 5 mm $\sqrt{K}$ *Class II*	6 mm $\sqrt{K}$	8 mm $\sqrt{K}$	12 mm $\sqrt{K}$

* See text for discussion of instruments.

† The maximum length of line between connections may be increased to 100 km for double run for Second-Order, Class II, and to 50 km for double run for Third-Order in those areas where the First-Order control has not been fully established.

‡ Check between forward and backward runnings where K is the distance in kilometers.

SPECIFICATIONS AND ACCURACY STANDARDS OF GEODETIC CONTROL SURVEY

Table 4.--National Geodetic Networks

Horizontal

Classification	Nationwide high precision traverses—Satellite Control	*First-Order*	*Second-Order*		*Third-Order*	
			Class I	*Class II*	*Class I*	*Class II*
Network component	Basic horizontal framework (control establishes the National Network)	Primary horizontal network (control develops the National Network)	Secondary horizontal control (control strengthens the National Network)	Supplemental horizontal control (Control contributes to the National Network)	Local horizontal control (control is referenced to the National Network)	
Nominal accuracy or precision between adjacent points	I part in 1,000,000	1 part in 100,000	1 part in 50,000	1 part in 20,000	1 part in 10.000	1 part in 5,000
Recommended density of control	Traverses and satellite stations at 900-1200 km. Stations at 15 km. to limit of technical and geometric restraints	Arcs not in excess of 100 km. Stations at 12-20 km. Urban control 3-8 km.	Stations at 10-13 km. Urban control 1-3 km.	As required	As required	

Vertical

Classification	*First-Order*		*Second-Order*		*Third-Order*
	Class I	*Class II*	*Class I*	*Class II*	
Network component	Basic Vertical Network A (control establishes the National Network)	Basic Vertical Network B	Secondary Vertical Network (Control develops the National Network)	Supplemental Vertical Control (Control contributes to the National Network)	Local vertical control
Nominal accuracy *between points**	1.5 mm$\sqrt{K}$	2 mm $\sqrt{K}$	3 mm $\sqrt{K}$	4 mm $\sqrt{K}$	6 mm $\sqrt{K}$
Recommended density of *lines*	100-300 km	50-100 km.	25-50 km	10-25 km	As needed

* One-half of permissable closure

THE NATIONAL MAPPING PROGRAM AND
STATUS OF MAPPING
NEW JERSEY

by

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Introduction

In 1975 the Geological Survey established the National Mapping Program to identify, evaluate, and respond to current and future map and map data requirements effectively and in a timely way. The new Program was a reshaping and expansion of the National Topographic Program, which had as its prime objective the completion of national topographic mapping at 1:24,000 scale in 7.5-minute quadrangle units. The production of 7.5-minute quadrangle maps is still an important part of the National Mapping Program, but changes in recent years in technology as well as in uses and applications of maps and map data have brought new requirements for the mapping program. These requirements will not be met by the National Mapping Program, which will provide for the acquisition of the following categories of base map data: 1) reference systems, 2) cartography, 3) hydrography, 4) surface cover, 5) nonvegetative features, 6) boundaries, 7) transportation systems, 8) cultural features, such as building airports, and dams, 9) geodetic control and other survey information, 10) geographic names, and 11) orthophotographic imagery. The forms of the data provided in these different categories will be based on the known needs of users and will be available also at different levels of detail and scale.

These data will be processed and reduced by the Geological Survey for direct use by Federal, State, and local agencies and the public. The Program will consist of elements of topographic mapping, aerial and space imagery, special interest mapping, metrication, digital cartography, cartographic information, and research and development.

Many of the improvements in program, technological development, and responsiveness to national priorities are a result of recommendations by the Federal Mapping Task Force, a group organized in 1972 under the aegis of the Office of Management and Budget. The Task Force examined Federal civil mapping, charting, and geodesy (MC&G) programs and activities and made extensive recommendations for more efficient organization, management, and actions by the Federal MC&G community.1/

In this paper a general description will be given of the National Mapping Program; the status of the various elements and activities of the Program will be described, and special attention will be given to the status and condition of national topographic mapping in New Jersey. New Jersey is examined as an example of a State which has a wide array of planning and management problems that support extensive requirements for accurate and up-to-date topographic maps of high quality, yet the maps in New Jersey are to some extent below standards and to a considerable extent are out of date.

1/ Report of the Federal Mapping Task Force, 1973.

Topographic Mapping

Conventional multiuse line maps continue to be a large requirement by government agencies at all levels and by the general public. These maps in the National Topographic Map Series include the standard quadrangle maps, intermediate-scale maps, and small-scale maps to meet an ever-widening array of map uses. It is the U.S. Geological Survey objective to complete country coverage at 1:24,000 scale and to maintain that series up to date by revision. Considerable coverage in the 15-minute series at 1:62,500 scale still remains and is being replaced systematically by the newer, more detailed 1:24,000-scale maps. Coverage of Alaska is primarily in the Alaska 1:63,360 series, but areas of critical development are being covered at 1:25,000 scale.

Published 1:24,000-scale maps for the conterminous States and Hawaii are now available for about 70 percent of the country, and another 7 percent are available in advance copy. 1:62,500-scale maps published for 49 States now cover about 23 percent. This series will ultimately be phased out when it has been totally replaced by 1:24,000-scale mapping. In Alaska, 1:63,360 coverage is published for about 83 percent, and advance copy is available for an additional 1 percent.

It is estimated that 1:24,000-scale coverage for the conterminous States and Hawaii will be completed about 1986. A large standard mapping program will be continued to the extent necessary, but there has been, and will continue to be, some need to reprogram to accommodate requirements for other products.

An important addition to the family of maps provided by the National Mapping Program consists of intermediate-scale maps. These provide map coverage between the traditional 1:24,000 and 1:250,000 scales. They consist primarily of 1:100,000- and 1:50,000-scale maps in quadrangle and county formats. These multipurpose base maps are prepared to meet the needs of such agencies as the Bureau of Land Management for their surface management program, the soil Conservation Service for their Unique and Important Farmlands program, the U.S. Fish and Wildlife Service for their National Wetlands Inventory program, and various State agencies for planning and support of State and county programs.

The short-range goal is to produce either planimetric or topographic maps to meet immediate Federal and State map requirements. A long-range goal is to complete and maintain the 1:100,000-scale metric topographic quadrangle map coverage for the conterminous United States and Hawaii. Maps will also be prepared for Alaska where needed.

About 1,100 various intermediate-scale maps are now in the program, of which 150 have been published as either quadrangle or county maps. The remaining maps are available as either advance (diazo) copy or are still in work. An index showing the status of this map coverage is available on request.

Maps at 1:250,000 scale have been published for the entire United States. These maps were done by the military in the early 1950's and then turned over to the Geological Survey for maintenance and distribution. They have become a standard series of the National Mapping Program. The current 1:250,000-scale program is primarily one of revision. Some 30 to 35 of these maps are updated each year.

An important addition to the National Topographic Map series has resulted from the production of orthophotoquads. Orthophotoquads consist of an orthophoto or a mosaic of orthophotos in 7.5-minute format at 1:24,000 scale. These orthophotoquads have very few cartographic additions, and they can be used as interim maps or as updating complements to published maps. They meet many needs where conventional line maps are not yet available because they have been produced virtually for all areas of the 48 States not yet covered at 1:24,000 scale, about 27 percent of the country in the last 5 years.

Special-Interest Mapping

Photomechanical techniques for producing slope maps have been developed to provide limited capacity for producing slope maps to satisfy specific user needs. To date, over 500 slope maps have been prepared.

USGS is looking into distribution of maps in folded form, which may provide a product more easily displayed on sales counters and more easily carried and used by travelers and outdoors enthusiasts. Experiments are being conducted with both paper and plastic map jackets. Indications are that these maps will be of considerable popularity in recreational areas.

A second edition of the National Atlas of the United States is likely for the mid-1980's, and a committee has been appointed to plan its production. Communications are being maintained with geographers, demographers, and subject-matter specialists who contributed to the original edition so that they can help in improving the new addition.

Metrication

Congress passed a bill encouraging exclusive use of the metric system, the Metric Conversion Act of 1975, signed by the President in December 1975. Unfortunately no timetable for conversion was specified. In anticipation of metrication, USGS distributed a questionnaire to 150 State and Federal agencies, educators, and private mapping companies. The answers to the primary questions are as follows: a 2:1 preference for 1:25,000 over 1:20,000 scale for the 7.5-minute quadrangle maps; a 4:1 preference for a contour-interval sequence of 1,2,4,10,20,50,100 over other alternatives; and a 4:3 preference for completing the 7.5-minute coverage within individual States before changing to the metric system. After evaluating these responses, USGS has decided to proceed as fast as practical toward producing new metric maps.

As part of its 7.5-minute map production, the GS has programed 1,700 quadrangles in 19 States for topographic mapping in the metric system.

Current policy is to work separately with each State on how best to proceed with metric conversion. Three options can be exercised, depending on State preference. The first is fully metric mapping, in which all elements of the map are metric and the scale is 1:25,000. The second is to map with metric contours and data but to publish at 1:24,000 scale. The final option is to defer metric mapping until State coverage is completed at 1:24,000-scale in conventional units.

USGS metric policy also states that all complete revisions of 7.5-minute and 1:250,000-scale maps will be metric; also, all topographic editions of our new 1:100,000-scale series will be metric.

Research and Development

R&D in mapping is an integral part of the National Mapping Program. There are strong technological advances in photogrammetric instrumentation, automation in cartography, aerial photography and space imagery, digital data base development, sophisticated image analysis and enhancement, and applications of inertial navigational systems to surveying and mapping problems. These require careful attention to effective of existing technology and sponsorship or development of new capabilities.

Automation is an important collective objective of current research and experimentation. We will automate as many as possible of the mapping phases so that ultimately only a few critical phases will be done by human manipulation or intervention. Automatic data processing is already an integral part of the mapping system--in data acquisition; analytical aerotriangulation; production of orthophotos, digital terrain models, and some contour plots; computation and adjustment of geodetic control; and high-speed automatic plotting.

Map Revision

Much progress has been made recently in map preparation, particularly in instrumentation, new materials and techniques, and means of presentation. These new capabilities have permitted the production of more maps of different kinds than ever before to meet important requirements, adding substantially to the inventory of available maps. These growing inventories also mean more maps to go out of date. This problem is most acute with the 7.5-minute quadrangle maps. The truth is that maps do go out of date, and in many areas of the United States they go out of date very rapidly. Such areas as the urbanizing portions of vast metropolitan areas, transportation corridors, small- and medium-sized towns and cities, and coastal areas are particularly vulnerable targets for map obsolescence. Another problem lies in the inventory of a substantial number of old map bases made by outdated methods, which do not meet National Map Accuracy Standards. For these maps, the only remedy is a complete revision or remapping, which is as costly as a new mapping project. Accordingly, such obsolete maps must compete in priority for funds and scheduling with new maps, coverage where none exists. As a result, relatively few complete revisions are scheduled. Serious attention is being paid to this problem, and the number of map revision projects in the program will be substantially increased in fiscal year 1979 and future years.

The current revision program consists of prerevision photoinspection and then revision of as many of those candidate maps as can be scheduled with available resources. Photoinspection is a comparison with current high-altitude, quad-centered photographs, or equivalent; those quadrangles that pass as not needing revision have the review date added to the map at the time of the next reprinting.

The FY 1978 and 1979 revision programs are summarized as follows:

	FY 1978	FY 1979 (planned)
Inspections	4500	5000
Revisions (total)	2030	2175
Photo	1900	2000
Partial	70	100
Complete	6	75

In the later discussion on the status of mapping in New Jersey it will be clear how New Jersey mapping has been affected by the revision problem.

National Cartographic Information Center

The National Cartographic Information Center (NCIC), established in 1974, provides a national information service to make cartographic data of the United States more easily accessible to the user.2/

"Cartographic data" means maps and charts, aerial photographs, geodetic control, and map data in digital form. At present, more than 30 Federal agencies collect and prepare cartographic data. Most of the data are usable for purposes other than those for which they were originally acquired; however, they are not usually available without considerable effort. NCIC will not obtain all the cartographic data from the present holders. Instead, it will collect and organize descriptive information about the data and help users select and place orders.

NCIC has begun compiling records of many cartographic holdings throughout the United States. Currently the records include holdings of the various Federal agencies. The latest techniques of microphotography and computer technology will be used to reduce the vast amount of data to a manageable size. NCIC goals include cataloging the holdings of State, local, and private sources and providing information on plans for data scheduled to be collected. There are NCIC offices at Geological Survey Headquarters in Reston, Va., and at the Mapping Centers, in Rolla, Mo., Denver, Colo., and Menlo Park, Calif. There are now affiliate offices in about 10 States.

2/The National Mapping Program of the United States, R. H. Lyddan, 1976.

Mapping in New Jersey

The State of New Jersey is covered by 174 7.5-minute topographic quadrangle maps at 1:24,000 scale. The State also produces and maintains a series at 1:63,360 scale, covering the entire State (see Fig. 1).

Overall, the published 7.5-minute coverage needs modernization to adequately meet the needs of today's users, who require precise and up-to-date cartographic information for a host of problems and tasks. A recent evaluation of New Jersey 7.5-minute coverage indicates the following (see Figs. 2 and 3):

- 80 maps, covering the approximate north half of the State and a few maps along the Delaware River and Bay area, were prepared before 1947 and do not meet the horizontal requirement of the National Map Accuracy Standards. Most of these maps were originally prepared by the Army Map Service to meet military requirements, and many were done under contract.

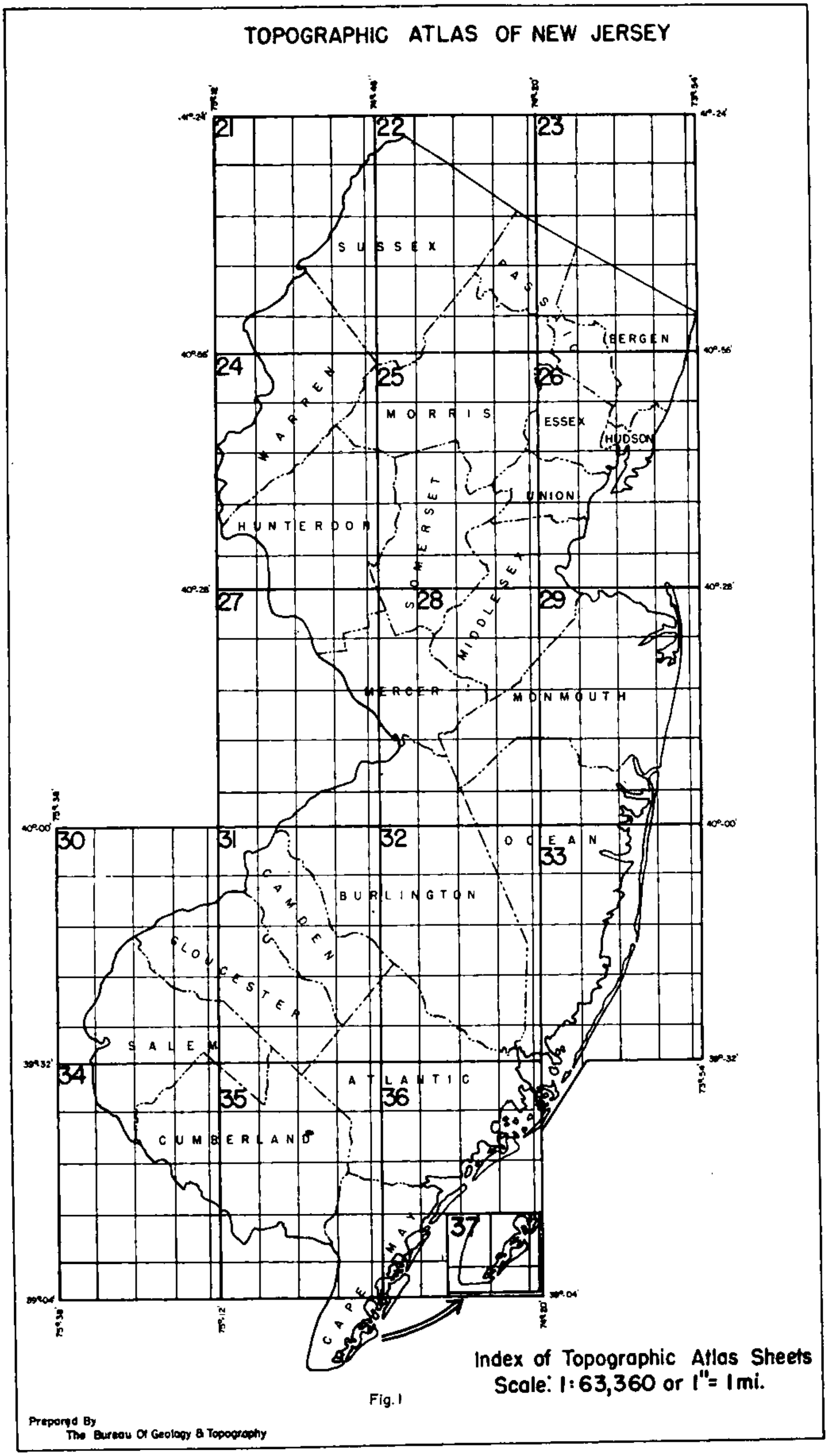
- 88 maps, covering the approximate south half of the State--roughly below Asbury Park and a few maps in the Staten Island area--were prepared between 1948 and 1957. These should all be checked to determine whether they meet National Map Accuracy standards.

In all areas of the State, many maps do not have the contour interval needed to best portray the terrain according to today's requirements.

- 6 maps along the New York/New Jersey line were completely revised or compiled between 1958 and 1969 or are being completely revised. Three of these have been published and the other three are still in work. These maps meet the National Map Accuracy Standards.

None of the maps show bathymetric data in the coastal areas, as is being done in other States.

In response to the annual solicitation for mapping requirements by Federal agencies (Office of Management and Budget Circular A-16) and in accordance with the Survey's revision policy criteria, most of the New Jersey maps have been authorized for photoinspection and photorevision. They will be revised when inspection shows that the amount of cultural change meets minimum requirements for updating. Recent studies show that nearly 50 percent of the quadrangles need cultural updating. Photorevision is a planimetric update only and does not include field verification of the newly added data or improvement of the map's basic accuracy. A photorevised map shows the new data in purple when the map is next reprinted. The date of the aerial photographs used in the inspection and revision is shown on the maps and on the State sales index. Copies of the index are free on request.


TOPOGRAPHIC ATLAS OF NEW JERSEY
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SOMERSET
MIDDLESEX
MERCER
MONMOUTH
OCEAN
BURLINGTON
CAMDEN
GLOUCESTER
SALEM
ATLANTIC
CUMBERLAND
CAPE MAY
Index of Topographic Atlas Sheets
Scale: 1:63,360 or 1" = 1 mi.
Prepared By
The Bureau Of Geology & Topography

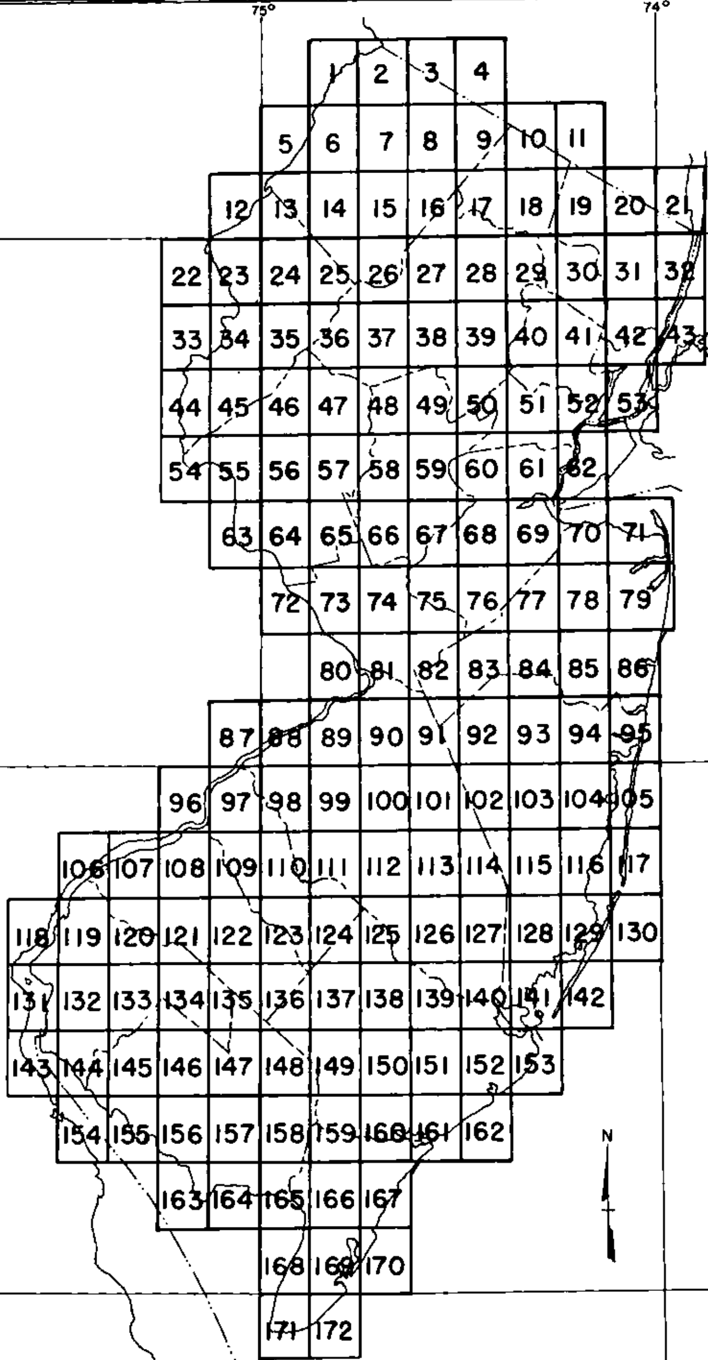
Fig. 1

STATE OF NEW JERSEY

7½ MINUTE TOPOGRAPHIC QUADRANGLES (1:24,000)

INDEX MAP

1 MILFORD
2 PORT JERVIS SOUTH
3 UNIONVILLE
4 PINE ISLAND
5 LAKE MASKENOZHA
6 CULVERS GAP
7 BRANCHVILLE
8 HAMBURG
9 WAWAYANDA
10 GREENWOOD LAKE
11 SLOATSBURG
12 BUSHKILL
13 FLAT BROOKVILLE
14 NEWTON WEST
15 NEWTON EAST
16 FRANKLIN
17 NEWFOUNDLAND
18 WANAQUE
19 RAMSEY
20 PARK RIDGE
21 NYACK
22 STROUDSBURG
23 PORTLAND
24 BLAIRSTOWN
25 TRANQUILITY
26 STANHOPE
27 DOVER
28 BOONTON
29 POMPTON PLAINS
30 PATERSON
31 HACKENSACK
32 YONKERS
33 BANGOR
34 BELVIDERE
35 WASHINGTON
36 HACKETTSTOWN
37 CHESTER
38 MENDHAM
39 MORRISTOWN
40 CALDWELL
41 ORANGE
42 WEEHAWKEN
43 CENTRAL PARK
44 EASTON
45 BLOOMSBURY
46 HIGH BRIDGE
47 CALIFON
48 GLADSTONE
49 BERNARDSVILLE
50 CHATHAM
51 ROSELLE
52 ELIZABETH
53 JERSEY CITY
54 RIEGELSVILLE
55 FRENCHTOWN
56 PITTSTOWN
57 FLEMINGTON
58 RARITAN
59 BOUND BROOK
60 PLAINFIELD
61 PERTH AMBOY
62 ARTHUR KILL
63 LUMBERVILLE
64 STOCKTON
65 HOPEWELL
66 ROCKY HILL
67 MONMOUTH JUNCTION
68 NEW BRUNSWICK
69 SOUTH AMBOY
70 KEYPORT
71 SANDY HOOK
72 LAMBERTVILLE
73 PENNINGTON
74 PRINCETON
75 HIGHTSTOWN
76 JAMESBURG
77 FREEHOLD
78 MARLBORO
79 LONG BRANCH
80 TRENTON WEST
81 TRENTON EAST
82 ALLENTOWN
83 ROOSEVELT
84 ADELPHIA
85 FARMINGDALE
86 ASBURY PARK



87 FRANKFORD
88 BEVERLY
89 BRISTOL
90 COLUMBUS
91 NEW EGYPT
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93 LAKEHURST
94 LAKEWOOD
95 POINT PLEASANT
96 PHILADELPHIA
97 CAMDEN
98 MOORESTOWN
99 MT. HOLLY
100 PEMBERTON
101 BROWNS MILLS
102 WHITING
103 KESWICK GROVE
104 TOMS RIVER
105 SEASIDE PARK
106 MARCUS HOOK
107 BRIDGEPORT
108 WOODBURY
109 RUNNEMEDE
110 CLEMENTON
111 MEDFORD LAKES
112 INDIAN MILLS
113 CHATSWORTH
114 WOODMANSIE
115 BROOKVILLE
116 FORKED RIVER
117 BARNEGAT LIGHT
118 WILMINGTON SOUTH
119 PENNS GROVE
120 WOODSTOWN
121 PITMAN WEST
122 PITMAN EAST
123 WILLIAMSTOWN
124 HAMMONTON
125 ATSION
126 JENKINS
127 OSWEGO LAKE
128 WEST CREEK
129 SHIP BOTTOM
130 LONG BEACH NE
131 DELAWARE CITY
132 SALEM
133 ALLOWAY
134 ELMER
135 NEWFIELD
136 BUENA
137 NEWTONVILLE
138 HARBOR CITY
139 GREEN BANK
140 NEW GRETNA
141 TUCKERTON
142 BEACH HAVEN
143 TAYLORS BRIDGE
144 CANTON
145 SHILOH
146 BRIDGETON
147 MILLVILLE
148 FIVE POINTS
149 DOROTHY
150 MAYS LANDING
151 PLEASANTVILLE
152 OCEANVILLE
153 BRIGANTINE INLET
154 BOMBAY HOOK
155 BEN DAVIS POINT
156 CEDARVILLE
157 DIVIDING CREEK
158 PORT ELIZABETH
159 TUCKAHOE
160 MARMORA
161 OCEAN CITY
162 ATLANTIC CITY
163 FORTESCUE
164 PORT NORRIS
165 HEISLERVILLE
166 WOODBINE
167 SEA ISLE CITY
168 RIO GRANDE
169 STONE HARBOR
170 AVALON
171 CAPE MAY
172 WILDWOOD

Fig. 2

MARCH 1977

SCALE: 1:500,000

10 5 0 10 20 30 Miles

10 0 10 20 30 40 Kilometers

NEW JERSEY

Status of 1:24,000-Scale Mapping

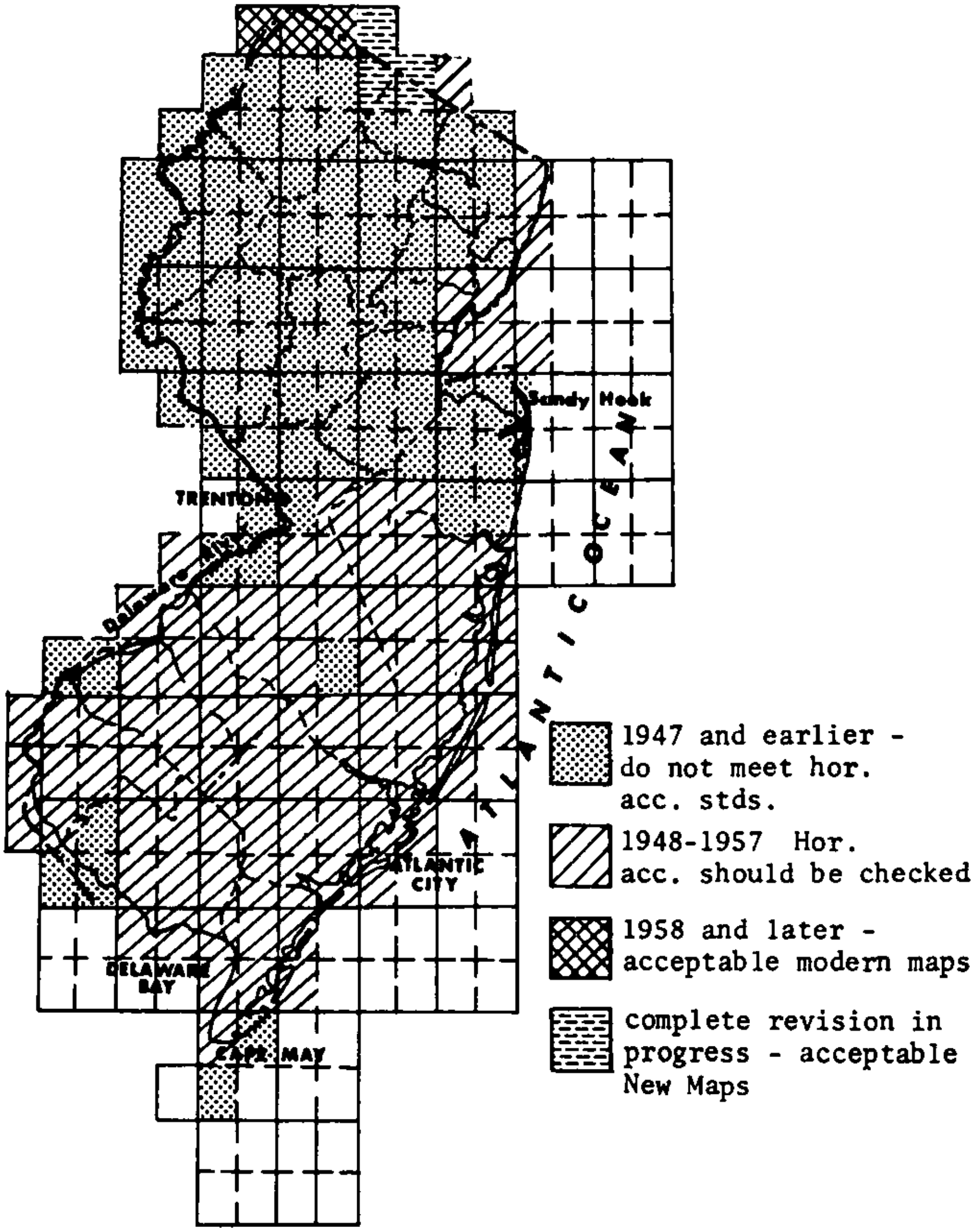

Fig. 3

In the new intermediate-scale map series (1:100,000 scale), 5 county maps and 1 quadrangle planimetric map are currently in work.

In the special mapping category we prepared, at State request on a repay basis, 172 7.5-minute slope maps, each showing 5 slope categories. While the maps were not published, reproducible copy was provided to the State for diazo reproduction.

The upgrading of the State's 7.5-minute map coverage to meet requirements as cited above is a difficult and complex problem. High national priorities to provide a variety of maps for environmental, energy, and urban programs, for example, require use of all available resources in terms of dollars, personnel, and mapping capacity. USGS is exploring with the State of New Jersey the development of plans and strategies that can advance the updating and improve most of the map coverage for the State, which is so much needed.

Conclusion

The National Mapping Program is in a state of rapid development in an attempt to meet the demands for cartographic products and data that continue to expand in magnitude as well as in detail and variety of products. Many successes have been attained, but much remains to be done to achieve the quality of basic cartographic data that are needed. We will find and apply solutions to problems like those concerning the quality of quadrangle maps for New Jersey.

SURVEYING THE TIDAL BOUNDARY

by

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INTRODUCTION

The tidal boundary, being a legal property line, can only be mapped by a professional land surveyor. This responsibility is that age old duty of a boundary line surveyor: to locate property lines. The surveyor in this case, however, must possess additional education and experience in order to employ the specialized techniques necessary to an accurate determination of the tidal boundary. This paper is designed to provide an understanding of the Legal Concepts, the Tidal Phenomenon, Tidal Characteristics, Tidal Datums, The Determination of Tidal Datums, Past and Present Methods to Delimit the Boundary, and a Discussion of some Sophisticated Techniques.

THE TIDAL BOUNDARY

The tidal boundary can be defined as the intersection of a tidal datum with the land. The datum to be employed is dependent upon State Law, and must be determined in relation to a 19 year tidal epoch. This definition is to the latest refinement of the original concept, which is rooted in English common law. The refinements are the result of court decisions which have continually sought a finer definition to eliminate misunderstanding. This is proper because, as all good boundary surveyors know, it is the courts which control property boundaries, not surveyors.

LEGAL CONCEPTS

Surveying, in addition to being the science of measurement, is also the art of boundary line location. Historical surveying methods and materials, the rules of evidence, previous court decisions, and like precepts govern the actions of the surveyor. In fact, it has been said that the surveyor holds a "field court" each time he surveys a boundary line. It is, therefore, appropriate to give some thought to the legal concepts involved in the tidal boundary.

The title to lands subject to the ebb and flow of the tides can be traced back to old English doctrine which placed ownership with the King. This has been accepted since the publication of Lord Chief Justice Hale's De Jure Maris (By the Law of The Sea) in the late Seventeenth Century.[1/] English courts, in Attorney General v. Chambers (1854), enunciated the common law and defined Hale's "ordinary" tides to be the medium high tide between springs and neaps.

Following the Revolution, the original colonies became vested with the title to the lands within their boundaries which were subject to the ebb and flow of the tide. All States joining the Union were given equal footing with the others and, therefore, have similar rights.

In 1935, the United States Supreme Court, in Borax Consolidated, Ltd. v. Los Angeles, (296 US 10), defined the "ordinary high water mark" as the mean of all the high waters over an 18.6 year period (the complete nodal cycle of the moon).[1/] This was in accord with the 1927 edition of "Tidal Datum Planes" by the Coast and Geodetic Survey (see reference 3 for revised edition).

Subsequently, the New Jersey Supreme Court, on November 6, 1967, restricted State of New Jersey claims to the area between the mean high tide boundaries of a given watercourse. This case, O'Neil v. State Highway Department clearly denounces the elevation test in favor of the tidal boundary test (mean high water where it strikes the land), eliminating claims to lands beyond the tidal boundary that are lower in elevation. This case also recognized the National Ocean Survey as the most authoritative source on the subject of tides.3/ Such recognition is consistent with that which was afforded the Cost and Geodetic Survey in the Borax case.

Our definition of the tidal boundary, the intersection of a tidal datum with the land, is consistent with the courts and, therefore, valid. Various datums are in current use. Mean high water, in concert with English common law, is in use in Alabama, Alaska, California, Connecticut, Florida, Georgia, Maryland, Mississippi, New Jersey, New York, North Carolina, Oregon, Rhode Island, South Carolina and Washington.2,4/ Mean low water has been adopted in Delaware, Massachusetts, Maine, New Hampshire, Pennsylvania, and Virginia.2,4/ The remaining States use variations of these: Texas uses mean high water for common law grants and higher high water for Spanish and Mexican Grants; Louisiana uses highest winter tide; and Hawaii uses the upper reach of the wash of the waves.2/

THE TIDAL PHENOMENON5,6/

Although the height of the tide is affected by wind and atmospheric pressure, the tide is a direct result of the gravitational forces of the Moon, and to a lesser extent, the Sun. An understanding of the astronomic forces which generate the rise and fall of the tides, as well as its variations in range, is mandatory to the proper determination of the tidal datum which will form the vertical component of the tidal boundary.

Isaac Newton, in 1687, was the first to mathematically formulate the fundamental cause of the long observed rise and fall of the tide. Newton's observations eventually lead to the equilibrium theory of tides. This theory is based on the assumption that the earth does not rotate, and that it is covered with water. The only celestial body having influence is the Moon. If these conditions were to exist, it was hypothesized, a bulge in the water surface would be evident on both the near and far surfaces of the earth, directly in line with the Moon.

This is explained by the equilibrium of the Earth-Moon system. The gravitational attraction between these bodies is balanced by the equal and opposite centrifugal force produced by the orbits of the earth and Moon about their common center of mass. Since this balance of forces exists only at the gravitational center of the bodies, an imbalance exists between the forces at any point on the surface. This difference in forces constitutes the tide producing force.

Returning to our ideal, water covered, Earth: at that point on the surface facing directly toward the Moon, the gravitational force exceeds the centrifugal causing the water to bulge toward the Moon. On the opposite surface (facing away from the Moon) the centfifugal force exceeds the gravitational, causing a similar bulge away from the Moon. Like forces cause the surface to subside at points which are at right angles to the Earth-Moon line (Figure 1).

Now, if we allow the earth to rotate, the water bulge will circle the globe in concert, accounting for the twice daily rise and fall of the tides. It should be noted here that the Moon's ability to "lift" the oceans, physically, against the overwhelming effects of the Earth's own gravity is nonexistant. The tide producing force acts to draw the water horizontally over the surface, propagating the tide as a forced wave.

The equilibriuim theory is not sufficient to fully explain the tides because it does not take into account friction, inertia, the shape of the ocean basins, the coriolis force resulting from the rotation of the earth, or other restraints. In addition, we must not overlook the effect of the Sun on the Earth's tides. Although considerably more massive, distance reduces the sun's tide producing effect to somewhat less than one half (about 46%) that of the Moon.

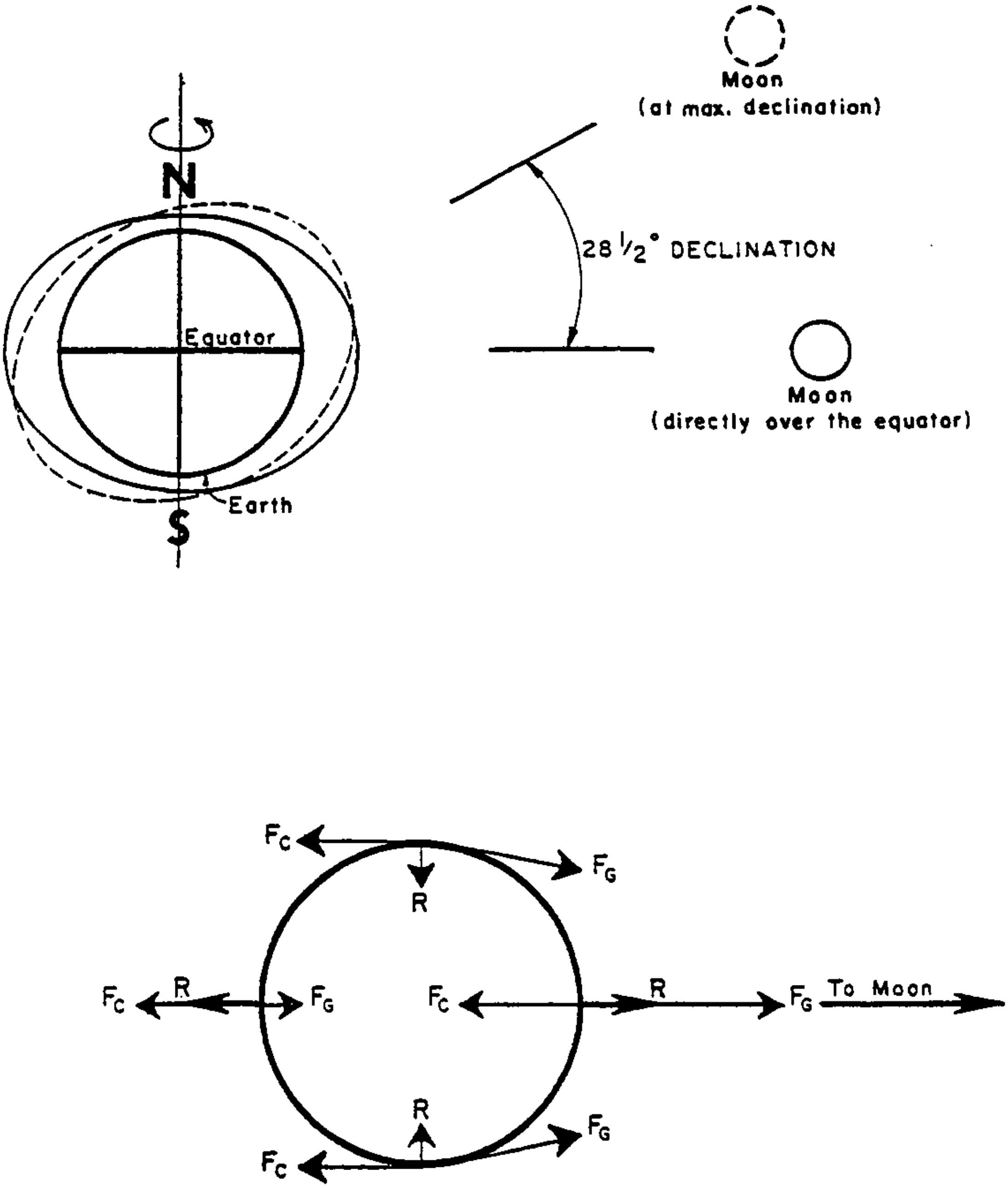
In addition, there are a wide range of astronomical variables caused by changing distances of the Moon from the Earth, the Earth from the Sun, the changing declination of the Moon, varying phase relationships of the Moon, and other smaller variations. The principle influences are as follows:

Parallax Inequality

Parallax Inequality results from the elliptical orbits of the Moon about the Earth and the Earth about the Sun. The changing distances cause variations in the gravitational force, which is greater at perigee (closest approach of the Moon) and perihelion (Earth's closest approach to the Sun). The period from one perigee to the next, the anomalistic month, requires 27.55 days. the period from one perihelion to the next, the anomalistic year, is nearly equal to the calendar year.

Phase Inequality

Phase Inequality results from changes in the phase of the Moon. At times of new and full Moon the tide producing forces of the Sun and the Moon act in conjunction, thereby increasing the range of tide. Such tides are known as "spring tides," a term that is not related to the season. Conversely, at quarter and three quarter phases (quadrature), the tide producing forces are opposed to each other, thereby decreasing the range of tide. The tides at quadrature are known as "neap tides." The phase cycle, the synodic month, averages 29.53 solar days. Because the Moon's orbit about the Earth is in the same direction as the Earth's rotation, there are only 28.53 lunar days in each synodic month. The lunar day then, on the average, is 24 hours and 50.47 minutes. Therefore, it is necessary to modify our twice daily rise and fall of the tides from every 12 hours to every 12 hours and 25 minutes.



F_G = Gravitational Force

F_C = Centrifugal Force

R = Resultant (tide producing force)

TIDE PRODUCING FORCES

FIGURE NO. 1

Diurnal Inequality

Diurnal Inequality results from the varying declination of the Sun and Moon. The Earth's axis maintains a constant tilt of 23-1/2° with respect to a line perpendicular to the plane of its orbit. During the year, the earth's orbit results in a declinational shift in the direction of the Sun's rays. The obvious result of the Sun's declination is the seasons.

The Moon, on the other hand, because of the small angular difference in orbital planes (5°) moves North and South each month about as much as the Sun does during the year. There are times that the Moon's declination reaches 28-1/2° (23-1/2° + 5°) and other times, 9.3 years later, when its declination does not exceed 18-1/2° (23-1/2° - 5°). A complete cycle takes 18.6 years and is known as the regression of the Moon's nodes. The Moon's declination moves through the earth's equatorial plane twice during a tropical month (27.32 solar days), that is from maximum North to maximum South and back. The effect of declination is to cause successive heights of tide to differ markedly in some areas.

These major variables, the anomalistic month of 27.55 days, the synodic month of 29.53 days, and the tropical month of 27.32 days, cause significant differences in the range of the tide due to the changing relationships of the Sun, Moon and Earth.

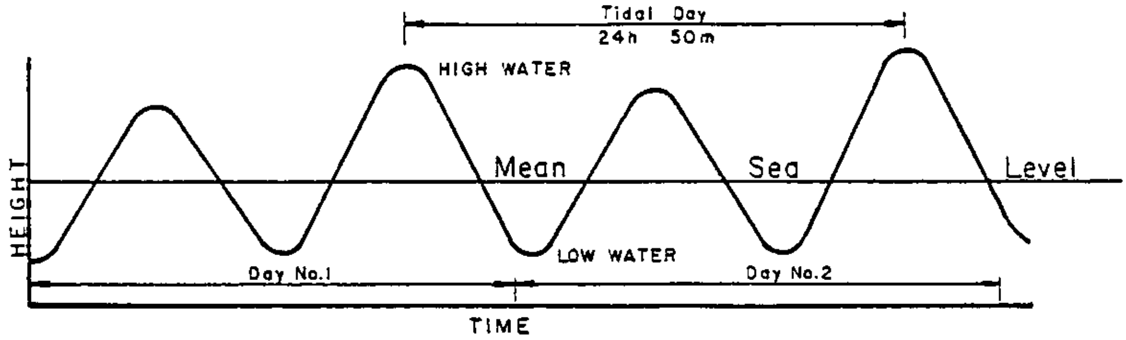
As a point of information, the greatest tidal force is generated when the Sun is in perigee, the Sun and Moon are in conjunction or opposition, and both bodies are at zero declination. This occurs about every 1600 years and will happen next around 3300 A.D.[5/]

TIDAL CHARACTERISTICS

The Moon, and to a lesser extent the Sun, provide the forces which control the propagation of the tide. If we consider the effects of friction and inertia, we can understand why the tide does not correspond exactly to the transit of the Moon, the phase of the Moon, the exact time of perigee, or the declination. In addition, the effect of the shape of the ocean basin causes large and varied differences in tidal range. Finally, as the tidal wave reaches the coast and enters the gulfs, bays and tributaries, the ever restricting shape of these bodies of water further modify the tide. Obviously, the rise and fall of the tides is an extremely complex natural phenomenon!

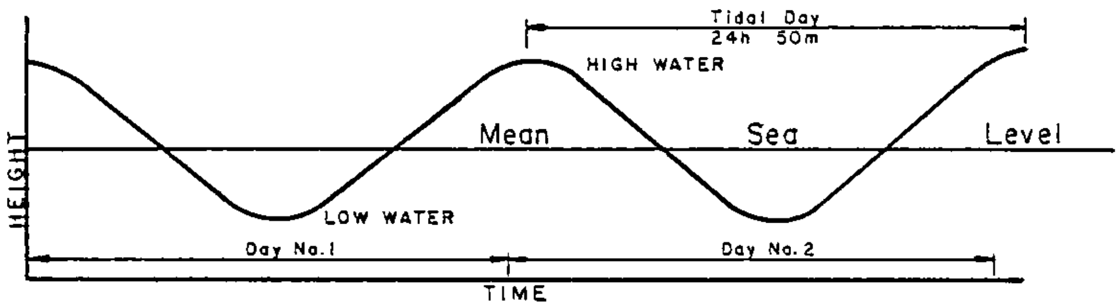
Certain generalities are, however, readily evident. For most of the world, including the East Coast of the United States, the tides rise and fall twice daily, in agreement with the lunar day of 24 hours, 50 minutes. Said tides are known as semi-diurnal or semi-daily. The range of the semi-diurnal tides are controlled, primarily, by changes in the phase of the Moon and by parallax. To a small extent, diurnal inequality is also a factor.[7/] (Figure 2A)

In some areas, there is but one high and one low tide per day. Such tides are known as diurnal and occur along portions of the Gulf Coast of



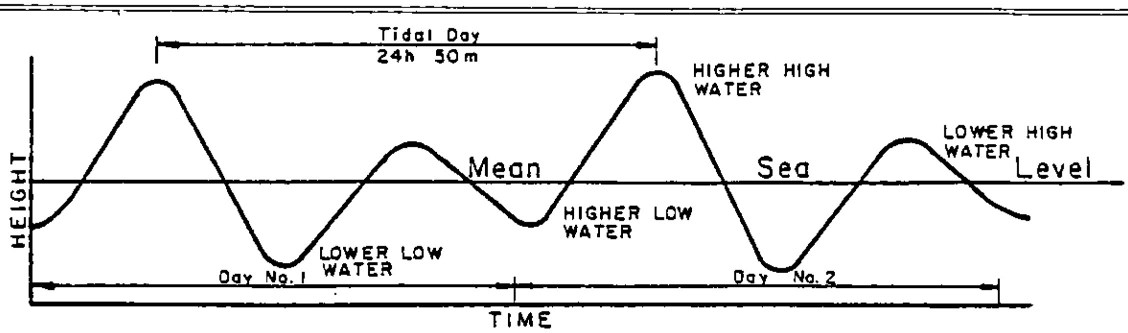
SEMIDIURNAL TIDE

FIGURE 2A



DIURNAL TIDE

FIGURE 2B



MIXED TIDE

FIGURE 2C

the United States and in Southeast Asia. In this daily type of tide the variation in range is principally in connection with the changing declination of the Moon.7/ (Figure 2B)

The transition from the semi-diurnal to the diurnal type of tide accounts for the mixed tide, characterized by two high and low waters each day having marked inequalities in height. Mixed tides result from the interaction of daily and semi-daily tidal forces and may be manifested by inequality principally in the high waters, the low waters, or both. These tides are common throughout the Pacific Coast of the United States, as well as Alaska, Hawaii and portions of the Gulf Coast.7/ (Figure 2C)

TIDAL DATUMS

A datum is a reference from which to make measurements. In tidal usage the datums define various water levels. The period of observation for each datum is 19 years. This is the Metonic Cycle of 235 lunations, which includes the 18.6 year cycle for regression of the Moon's nodes, and other important astronomic variables. The period also helps to even meteorological variables and provide an easily computed base. This specific 19 year cycle is known as a tidal epoch. As a point of information, consecutive epochs show a steady rise in sea level, relative to the land, which may be due to an even longer background cycle, glacier melt, land subsidence, or a combination of these factors. As such, a specific 19-year epoch, known officially as the National Tidal Datum Epoch, has been adopted. The present National Tidal Datum Epoch is 1941 through 1959. The National Ocean Survey reviews the Epoch for possible revision every 25 years.

Several tidal datums are in common usage. These may be defined as follows:8/

Mean Sea Level

Mean Sea Level is the arithmetic mean of hourly water elevations over a specific 19 year metonic cycle. This datum, until recently, formed the reference for all leveling in the United States.

Mean Tide Level

Mean Tide Level (sometimes known as half tide level) is the average between mean high and mean low waters. This should not be confused with mean sea level, which it is not.

Mean High Water

Mean High Water is the average of all high waters over a specific 19 year period.

Mean Low Water

Mean Low Water is the average of all low waters over a specific 19 year period. This datum is important because it forms the "chart datum" to which all soundings for east and Gulf coast charts are referenced.

Mean Lower Low Water and Mean Higher High Water

Mean Lower Low Water and Mean Higher High Water are averages of the lower low or higher high waters over specific 19 year periods, in those areas where a mixed tide predominates. Mean lower low water forms the chart datum in those areas.

DETERMINING TIDAL DATUMS

Having an understanding of tide producing forces and tidal characteristics, we must now address the problem of obtaining an acceptable datum to fix the vertical component of the tidal boundary. Setting a tide gage for 19 years would, by definition, provide the data, but this would not be practicable for even the largest projects. Fortunately, methods have been devised to overcome this problem.

The National Ocean Survey (NOS) maintains a network of primary tide stations, some of which have been in operation since late in the Nineteenth Century. By simultaneous comparison with a primary station it is possible to derive datum from a shorter series of observations. The accuracies to be expected vary with the length of the observations. For example, the East Coast:[7,9]

Length of Observational Series	Expected Accuracy
Day	0.25'
Month	0.13'
3 Months	0.10'
6 Months	0.07'
Year	0.05'
3 Years	0.03'
9 Years	0.016'

Tide stations where datums have been derived by a short (less than 19 years) series of observations are known as subordinate stations. In addition to data on primary stations, NOS maintains data on a large network of subordinate stations, the datums for which are referenced to nearly tidal bench marks.

It should be noted that the simultaneous comparison of observations is expedited by computations involving automated techniques. The alternate method for reduction of simultaneous comparison for a period of a few days is, however, a simple task, as outlined in NOS Technical Report Number 64.

Another practical approach to establishing a local datum is the extrapolated water elevation (EWE) method. The procedure was devised by Jack Guth (NOS retired) and extensively tested in the State of Florida. In terms of expediency, this method is of great use to the surveyor because it is relatively simple, does not require simultaneous comparisons, and can be completed through a portion of one to three tidal cycles.

Extrapolated Water Elevation Method[4]/

To extrapolate a tidal datum, several conditions must exist. Initially, assuming mean high water is the required datum, it is necessary to have a tide that will equal or exceed mean high water. Reference to NOS tide tables will yield a prediction as to when this condition can be expected.

Secondly, an established tide station must exist in reasonable proximity to the project site. This is a judgment determination that should be carefully evaluated. Both sites should be in the same body of water and must possess similar topographic, tidal, and meteorological conditions. Several States have undertaken programs to increase the density of tide stations in an effort to assure that there will be an available station within reasonable proximity to any site.

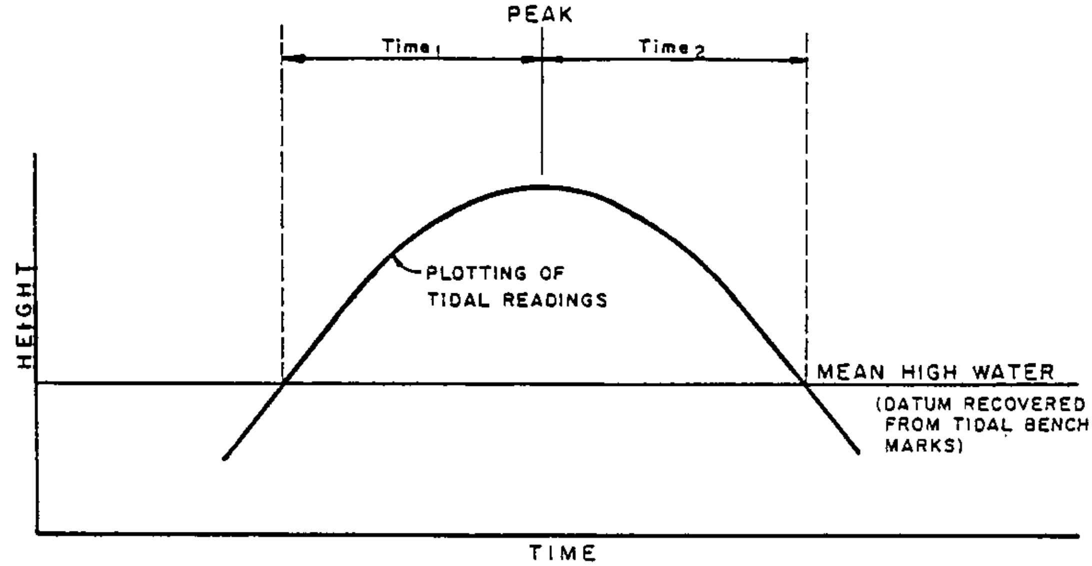
Next, a sufficient number of project staffs must be set. Here again, the actual number is dependent upon experience and judgment. It is imperative that the surveyor keep in mind that there may be significant differences in datums on opposite sides of islands, points of land, jetties, and similar features, as well as the opening and headwater of streams. Project staffs may be long 2 x 4 timbers with folding rules affixed, drived into the ground just seaward of the location where mean high water is expected to intersect the land (but open to the water). The project staffs should, if at all practical, be connected to the National Geodetic Vertical Datum by second order leveling. If wave action limits the ability to read the staff, a stilling device, such as a clear plastic tube, can be connected to the staff.

For accurate measurements, weather conditions must be good, especially when the project staffs are at a distance from the control staff. Wind conditions and other meteorological conditions should be carefully evaluated.

When all conditions are favorable, staff readings are simultaneously recorded on the control and project staffs. The readings should begin early, while the tide is below the level of mean high water. At intervals of three minutes, readings are recorded through the rising tide, to the crest, and until the tide has fallen below mean high water again.

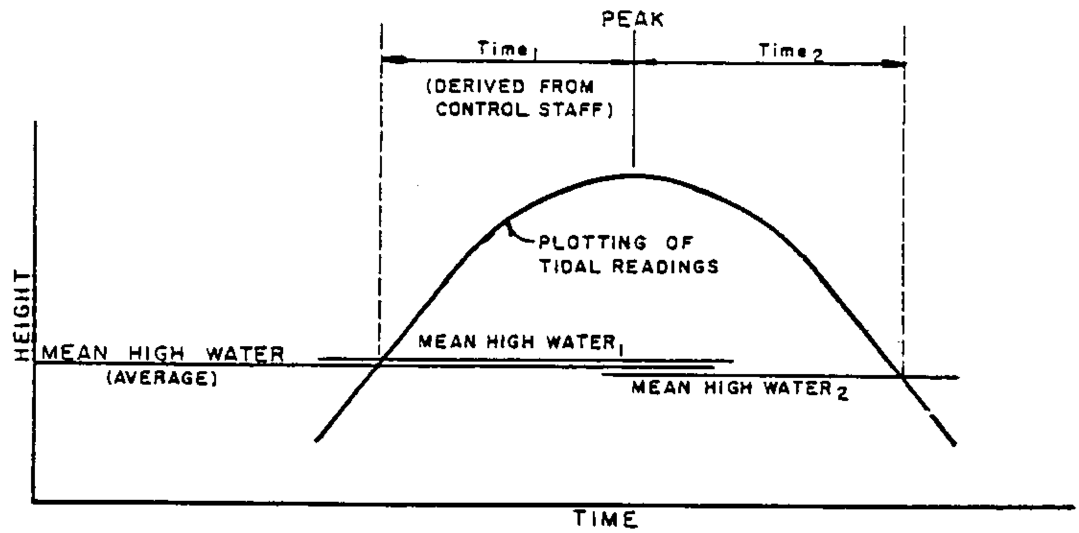
A comparison of the time intervals between the datum of mean high water and the peak of the crest, both at the project and control staffs, will result in the derivation of the local tidal datum. As shown in (Figure 3A) (the control staff) the staff reading that corresponds to mean high water is known (by connection to the tidal bench marks) as is the time of the peak staff reading. Time intervals, both on incoming and outgoing tides, can easily be computed.

At the project staffs (Figure 3B) the time of the peak staff reading is known and, by applying the time intervals as derived from the control staff, the elevation of mean high water can be read. Minor differences is elevation resulting from the incoming and outgoing tides may be averaged.



CONTROL STAFF

FIGURE 3A



PROJECT STAFF

FIGURE 3B

The resultant elevations on the project staffs are the local tidal datums, and may be relied upon to form the vertical component of the tidal boundary. If a primary tide station was not used the results are subject to the tolerances of the subordinate station employed (depending on the length of the collection period), as well as the probability of small errors in the extrapolation process. The accuracy of the extrapolation process has not yet been quantified, to the Author's knowledge. For increased accuracy it may be advisable to extrapolate the elevations two or three times. If extreme accuracy is required a tide gage should be installed.

PAST AND PRESENT METHODS TO DELIMIT THE TIDAL BOUNDARY

In the past, great precision in locating the line of sovereign/upland ownership was not generally necessary. Therefore, approximate procedures were normally employed. These included location of the shore line, or the limit of vegetation. For increased accuracy, the height of the tide from a short series of observations was used, without simultaneous comparisons.2/

Since the late 1950's, the growing conservation movement has forced a more accurate determination of the actual ownership of the tidal marshlands. Many surveyors began to use the contour method, whereby a tidal datum was derived and the resulting contour mapped throughout the property. This may be reasonable, but the tidal datum is not necessarily level throughout an area. Although more accurate than previous methods, it was obvious that more precise procedures were necessary.

Today, it is necessary to couple the accurate determination of the local tidal datum with conventional surveying techniques to actually map the tidal boundary.

Although more precisely defined by longer term tidal studies, the expeditious EWE method is a recognized procedure for determining the vertical component of the tidal boundary, as is the alternate method of simultaneous comparisons described in NOS technical Report No. 64. Interpolation, providing the tide range difference between respective gages is small and the shoreline unobstructed, is also a recognized procedure. Assuming an adequate number of points to properly define the local tidal datum, the surveyor must then locate and map the boundary on the ground.

Initially, the surveyor must establish a control survey, both horizontal and vertical, accurate to second order standards. This survey, and all the other project boundaries, should be tied to the State Plane Coordinate System if at all economical. Vertically, the network should be referenced to the National Geodetic Vertical Datum and all applicable tidal bench marks connected by levels. Using the control network, it will be possible to physically record the location of the tidal boundary as it is determined on the ground.

Beginning at a project staff, the surveyor progresses inland, checking elevation regularly, until the boundary is found. He then follows this line, progressing generally toward the next project staff. It is important to "walk" the boundary to eliminate the possibility of unknown inlets or depressions.

In all probability, the elevation of the boundary line will not be level. In that event, the difference should be apportioned to the distance between the project staffs. With practice, a surveyor will be able to "feel" the boundary and the mapping can be expedited. As additional proof, elevations seaward and landward of the boundary should be recorded.

Normally, stadia methods are satisfactory for recording the location of the tidal boundary from the control network.

1:500 accuracy can easily be maintained horizontally, and 0.05 feet vertically. In some instances a traverse, run roughly along the boundary, might be more practicable. Other procedures to locate the physical boundary may also yield relevant data. The experience and originality of the surveyors are the only bounds in this area.

The resulting plat of the survey is the record of the tidal boundary. This boundary is "monumented" by the State Plane Coordinate System and by ties to the other property boundaries.

SOPHISTICATED TECHNIQUES

There are other acceptable ways to locate the tidal boundary, if conditions are favorable. These include physically staking the edge of the water at the exact moment of mean high water and the use of tide coordinated aerial photography.

To physically stake the edge of the water, it is necessary to observe the tide at each project staff. When the datum is reached (on a rising tide) stakes are driven at the waters edge. This procedure, however, may necessitate a great number of additional project staffs and a considerable expenditure of manpower because vegetation often severely limits the mobility of the observer. Inasmuch as the results should closely approximate those of the previously described method, its value is a matter of economics and practicality.

Tide coordinated aerial photography must be of high quality and should be interpreted by qualified personnel. Normal color photography, false color infrared and black and white infrared seem to be the most useful in tidal marshes. The procedure requires air-to-ground communications between the aircraft and a ground observer at the tide staff. Coordination between these ground and air parties is essential to bring the flight over the project at the exact moment of mean high water.

On open beaches, this procedure is unbeatable for a clear determination of the water/land interface. Disadvantages occur, however, when this interface is obstructed by vegetation. This problem can be overcome by cutting transects from the upland to the water's edge. The location of the intersection of the datum and the land is found for each transect and a panel deployed for photo-identification. Following the mapping of the line from the photography, additional checks are made in areas heretofore unsurveyed. If a high correlation is found, a logical assumption can be made that the mapping is accurate.10/ Without correlating ground truth this method (or any other remote sensing technique), can not be considered accurate. In addition to economical considerations, the employer of this procedure may have a "proof" problem should the matter reach a court of law.

In recent years, several scientists have experimented with methods of locating the tidal boundary based on vegetation. This "biological mean high water line" should be approached with extreme caution because it does not meet the legal definition of the tidal boundary. As discussed earlier, the intersection of the water surface at the elevation of the local tidal datum with the land, based on a specific 19 year tidal epoch, is the tidal boundary, irrespective of the indigenous vegetation.

Further, elevation alone is an inconclusive test of validity. Vegetation growing interior of the boundary, even though it may be below the datum, is not to be considered. Furthermore, it can not be proven that a particular plant is responding to a short term tidal range or the 19 year mean. Nor can it be demonstrated that a plants normal growth cycle has not been altered by the effects of pollution, or that its image in the photography has not been confused by other outside stimuli. Obviously, this technique should be confined to general delineations of the coastal zone and not applied to land boundaries.

CONCLUSION

The tidal boundary, being a legal property line, falls within the exclusive purview of the professional land surveyor. To properly demarkate the tidal boundary, however, the surveyor must possess additional education and experience.

Some sophisticated techniques are available to assist the land surveyor while others must be approached with caution. The only acceptable demarkation of a tidal boundary is the accurate determination of a local tidal datum coupled with conventional surveying techniques to map the boundary.

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TIDAL DATUM AND MARINE BOUNDARY SURVEYS

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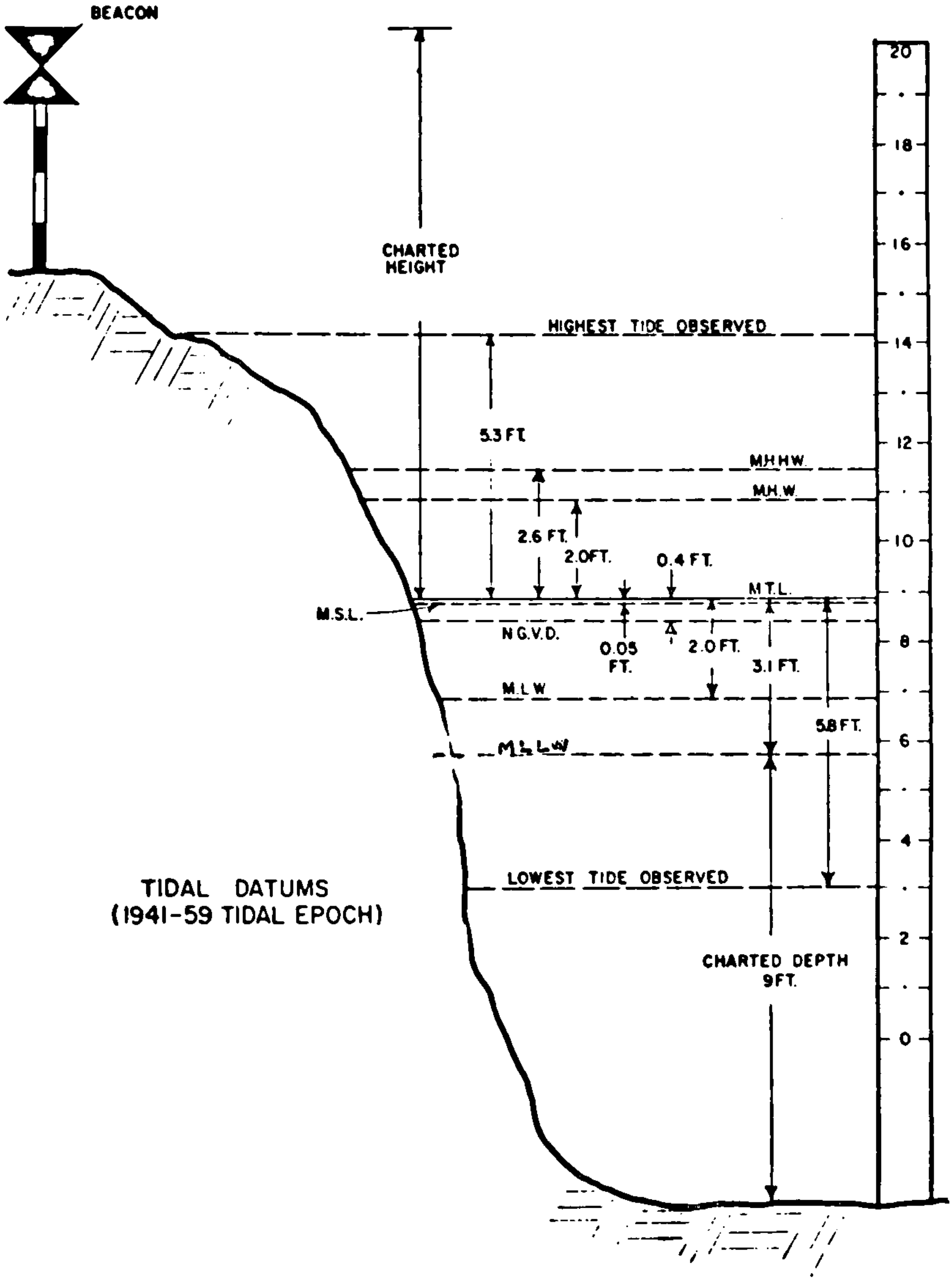


Figure 1.--Chart datum at the Presidio, San Francisco, California.

The determination of marine boundaries in the delimitation of private property and sovereign lands for ownership and jurisdictional purposes has been a universal problem in coastal countries for many decades. The use of tidal datums to define these baselines has long been the accepted method for locating these boundaries. However, the information require for accurate demarcation of these baselines in most cases has been very limited, or more often, non-existent. The following discussion is presented to provide a brief review of the history of the basis for marine boundaries, the derivation of tidal datums and their application in resolving some of the paramount issues concerning the coastal wetlands today.

The National Ocean Survey (NOS) and it precedessor, the Coast and Geodetic Survey (C&GS) have been measuring the rise and fall of the sea and computing tidal dataums since the early 1800's.

The "Survey of the Coast", as it was originally called, is the oldest scientific organization in the United States. It was established by President Jefferson under an Act of Congress on February 10, 1807 which authorized the President to "cause a survey to be taken of the coasts of the United States..."

One of the basic requirements in surveying the coasts was to locate and map the line of intersection of the land and the sea. In addition, since the elevation of the sea surface is continually changing, the depths of the water surrounding the coast must be based on a common reference for elevation in order to be meaningful to the mariner. Therefore, the establishment of chart datums which could be utilized as the reference for a common zero elevation was a necessity. These datums have since become the reference points for the determination of marine and coastal boundaries (see Fig. 1).

Marine and Boundary Lines

The use of water levels and tide levels to describe and define marine boundaries and property lines dates back to Roman times. When the United States was formed, the traditional English rule regarding marine property lines was adopted by the colonies and states--the "ordinary high water marks" was the boundary between sovereign lands and private property. However, because the definition of "ordinary tide" was vague and imprecise, marine boundary conflicts generally resulted in litigation.

One of the landmark cases on the question of marine boundaries was rendered by the Supreme Court, in 1935 in the case of Borax Consolidated, Ltd. vs. The City of Los Angeles. It specified that definitions, methodology and procedures of the C&GS to determine mean high water would be used in Federal cases involving marine boundaries; that mean tide elevations must include the average of 18.6 years of all observations. Further, a specified water line was said to be the intersection of the elevation of the datum with the shore.

Seaward Boundaries

Three Supreme Court decisions between 1945 and 1950 involving the rights to submerged lands outside the inland waters of California, Louisiana, and Texas, specified that the Federal government, rather than the States, had the paramount rights over the offshore lands in the 3-mile belt, including resources under the seabed (Fig. 2).

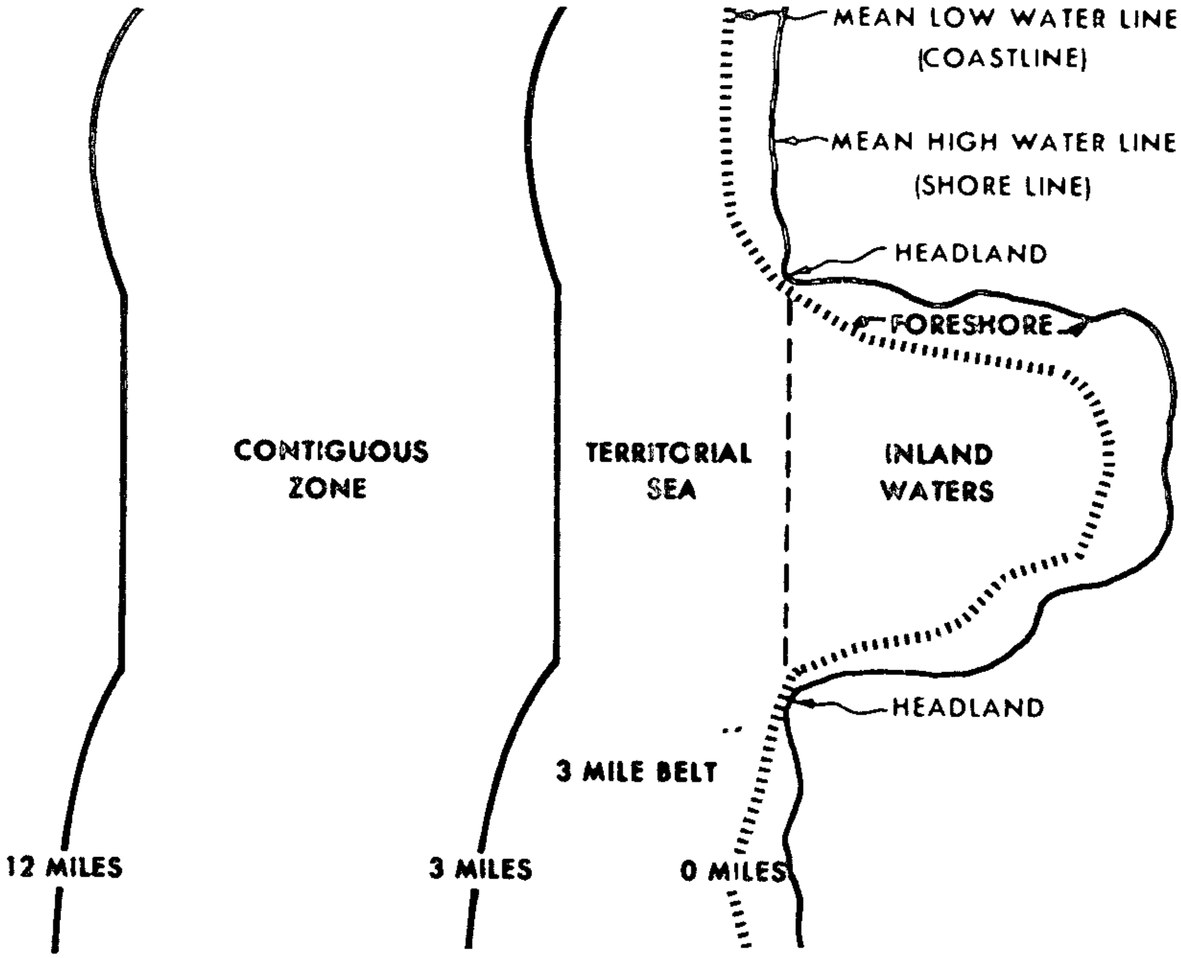


Figure 2.--Seaward boundaries for offshore State lands.

Then in 1953, the Submerged Land Act was passed which granted lands beneath navigable waters to the States, reversing the 1947 Supreme Court decision. At the same time, Congress enacted the Outer Continental Shelf Lands Act which claimed for the United States, the submerged lands seaward of the 3-mile belt.

In 1965, the Supreme Court ruled that the baseline for delimiting the boundaries of submerged lands, granted to states is the same baseline from which seaward boundaries are measured according to the 1958 Geneva Convention.

The Coastal Zone Management Act of 1970 specifies the outer limit of the territorial sea as the seaward limit of the coastal zone, and thus requires delineation of the baseline to describe its area of responsibility.

In 1975, the Supreme Court ruled that the Federal government has sovereign rights over the submerged land beyond the 3-mile belt to the outer edge of the continental shelf in a case against 13 coastal states bordering the Atlantic Ocean. The Fisheries and Conversation and Management Act of 1976 extended the U.S. jurisdiction to 200 nautical miles from the U.S. coast. This act requires that the boundary be accurately defined and adequately delimited to inform other nations of the point to which the U.S. claims jurisdiction.

Implementation of other Federal statutes, including the Rivers and Harbors Act of 1899, and the Federal Water Polution Act of 1972 are dependent on tidal datum baselines.

Within recent years an unprecedented interest has been created in the coastal zone and the nation's wetlands. The desire for marine-oriented development and the ecological preservation of coastal marshes have resulted in major conflicts. Officials of both State and Federal agencies have been presented with arguments they could not technically resolve.

In coastal estuaries with irregular shapes, restrictions, obstructions, unusual bathymetric features and heavy vegetation, relatively large differences in tidal datums will be found from one place to another in a given area. Even in the same estuarine system, changes in the slope of the water surface may cause large differences in tidal datums around a prominent point of land or in connecting tributaries (Fig. 3).

The accurate determination of tidal datums and boundary demarcation are extremely important in determining ownership of valuable land. For example, a vertical error of 0.1 foot in datum location along a beach with a 2 percent slope can result in an offset of 6 feet in the boundary lie demarcation on ashore. With a 0.1 percent slope the offshore could be 115 feet. In some cases the horizontal displacement error could involve many acres of shoreline property. It could also involve large areas of valuable submerged lands, since offshore boundaries are measured from tidal datum baselines.

The value of a tidal datum as a reference can be accredited to the simplicity of its definition, the accuracy of its determination and the certainty with which it can be reproduced at some future time. However, in order to relate tidal datums as baselines, certain aspects of the tidal phenomena should be understood.

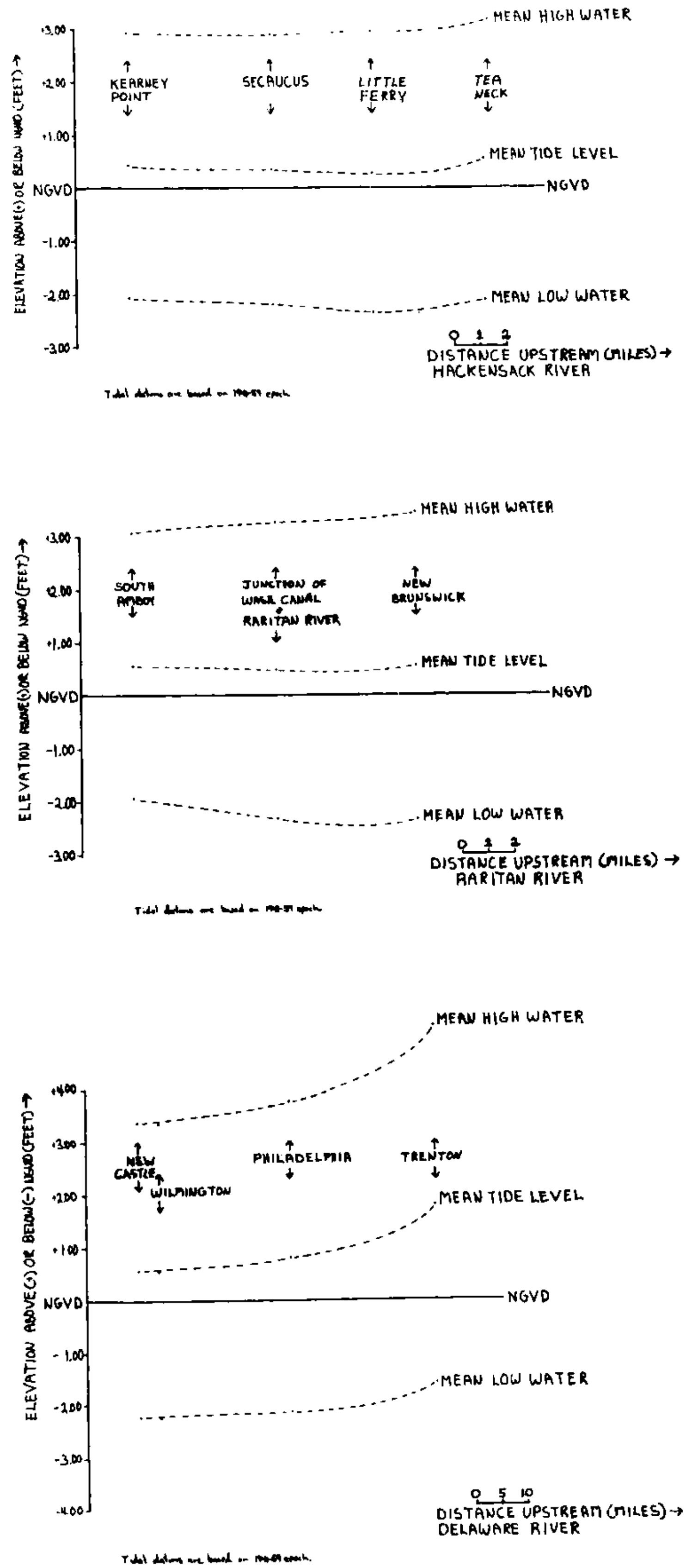


Figure 3.--Relationship of tidal datums to NGVD along several New Jersey rivers.

Tide is defined as the periodic rise and fall of the water (see Fig. 4) on the Earth's surface as a result of the changing gravitational interactions between the Sun, the Moon, and the rotating Earth (Fig. 5).

The tidal effect at any given place depends on the relative positions of these three bodies at any particular time. Since the tide producing forces of the Moon are slightly more than twice that of the Sun, changes in the tide are more readily identified with th variations in the lunar orbit. However, the Sun's role in tidal fluctuations cannot be ignored.

When one considers the different time periods for the Sun and the Moon to complete their various cycles which all exert distinct influences on the total tide producing force, it is evident that the length of tidal measurements must be selected to consider the combined effects of these time periods.

The length of the solar day is 24 hours while the length of the lunar day is 24 hours and 50 minutes. The Earth completes one orbit about the Sun every 365 1/4 days. During this time, the Sun completes one declination cycle. That is, relative to the Earth the Sun moves from a maximum south position to a maximum north position, and back to the south. These changes are related to the seasons. In addition, the orbit of the Earth about the Sun is an ellipse so that effects of maximum and minimum distance from the Sun must be considered. During the year, the moon has completed 13 orbits about the Earth. The phases of the Moon repeat every 29 1/2 days and the Moon's declination cycle has a period of 27 1/3 days. Due to its elliptical orbit about the Earth, the Moon also has a maximum and minimum distance cycle of 27 1/2 days. It is therefore readily seen that there are an infinite number of combinations which can be attained to produce varying degrees of tide producing force.

A series of 19 years is considered as constituting a full tidal cycle, for during this period of time the more important of the tidal variations will have gone through complete cycles. This 19 year cycle was first described by Meton, an Athenian scholar in the 5th century before Christ as a period in which an exact 235 lunar orbits are completed.

The advantage of the Metonic cycle for the determination of primary tidal datums can be seen in that it includes the 18.61 year period which is required for a complete 360o revolution of the positions where the Moon's orbit intersects the plane of the Earth's orbit about the Sun (Fig. 6). Additionally, it includes complete annual sea level measurements during which seasonal variations are significant (Fig. 7).

Non-periodic changes in tidal heights such as wind, barometric pressure, fresh water runoff, etc., do have a pronounced local effect on water elevations, particularly in estuarine areas. While the measurements of these heights are included in daily observations, the 19 year means have the advantage of averaging out these short-term irregularities (Fig. 8).

The movement of the tide in the ocean has been described as a wave which radiates around amphidromic points where the tide does not change. These are described by lines radiating from these points, and along these lines, high waters will occur simultaneously with amplitudes of 1- to 3- feet (Figure 9). However, as these waves approach the coastlines of the continents, changes in times may be observed, and heights increased up to 40 feet.

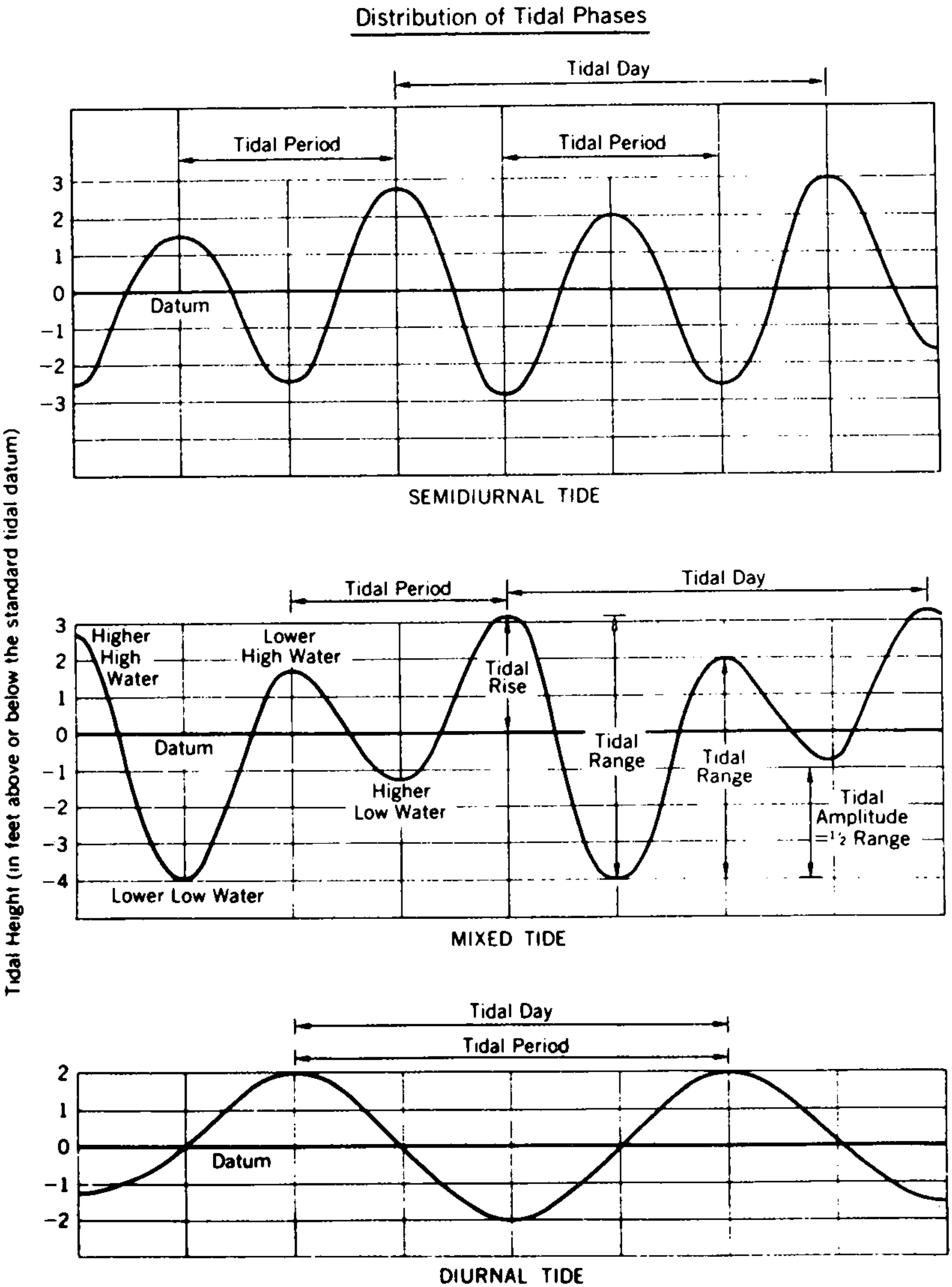
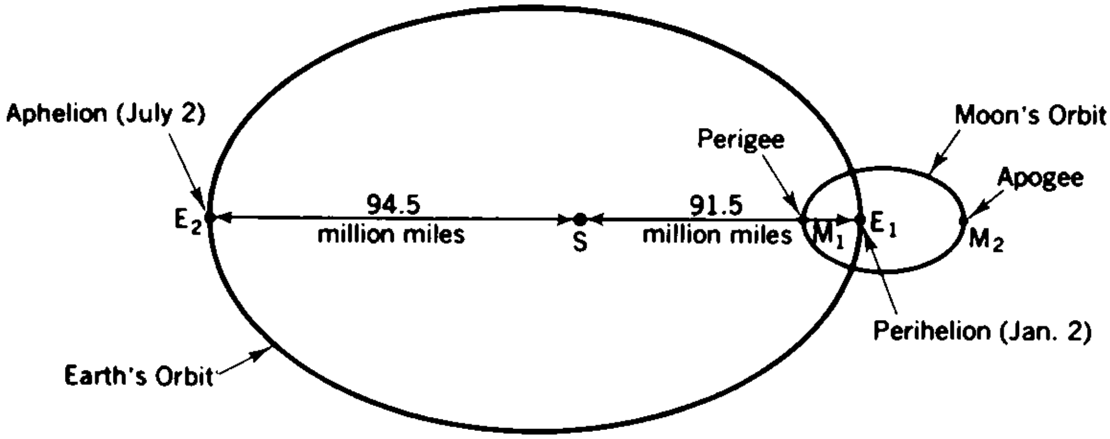


Figure 4.--Principal types of tides: Showing the moon's declination effect in production of semidiurnal, mixed, and diurnal tides.



Common projection of the earth's orbital plane around the sun (the ecliptic) and the moon's orbital plane around the earth.

S = Sun
E_1 = Earth at perihelion (Jan. 2)
E_2 = Earth at aphelion (July 2)
M_1 = Moon at perigee
M_2 = Moon at apogee

Both the moon and the earth revolve in elliptical orbits and the distance from their centers of attraction vary. Increased gravitational influences and tide-raising forces are produced when the moon is at position of perigee, its closest approach to the earth (once each month) or the earth is at perihelion, its closest approach to the sun (once each year). This diagram also shows the possible coincidence of perigee with perihelion to produce tides of exceptional range.

Figure 5.--The lunar parallax and solar parallax inequalities.

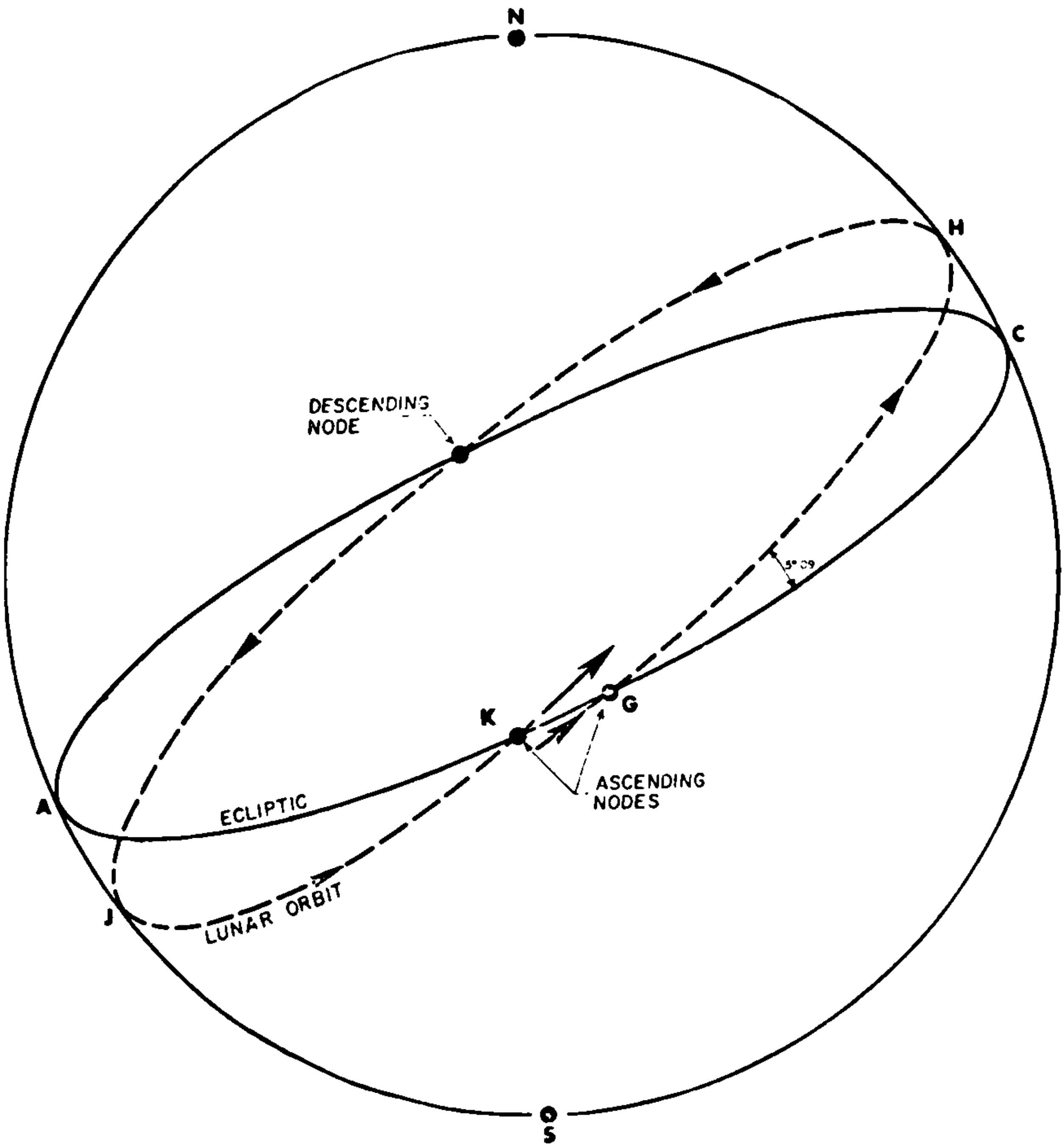

N
H
C
DESCENDING
NODE
5° 09
K
G
ASCENDING
NODES
A
ECLIPTIC
J
LUNAR ORBIT
S

Figure 6

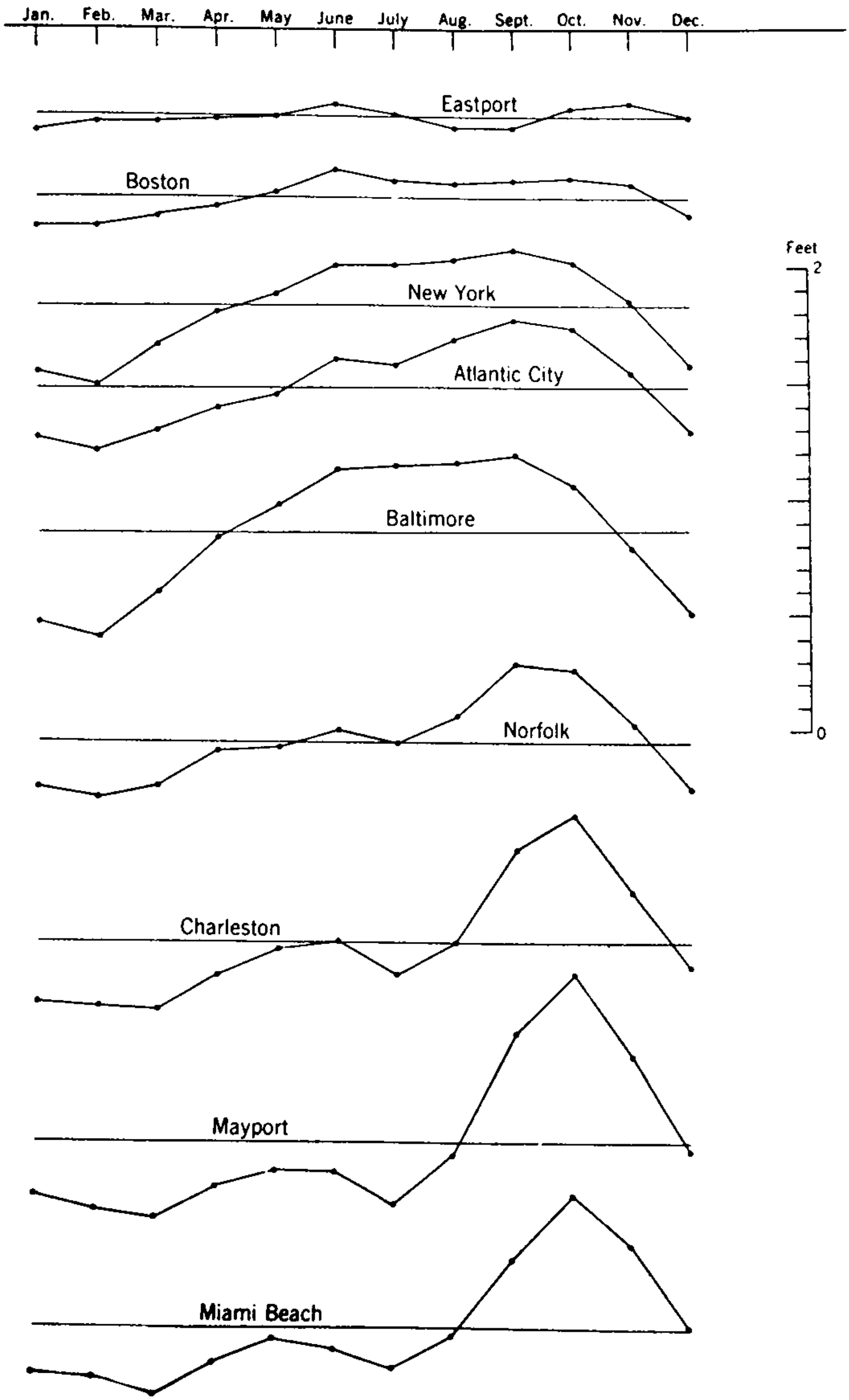


Figure 7.--Annual variation in sea level, Atlantic coast.

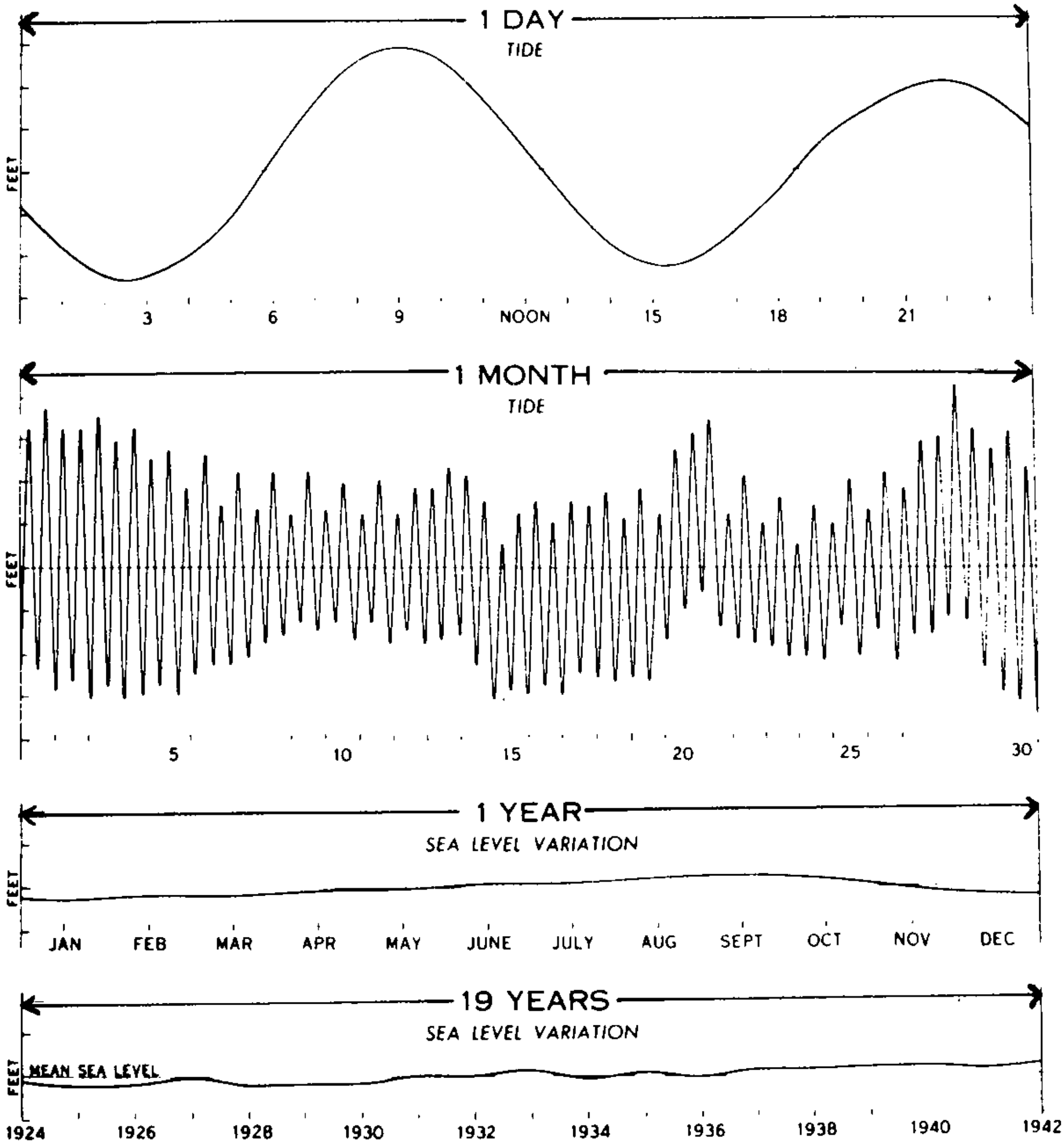


Figure 8.--Sea surface variation from Coast and Geodetic Survey tide station at Atlantic City, N.J.

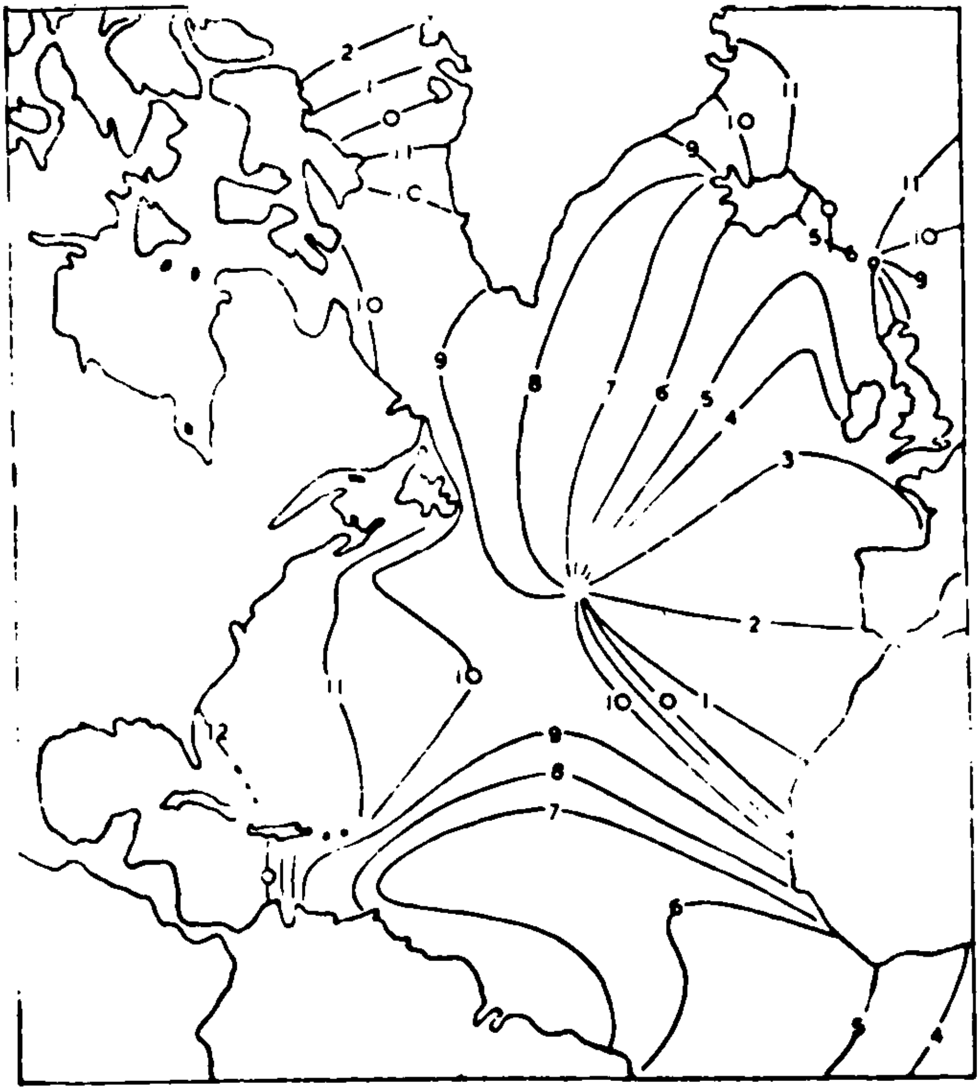


Figure 9.--Cotidal lines Atlantic Ocean (Harris 1904).

While some generalization can be made about tide behavior, it is a local phenomenon. The sea surface is not level--it undulates according to many forces and restrictions (Fig. 3). Consequently, a description of the tide in one area may not be the same as the tide in an adjacent area.

To accurately determine tidal datums, a number of short-term tide measuring stations may be required in relatively close proximity (Fig. 10). To acquire tide level data, NOS employs three kinds of tide observation stations. These are classified as Primary Control, Secondary Control and Tertiary stations.

The primary control stations are permanent instllations which are operated on a continual basis and form "The National Tide Observation Network" (Figure 11). There are now over 130 stations in this network, two of which have been recording tide level continuously for more than 100 years.

Primary control tide stations provide continuous measurements through the 19-year Metonic cycle--the span of time required to compute mean values and determine accurate tidal datums.

A primary control tide station may be equipped with various types of instruments such as:

1. An analog tide gage which records the rise and fall of the water level as a continuous senusodial curve (Fig. 4);

2. An analog-to-digital tide gage which converts the analog curve to digital format for automatic data processing.

3. A pressure gage which records the rise and fall of the water as a function of the change in the water column above the sensor.

In addition, there may also be thermographs to record temperature, salinometers to measure salinity, meteorological telemetry equipment to transmit sea and weather conditions to the National Weather Disaster Warning Centers, and other support equipment.

Subordinate Tide Stations

It has been established that 19 years of measurements constitutes a primary determination of a tidal datum at a given place. However, what about locations hydrographically remote from these control stations?

It was recognized, long ago, that it was not only impracticable but infeasible to obtain 19 years of measurements at every location along the shoreline to determine a tidal datum. Therefore, a method was developed whereby the equivalent of the 19 year value could be computed from a short series of measurements with a defined degree of reliablity. The accuracy of a tidal datum computed from a short series of measurement will depend upon the length of the series obtained and the availability of a suitable control station.

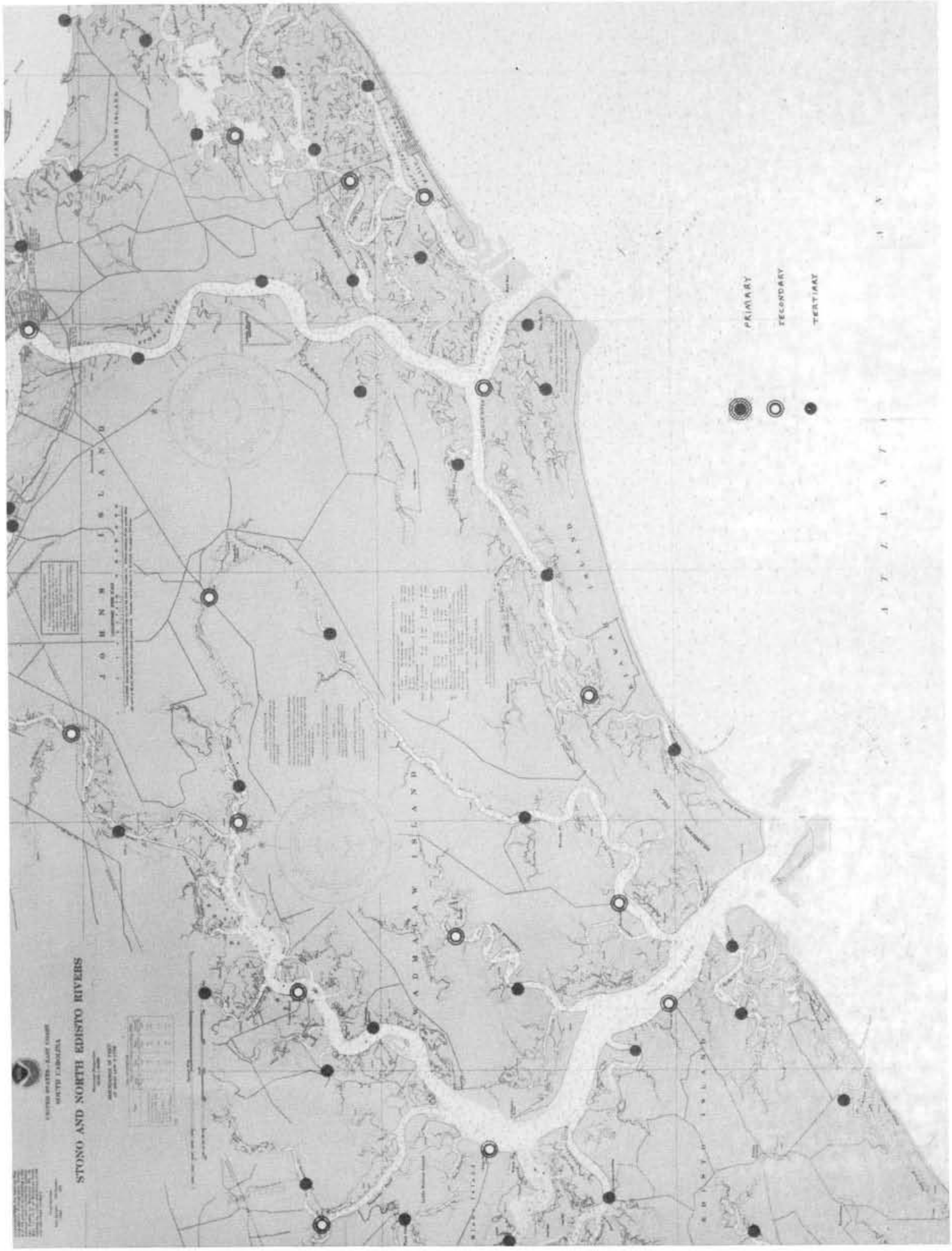

STONO AND NORTH EDISTO RIVERS
SOUTH CAROLINA
PRIMARY
SECONDARY
TERTIARY

Figure 10

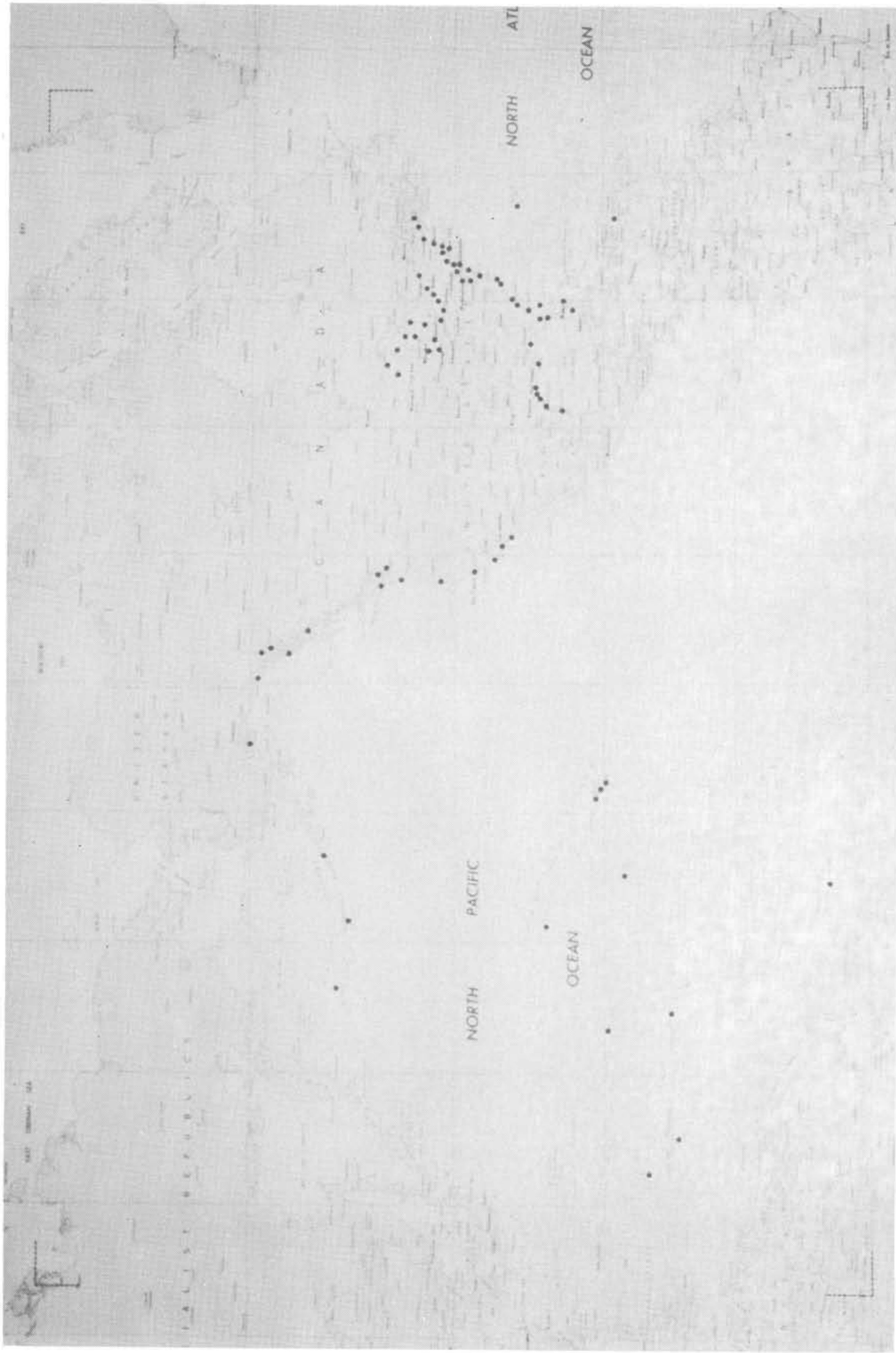


Figure 11

Secondary Control Tide Station

A secondary control tide measuring station is operated for 1 year, or longer, within each estuary and at coastal locations intermediate of primary control stations. These stations measure seasonal variations in water levels and establish tidal datums for the local area.

These stations also provide the required data for a 369 day analysis to derive all of the constituents required for tidal predictions. These data are reduced by comparison with simultaneous observations made at a primary control tide station to determine equivalent 19-year tidal datums. If the requirement exists, secondary control stations may remain in operation and become primary control stations when 19 years of measurements have been obtained.

Tertiary Tide Station

Tertiary tide observation stations are operated for 1- to 3- months to secure tide level data at local areas throughout an estuary. These stations provide the 29 day measurements which is the shortest series now utilized to derive the minimum harmonic constituents for predictions.

Data from these stations are reduced through simultaneous observations made at secondary and primary control tide stations to determine the equivalent of 19-year tidal datums.

Data Conversion Process

Tide records are tabulated in two forms, the hourly stage of the water elevation from which mean sea level is derived and the times and heights of high and low waters from which the other tidal datums are computed.

Records on digital format are transferred to magnetic tape and processed through computer techniques. Analog-type records are basically processed by hand tabulation methods.

Classification of Tides

The type of tide refers to the characteristic rise and fall of the water as revealed by the shape of the tide curve (Figure 4). The tide curves at any two locations will reveal differences in some respect, however, the types of curves are generally classified into three major groups:

1. Semi-daily or semidiurnal in which two high waters and two low waters approximately equal in height occur in one tidal day.
2. The daily or diurnal type of tide in which one high water and one low water occur in one tidal day.
3. The mixed type of tide in which two high waters and two low waters occur in one tidal day, but with marked differences between the heights of the two high waters or the two low waters.

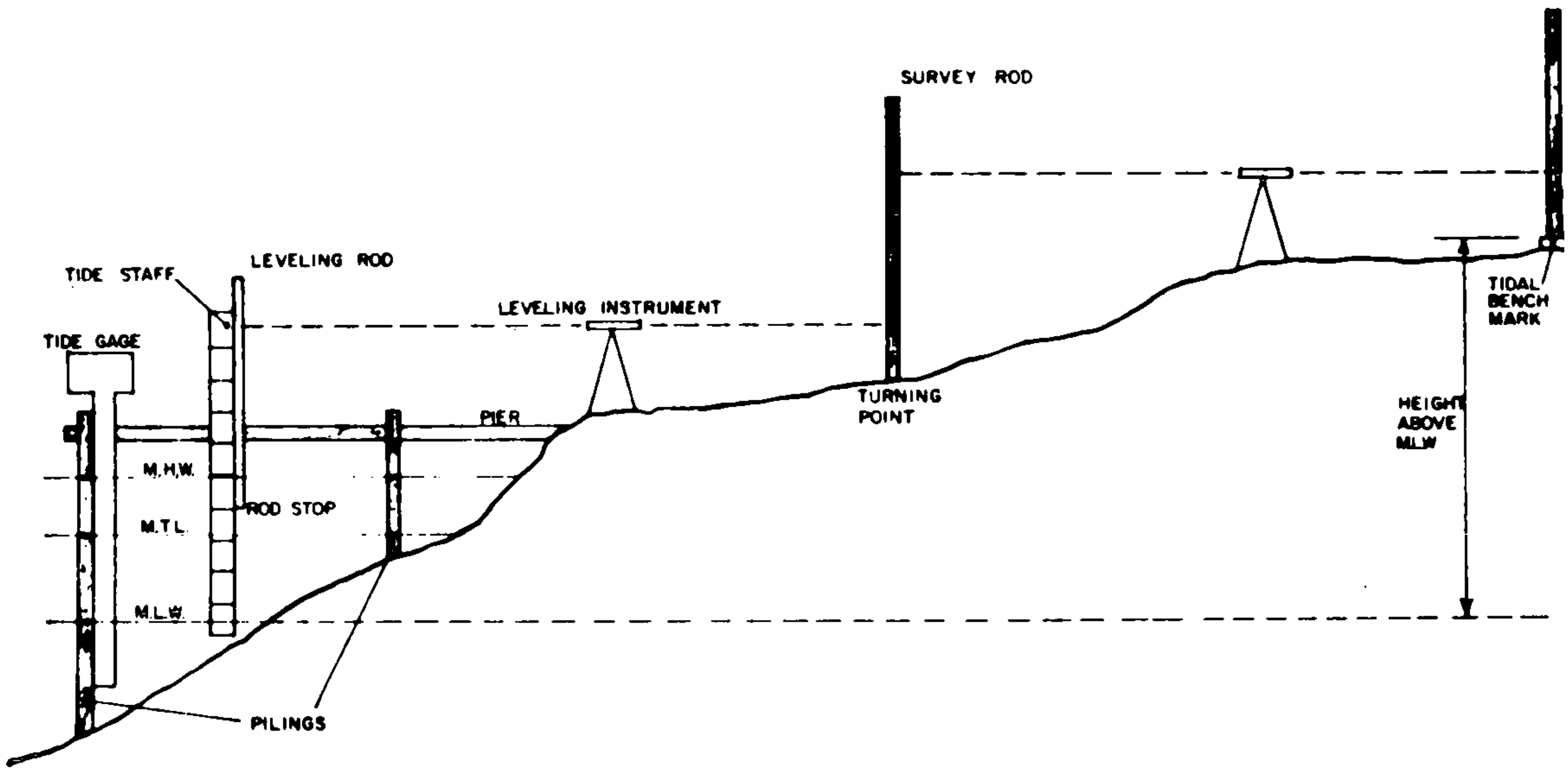

SURVEY ROD
TIDE STAFF
LEVELING ROD
LEVELING INSTRUMENT
TIDE GAGE
PIER
TURNING POINT
TIDAL BENCH MARK
HEIGHT ABOVE MLW
M.H.W.
M.T.L.
M.L.W.
ROD STOP
PILINGS

Figure 12

The basic component in a tide station is the staff or graduated scale which is established in the water at the site selected for the measurements. This staff is generally set with the zero at an arbitrary elevation, but low enough that no low waters will fall below it. It is essential that this staff be mounted in a plumb position. Elevations of the staff zero can then be transferred through differential levels to fixed points known as bench marks which are established on shore (Fig. 12).

The tide staff was the earliest type of tide gages utilized in making tidal measurements, the direct elevation of the water surface and time of measurement being visually observed and recorded.

The tedious and time-consuming job of reading all heights individually was replaced in the mid-1800's with automatic recording tide gages (Fig. 13). These were designed with a stilling well with intakes for the water which would filter outwaves but permit the water to rise and fall within the well at the same rate as outside the well. A float inside the well rose and fell with the tide and actuated a drum which caused the recording mechanism of the gage to draw a continuous analog trace of the rise and fall of the water. This, however, still contained one missing link--the transfer of the heights on the recorded graph to the ground where it was needed for surveying purposes.

To overcome this problem, the use of the tide staff was continued as the means to get the datums through levels, to the bench marks. In order to get the heights of the water on the staff related to the tide curve, daily observations of the height of the water on the staff as well as the time of the recording was annotated manually on the tide record.

This procedure not only provided the link between the curve on the tide record and the ground, but also verified the time recorded by the gage. Additionally, and of significant importance, it verified the proper functions of the tide gage. Malfunction of instrumentation or clogging of the intake to the well can be discerned by comparing the difference in staff/gage readings.

Bench Marks

Bench marks are established in the vicinity of all tide stations for reference points for elevations on the tide staff. At least ten such monuments are installed around the primary control stations. A minimum of five bench marks are required at secondary and tertiary control tide stations to assure preservation of the established datums.

The elevation of each bench mark in relation to the tide staff is determined by differential leveling. Complete information on the location and elevation of each bench mark relative to the tidal datums are described on tidal bench mark sheets (Figs. 14 and 15).

A significant aspect of the use of tidal datums is the accuracy and consistency of recovery. Even if bench marks are destroyed, it is possible to recover the same datum from a short series of tidal observations with remarkable accuracy.

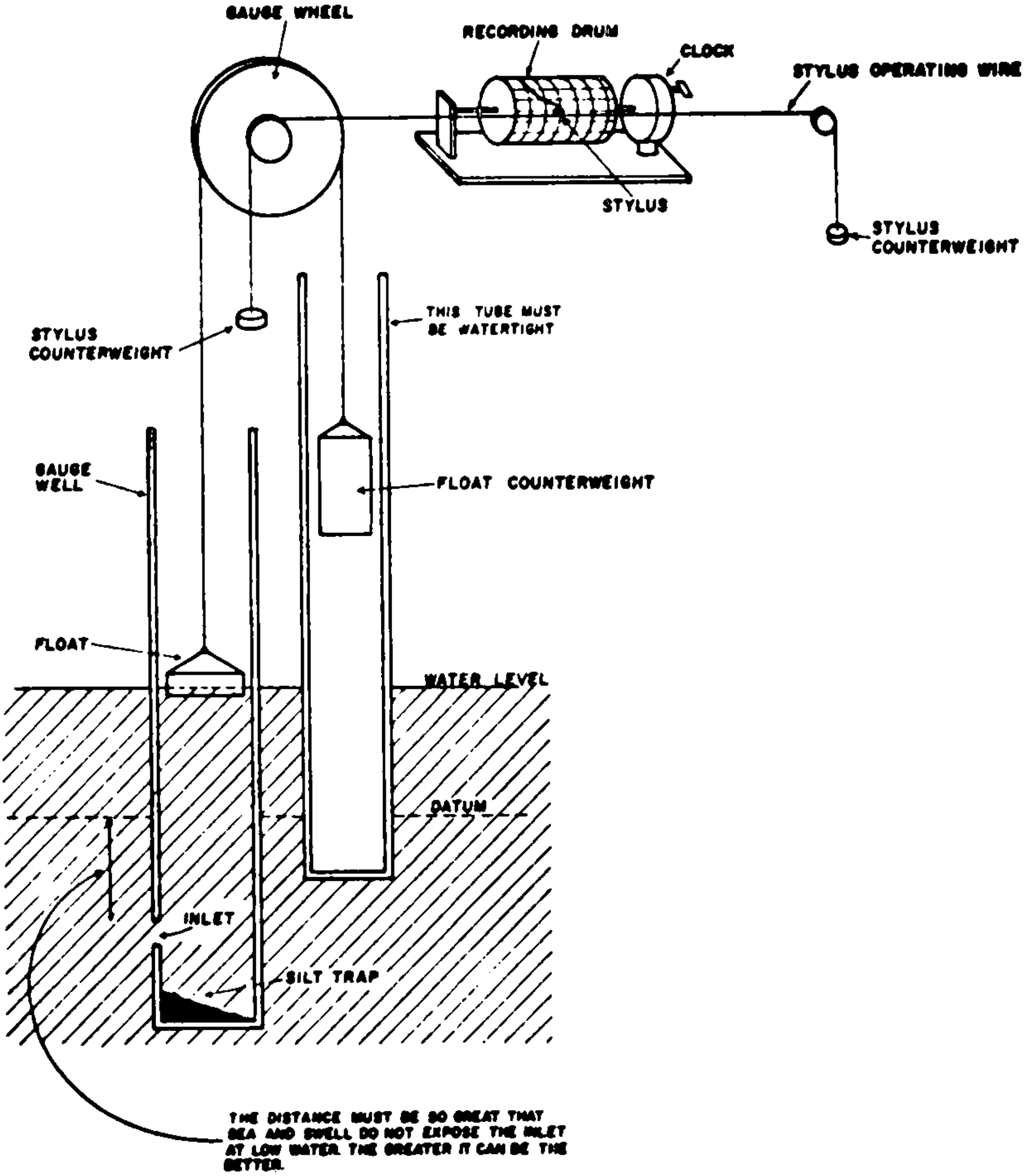


Figure 13.--Float-actuated automatic tide gage.

CALIFORNIA - III - 1 - 2

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NATIONAL OCEAN SURVEY

TIDAL BENCH MARKS

The Presidio, San Francisco
Lat. 37°48.4'; Long. 122°27.9'

BENCH MARK 173 (1925) is a standard disk, stamped "BM 173 1925," set in concrete post in southeast corner of yard of Fort Point Coast Guard Station, 125 feet northeast of northeast corner of an iron sewer grating, about 61 feet southeast of southeast corner of Crew's Quarters Building, 0.5 foot west of west edge of concrete curb, and 2 inches below surface of ground. Elevation: 10.18 feet above lower low water datum.

BENCH MARK 174 (1925) is a standard disk, stamped "BM 174 1925," set in concrete post flush with ground, about 125 feet west or prolongation of west edge of Engineers' Dock where it crosses Marine Drive, at center of "Y" between Marine Drive and road leading southeast to Fort Winfield Scott, about 42.5 feet southwest of fire hydrant, and 28.5 feet south of south edge of an iron manhole cover. Elevation: 16.90 feet above lower low water datum.

BENCH MARK 175 (1925) is a standard disk, stamped "BM 175 1925," set in top surface of seawall, about 400 feet west of prolongation of west edge of Engineers' Dock where it crosses Marine Drive, about 38 feet west of northwest corner post of woven wire steel fence around Engineers Supply Yard, at seawall line, and on north side of Marine Drive between Engineers' Dock and Fort Point and about 2.5 feet south of north edge of wall. Elevation: 14.63 feet above lower low water datum.

BENCH MARK 176 (1925) is a standard disk, stamped "BM 176 1925," set in west end of lowest concrete step at main entrance to porch of California Military District Headquarters Building at NO. 651 Mason Avenue, at south side of Crissey Field, 98 feet southeast of intersection of Crissey Field and Mason Avenues, 50 feet south of centerline of Mason Avenue and about 0.7 foot above the sidewalk around building. Elevation: 16.19 feet above lower low water datum.

Figure 14

Lower low water datum at The Presidio, San Francisco, is based on miscellaneous observations prior to 1907, and adopted as standard in March 1907. Elevations of other tide planes referred to this datum are based on 19 years of records, 1941 - 1959, and are as follows:

	Feet
Highest tide (December 24, 1940)	8.2
Mean higher high water	5.90
Mean high water	5.30
Mean tide level	3.30
Mean low water	1.30
Mean lower low water	0.20
Lower low water datum	0.00
Lowest tide (December 26, 1932 and December 17, 1933)	-2.5

Figure 15

Physically locating the tidal datum intersection points can be accomplished through standard ground survey procedures. Stake points and monuments can be physically located along the points of intersection, using tidal bench marks and their elevations. Once the datum line has been demarcated on the ground, it must be referenced to appropriate control points using accepted survey procedures.

Geodetic Datum Relationship to Tidal Datums

The NOS, early in its history, recognized the need for an engineering datum throughout the United States to which local elevations could be referenced. Since mean sea level was recognized as the basic tidal datum, it was felt that this would be the logical point to use as the zero for the starting elevation of the geodetic level network.

As the leveling progressed from east to west and north to south, it became desirous to connect points along the Atlantic, Gulf, and Pacific coasts. It was decided that 26 tide stations, 21 in the U.S. and 5 in Canada would be used as zero points for the basic net (Fig. 16). In 1929, a general adjustment of the geodetic levels throughout the network was used to make the geodetic datum conform to mean sea level around the entire country. The resulting datum for the geodetic net was then designated "Sea-Level Datum of 1929."

As more tide stations were placed in operation, and the geodetic network was densified, more differences in the relative elevations of the geodetic datum and the local mea sea level were revealed. Yet, in areas along the open coast these differences were not considered to be significant. However, as the requirements for tidal boundaries in estuaries increased, it became evident that these differences could indeed be significant; therefore, in 1972, the National Ocean Survey decided that, to avoid the misconception that mean sea level and sea level datum were everywhere synonymous, the name of the geodetic datum should be changed to the "National Geodetic Vertical Datum of 1929 (NGVD)," omitting any reference to sea level.

The NGVD is still recognized as the basic engineering and surveying datum throughout the U.S. and is still maintained by the National Ocean Survey's, National Geodetic Survey. However, in coastal areas where tidal datums are to be used for ownership and jurisdictional boundaries, the use of elevations of bench marks within the National Geodetic Network to locate marine boundaries are of themselves, not acceptable. To obtain the elevations of the geodetic datums relative to tidal datums (MHW, MLW, etc.), engineers and surveyors are requested to contact the National Ocean Survey, Rockville, Maryland, Attention C331, where these differences, if available, are maintained (see Fig. 17).

Changes in Sea Level

Long-term tidal records reveal that changes in mean sea level and other tidal datums, relative to the land, are occuring. To a small degree these changes are attributable to the eustatic rise in sea level related to long-term climate changes. However, in other areas where it is more pronounced, much of this change is considered to be caused by tectonic movement. In many areas subsidence of the land relative to the sea is shown

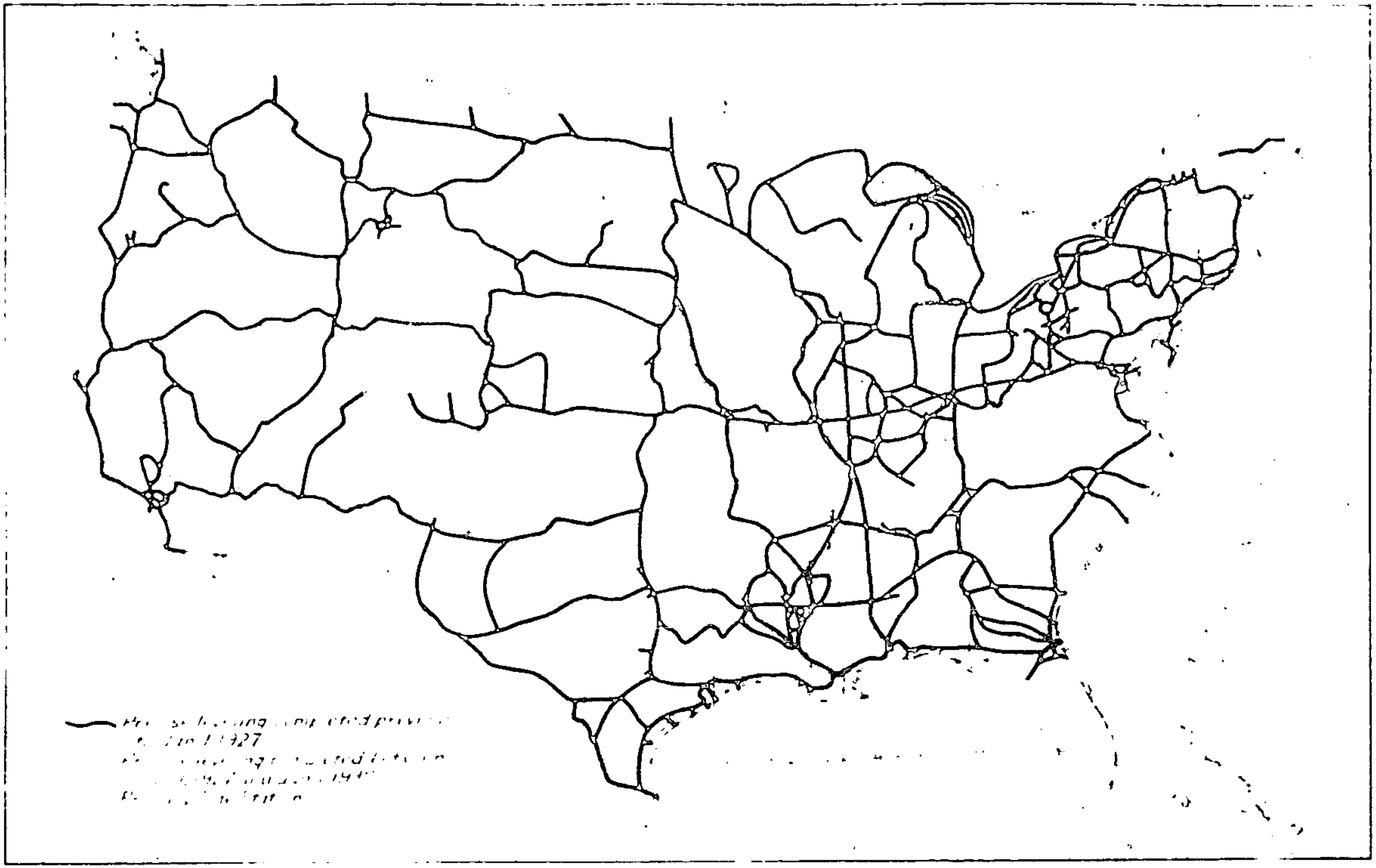


Figure 16. First-order leveling. U.S. Coast and Geodetic Survey.

CALIFORNIA - PART III

U.S. DEPARTMENT OF COMMERCE
NATIONAL OCEANIC AND ATMOSPHERIC ADMINISTRATION
NATIONAL OCEAN SURVEY

San Francisco Bay and San Joaquin
Sacramento Delta Region

The difference between National Geodetic Vertical Datum (formerly sea-level datum of 1929) (SLD) and the mean lower low water (MLLW) for each location where the tidal bench marks and the geodetic bench marks of the national geodetic Network have been connected by differential levels is given below.

Bench mark elevations above National Geodetic Vertical Datum may be obtained by applying the tabular difference to the published elevations above mean lower low water; subtracting the difference when positive and adding the difference when negative.

Index Map Number	Locality	NGVD-MLLW Feet
1	The Presidio, San Francisco	3.06**
	North Point (Pier 41)	3.01*
1A	Ferry Building	3.14*
	San Francisco (Pier 22 1/2)	2.99*
2	Rincon Point, San Francisco	2.92*
3	Potrero Pt. (Bethlehem Shipbuilding Corp. Plant), San Francisco	2.94
4	Hunters Pt., San Francisco Bay	3.08
5	South San Francisco, San Francisco Bay	3.65*
6	San Mateo (Coyote Pt. Marina), San Francisco Bay	3.45*
7	San Mateo Bridge, San Francisco Bay	3.82
8	Redwood Creek, San Francisco Bay	3.83*
9	Palo Alto Yacht Harbor, Mayfield Slough, San Francisco Bay	4.04*
12	Dumbarton Highway Bridge, San Francisco Bay	4.14*
13	Oakland Municipal Airport, Oakland, San Francisco Bay	2.99
15	Oakland (Park Street Bridge), San Francisco Bay	2.92

Figure 17

and in other areas uplift of the land is revealed. The National Ocean Survey publishes secular changes in sea level as measured at the primary control tide stations (Figs. 18 and 19).

National Tidal Datum Epoch

The general rise in sea level as well as regional changes in land elevations measured at the control tide stations are rather pronounced in many instances. In order to maintain a uniform datum for mapping the shoreline of the United States, it has long been evident that a common period of time must be used as a reference. Therefore, a certain period or epoch is designated as the "National Tidal DAtum Epoch." The present epoch in use is the 1941-1959 period to which all datums are referred. The National Ocean Survey compiles the changes which are occuring annually and must evaluate the need for changing the national epoch at 25 year intervals.

Predicted Tides

The time tables of the daily predictions of times and heights of the tides are published annually by the National Ocean Survey as a supplement to the nautical charts for aids to navigation (Fig. 20). These predictions are based on the astronomic tide and are computed from harmonic constants (Fig. 21). while generally quite reliable, they do not account for nontidal variations in water levels which are the result of short-term meteorological forces or river outflows. These predictions must be recognized for what their intended purpose is; i.e., to provide a guide for the occurence of times of high and low waters for navigation purposes. They are based on prediction of changes in range of tide and not elevations of water level relative to tidal datums for surveying and engineering purposes. Daily departures from the predicted heights, mainly due to nontidal effects, can be significant (Fig. 22).

Demands for tidal information in recent years by various users have far exceeded the capability of the National Ocean Survey to respond in a timely fashion. To overcome this problem, a mini-computer system, which was delivered in March 1978, is presently being installed. This dedicated system will permit on-line processing and permit interaction by the oceanographers who must interpret the digitized data.

In addition, a new telemetry system has been developed which will interrogate the primary tidal network stations from the Rockville office on a daily-basis via phone lines, radio or satellite transmission (Fig. 23). The initial units will be delivered in late 1978. This new equipment will provide the National Ocean Survey with the capability to respond rapidly to all user requests.

Summary

In summary, NOAA's National Ocean survey collects tidal data in the best possible scientific manner in support of its mission responsibilities; to provide aids to navigation in the form of charts, tide predictions, long period sea level trends, and tidal datums for use in delimiting marine boundaries. The National Ocean Survey has no regulatory or jurisdictional powers; and these data are provided on an impartial basis to any user upon request.

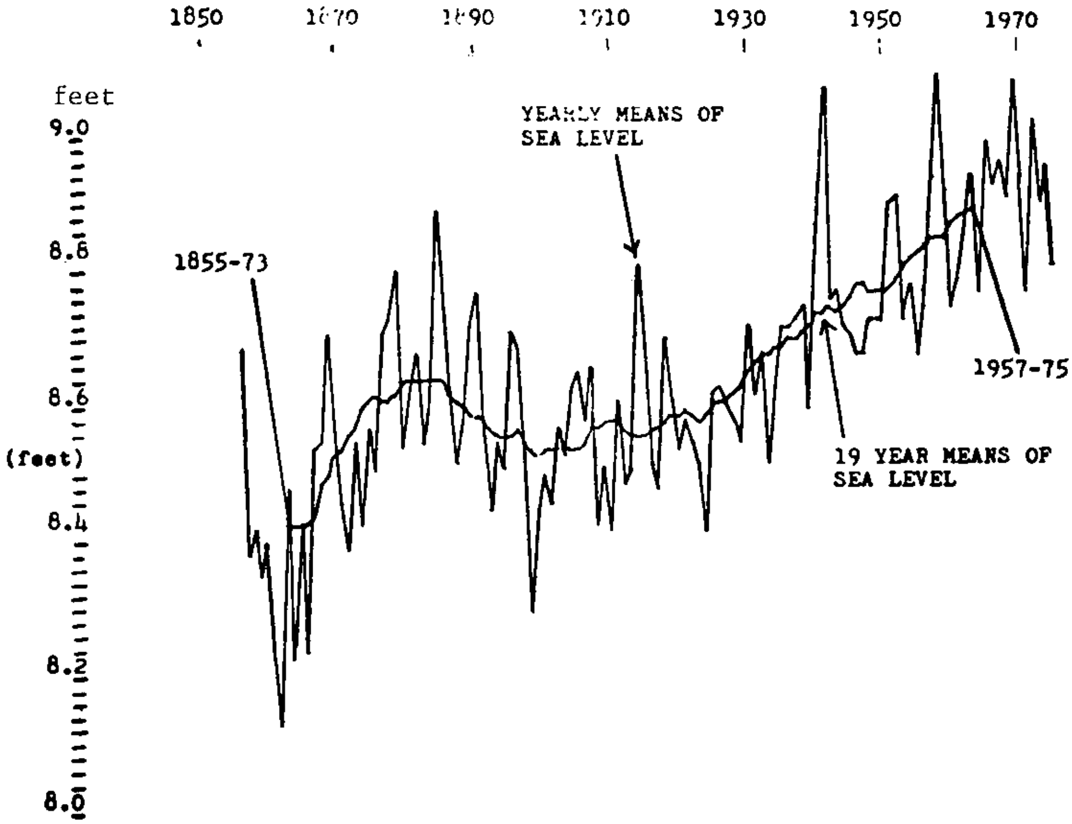


Figure 18.--Sea level curve, Golden Gate area, 1855-1975.

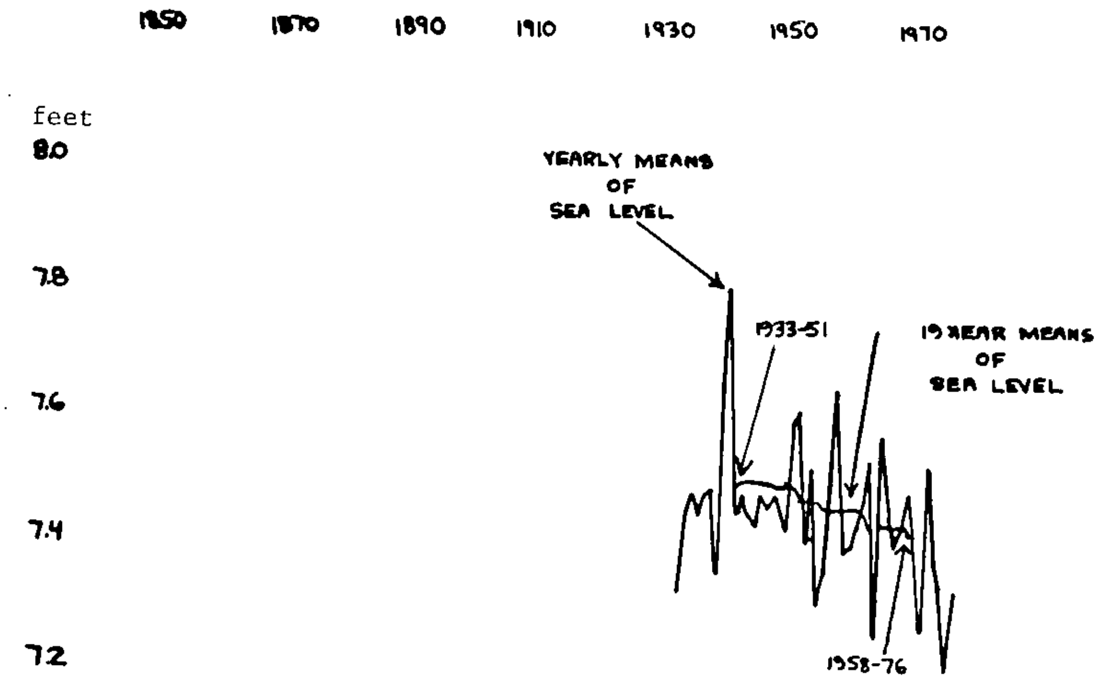


Figure 19.--Sea level curve, Crescent City, 1933-1976.

TIMES AND HEIGHTS OF HIGH AND LOW WATERS

APRIL

DAY	TIME h.m.	HT. ft.	DAY	TIME h.m.	HT. ft.
1 SA	0225 0909 1507 2132	4.6 0.0 4.1 0.2	16 SU	0229 0922 1516 2145	3.8 0.7 3.6 1.0
2 SU	0333 1009 1615 2233	4.6 -0.2 4.3 0.0	17 M	0330 1013 1615 2236	3.9 0.5 3.8 0.7
3 M	0441 1105 1716 2329	4.7 -0.4 4.6 -0.3	18 TU	0431 1100 1708 2325	4.0 0.3 4.1 0.4
4 TU	0542 1158 1811	4.8 -0.6 4.9	19 W	0523 1145 1753	4.2 0.1 4.5
5 W	0022 0635 1246 1900	-0.6 5.0 -0.7 5.2	20 TH	0011 0611 1227 1835	0.1 4.4 -0.1 4.8
6 TH	0113 0723 1332 1945	-0.7 5.0 -0.8 5.3	21 F	0057 0654 1309 1913	-0.2 4.6 -0.3 5.1
7 F	0200 0808 1417 2028	-0.8 5.0 -0.8 5.3	22 SA	0140 0736 1351 1953	-0.5 4.7 -0.4 5.4
8 SA	0246 0851 1459 2109	-0.8 4.8 -0.6 5.2	23 SU	0225 0818 1432 2034	-0.7 4.7 -0.5 5.5
9 SU	0328 0936 1537 2155	-0.7 4.6 -0.4 5.0	24 M	0307 0902 1513 2120	-0.8 4.7 -0.5 5.5
10 M	0408 1019 1616 2237	-0.4 4.3 -0.1 4.8	25 TU	0353 0955 1557 2211	-0.8 4.6 -0.4 5.4
11 TU	0448 1107 1651 2320	-0.2 4.1 0.2 4.5	26 W	0439 1053 1643 2311	-0.7 4.5 -0.2 5.3
12 W	0528 1154 1730	0.2 3.8 0.6	27 TH	0530 1153 1736	-0.4 4.4 0.1
13 TH	0005 0618 1241 1815	4.3 0.5 3.7 0.9	28 F	0010 0631 1254 1849	5.1 -0.2 4.4 0.3
14 F	0050 0718 1329 1929	4.1 0.7 3.6 1.1	29 SA	0111 0742 1353 2007	4.9 -0.1 4.4 0.4
15 SA	0137 0823 1420 2044	3.9 0.8 3.5 1.1	30 SU	0212 0849 1455 2117	4.7 -0.1 4.4 0.3

MAY

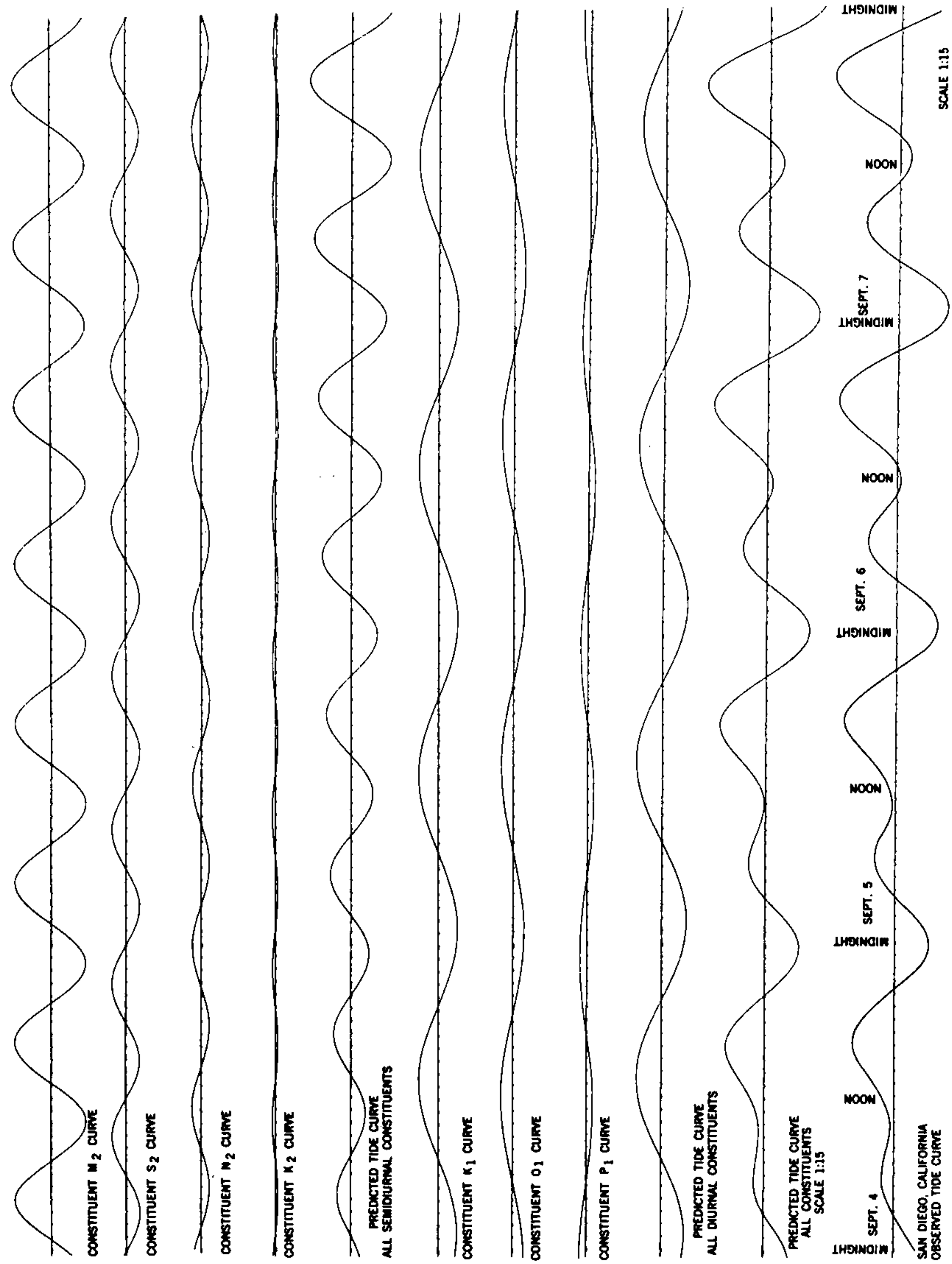
DAY	TIME h.m.	HT. ft.	DAY	TIME h.m.	HT. ft.
1 M	0316 0949 1558 2217	4.6 -0.2 4.6 0.1	16 TU	0231 0927 1523 2157	3.9 0.6 4.0 0.9
2 TU	0420 1042 1657 2313	4.6 -0.3 4.8 -0.1	17 W	0328 1015 1618 2247	4.0 0.4 4.3 0.6
3 W	0520 1132 1750	4.6 -0.4 5.1	18 TH	0432 1100 1709 2337	4.1 0.2 4.7 0.2
4 TH	0003 0612 1220 1838	-0.3 4.7 -0.4 5.3	19 F	0529 1145 1756	4.3 0.0 5.1
5 F	0054 0701 1307 1921	-0.4 4.7 -0.4 5.4	20 SA	0025 0619 1232 1841	-0.1 4.5 -0.2 5.4
6 SA	0140 0744 1349 2003	-0.5 4.6 -0.4 5.4	21 SU	0114 0707 1318 1925	-0.4 4.6 -0.4 5.7
7 SU	0225 0828 1432 2044	-0.5 4.5 -0.3 5.3	22 M	0202 0755 1406 2014	-0.7 4.7 -0.5 5.8
8 M	0307 0910 1512 2124	-0.4 4.4 -0.1 5.1	23 TU	0251 0846 1454 2103	-0.9 4.8 -0.5 5.8
9 TU	0346 0953 1548 2203	-0.3 4.2 0.2 4.9	24 W	0339 0942 1542 2158	-0.9 4.7 -0.4 5.7
10 W	0424 1040 1624 2245	-0.1 4.0 0.4 4.6	25 TH	0427 1043 1634 2259	-0.8 4.7 -0.2 5.5
11 TH	0503 1125 1659 2328	0.1 3.9 0.7 4.4	26 F	0519 1144 1730 2359	-0.7 4.7 0.0 5.3
12 F	0544 1213 1733	0.4 3.8 1.0	27 SA	0617 1242 1837	-0.4 4.7 0.3
13 SA	0010 0631 1257 1820	4.2 0.6 3.7 1.2	28 SU	0057 0719 1338 1950	5.1 -0.2 4.7 0.4
14 SU	0051 0729 1342 1950	4.1 0.7 3.7 1.3	29 M	0154 0824 1436 2058	4.8 -0.2 4.8 0.4
15 M	0137 0832 1430 2100	4.0 0.7 3.8 1.2	30 TU	0254 0923 1533 2159	4.6 -0.1 4.8 0.2
			31 W	0355 1017 1631 2252	4.4 -0.2 5.0 0.1

JUNE

DAY	TIME h.m.	HT. ft.	DAY	TIME h.m.	HT. ft.
1 TH	0454 1105 1726 2342	4.4 -0.2 5.1 0.0	16 F	0334 1016 1625 2304	4.0 0.3 4.9 0.3
2 F	0547 1153 1814	4.4 -0.1 5.3	17 SA	0445 1107 1721 2358	4.2 0.1 5.2 0.0
3 SA	0032 0638 1240 1859	-0.1 4.4 -0.1 5.3	18 SU	0547 1158 1814	4.4 -0.1 5.6
4 SU	0118 0723 1323 1939	-0.2 4.4 0.0 5.3	19 M	0049 0644 1251 1905	-0.4 4.6 -0.3 5.9
5 M	0203 0805 1405 2019	-0.2 4.3 0.0 5.2	20 TU	0141 0736 1346 1955	-0.7 4.8 -0.5 6.0
6 TU	0245 0847 1447 2058	-0.2 4.2 0.2 5.1	21 W	0233 0830 1438 2050	-0.9 4.9 -0.5 6.0
7 W	0324 0929 1526 2136	-0.2 4.1 0.3 4.9	22 TH	0322 0927 1531 2144	-1.0 4.9 -0.5 5.9
8 TH	0403 1014 1601 2214	-0.1 4.0 0.5 4.7	23 F	0411 1027 1622 2243	-1.0 5.0 -0.4 5.7
9 F	0437 1059 1634 2253	0.1 3.9 0.7 4.6	24 SA	0501 1125 1717 2341	-0.8 5.0 -0.1 5.4
10 SA	0514 1141 1704 2330	0.3 3.9 0.9 4.4	25 SU	0554 1221 1818	-0.6 5.0 0.2
11 SU	0549 1221 1738	0.4 3.9 1.1	26 M	0037 0652 1316 1926	5.1 -0.3 5.0 0.4
12 M	0007 0629 1303 1831	4.3 0.6 4.0 1.2	27 TU	0132 0754 1412 2033	4.8 -0.1 5.0 0.5
13 TU	0047 0722 1345 2006	4.2 0.6 4.1 1.2	28 W	0228 0854 1507 2135	4.5 0.0 4.9 0.4
14 W	0135 0827 1430 2114	4.1 0.6 4.3 1.0	29 TH	0326 0948 1603 2229	4.3 0.1 4.9 0.4
15 TH	0230 0925 1525 2212	4.0 0.5 4.5 0.7	30 F	0425 1038 1658 2321	4.1 0.1 5.0 0.3

TIME MERIDIAN 75° W. 0000 IS MIDNIGHT. 1200 IS NOON.
HEIGHTS ARE RECKONED FROM THE DATUM OF SOUNDINGS ON CHARTS OF THE LOCALITY WHICH IS MEAN LOW WATER.

Figure 20



CONSTITUENT M_2 CURVE
CONSTITUENT S_2 CURVE
CONSTITUENT N_2 CURVE
CONSTITUENT K_2 CURVE
PREDICTED TIDE CURVE
ALL SEMIDIURNAL CONSTITUENTS
CONSTITUENT K_1 CURVE
CONSTITUENT O_1 CURVE
CONSTITUENT P_1 CURVE
PREDICTED TIDE CURVE
ALL DIURNAL CONSTITUENTS
PREDICTED TIDE CURVE
ALL CONSTITUENTS
SCALE 1:15
MIDNIGHT
SEPT. 4
NOON
MIDNIGHT
SEPT. 5
NOON
MIDNIGHT
SEPT. 6
NOON
MIDNIGHT
SEPT. 7
NOON
MIDNIGHT
SAN DIEGO, CALIFORNIA
OBSERVED TIDE CURVE
SCALE 1:15

Figure 21

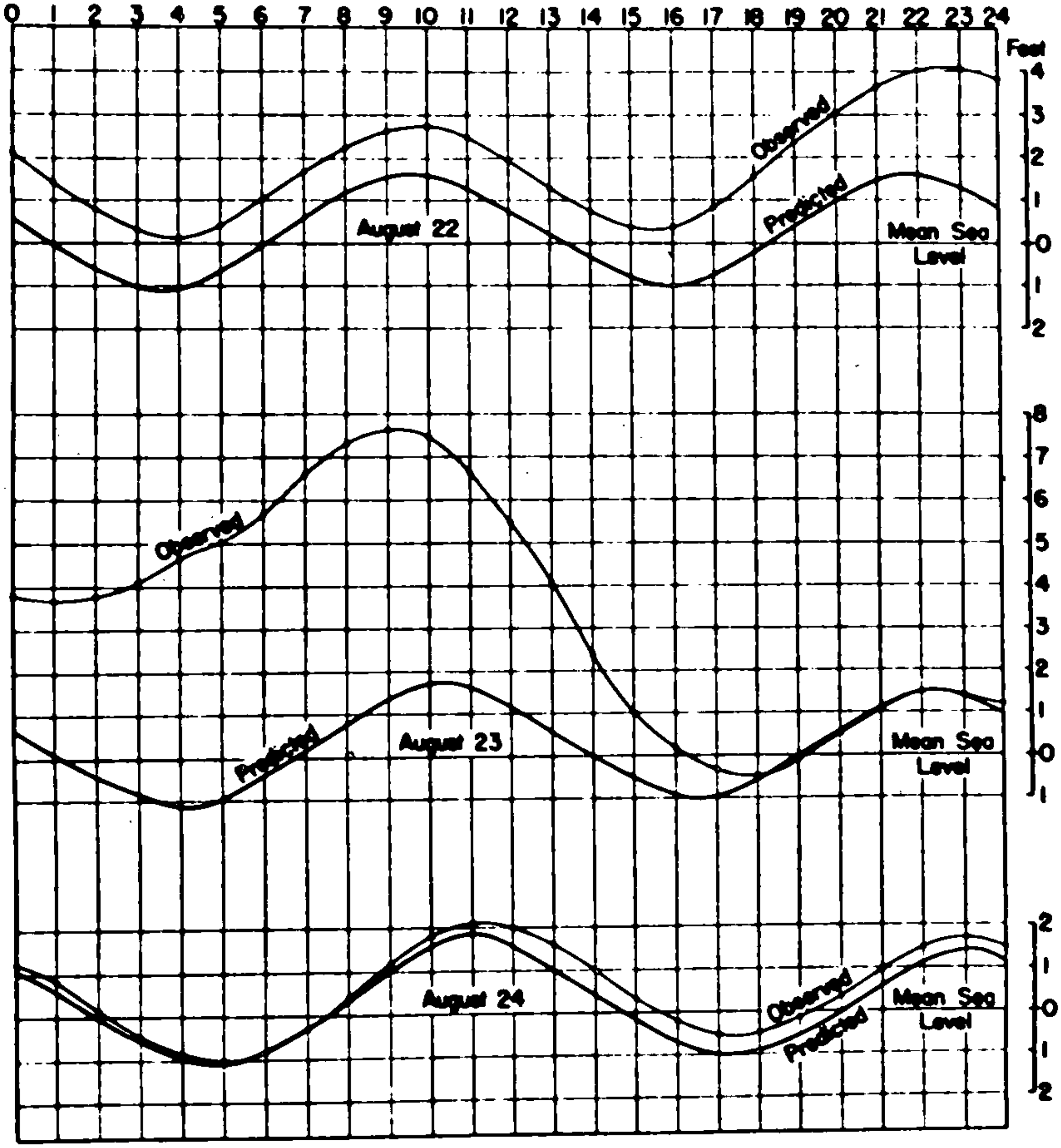


Figure 22.--Tide at Hampton Roads--hurricane of August 1933.

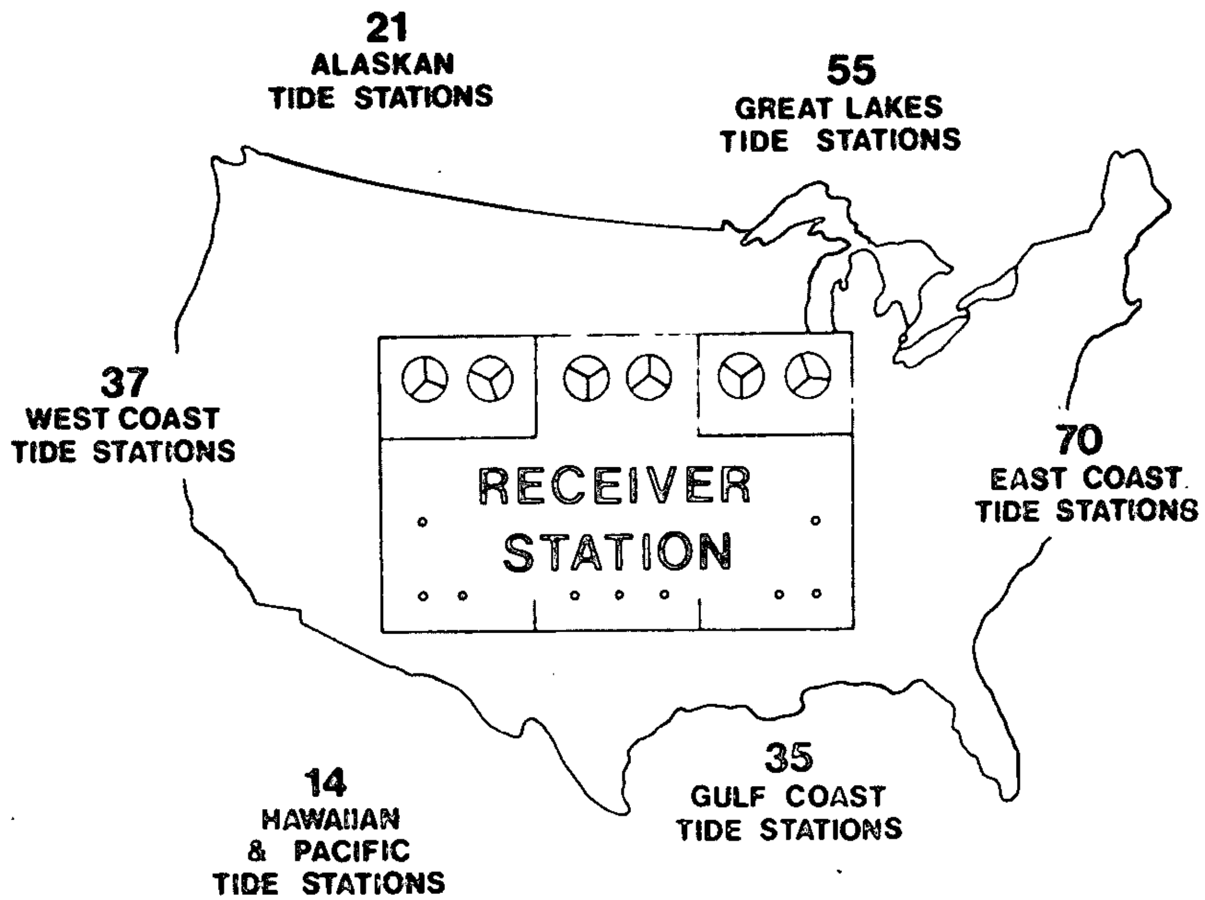


Figure 23. Tide and water telemetry.

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List of Abbreviations

MHW	Mean high water
MHHW	Mean higher high water
MLW	Mean low water
MLLW	Mean lower low water
MSL	Mean sea level
MTL	Mean tide level
NGVD	National Geodetic Vertical Datum of 1929

GEODETIC DEFORMATION MEASUREMENTS ON LARGER DAMS

by

STANLEY K. NAZALEWICZ, B.S., M.S. IN GEODESY

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INTRODUCTION

In most cases, the moment when construction is completed, is considered the end of project for the staff of design engineers. However, there are structures that due to their destination or location, are exposed to displacement or deformation, or both, and require continuous evaluation to insure their stability, structural safety and operational adequacy.

Dams are the structures which require upon completion continuing, thorough, highly professional examinations, observations and studies.

The importance of dam safety confirms number of dam catastrophes. It is enough to refer to the catastrophe of South Fork Dam taking 2500 lives, Welnot Grave, Bouzey, Gleno, St. Denis du Sig, Chazilly, San Francis and many others from which some have occurred recently; Piave in Italy, Frejus and Malpasset in France. The failure of a dam can result in a sudden release down-stream millions cubic meters of water, with catastrophic loss of lives and property damages.

Structural safety can be determined by systematic studies and evaluation of dam behavior during its construction and exploitation. The analysis of the dam is based on the design load on the dam. The water load is well known, but uncertain factors such as reaction of abutments and foundations remains. Therefore the calculated stress-strain conditions must be checked by accurate deformation measurements. These geodetic measurements are necessary to determine whether the dam behaves according to expectation. Determination of the deformation larger than permissable signalizes a state of emergency for the dam, and possible down-stream damage.

DAM SAFETY EXAMINATION

Dam safety examination is to be considered in the aspect of studies based on the recording of physical data as well as relative and absolute displacement measurments of the dam.

Deformation values on larger dams are influenced by many factors, such as weight of structure, water pressure, foundation, concrete due to temperature changes and water buoyancy. Periodical geodetic measurements should be performed under the same conditions, or, if not, some important physical data must be recorded and applied for proper evaluation and interpretation of deformation values.

The physical data which must be recorded are: measurements of stress-strain in the abutments and foundation, observation of filtration and leakage, examination of chemical and biological water composition, echosounding or sounding for determination of reservoir slime, measurement of the concrete temperature at various points and reservoir water temperature, measurment of water level.

By knowing concrete temperature changes at various points of the structure, we can be alerted to the problems of thermal expansion and concrete growth, causing a thrust that may crack the concrete.

Annual temperature changes cause horizontal displacement of a dam crest equal to 5-6 mm. It is illustrated below how seasonal temperature changes effect displacement of the dam. During spring and summer, horizontal displacement is in the reservoir direction, during fall and winter in the down-stream direction.

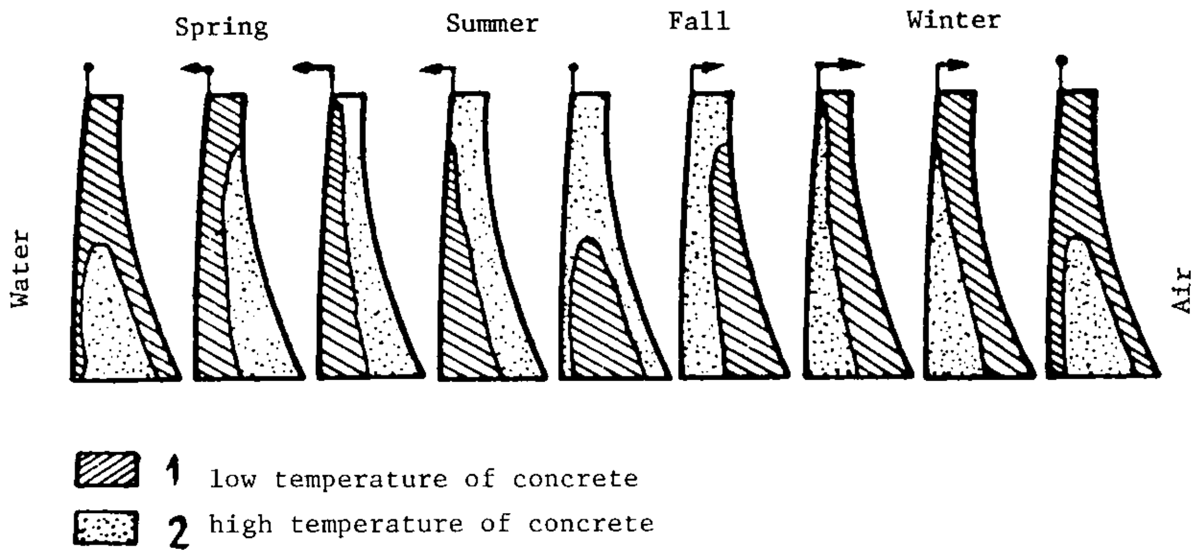


Fig. 1

For the early discovery of precarious changes <u>the relative measurements</u> are obtained using instruments installed directly in the structure. The instruments used are:

- pendulums,
- rockmeters,
- deformeters,
- clinometers

These measurements allow one to determine, the values of the displacement of one element of the structure. Relative displacement can also be controlled by geodetic traverses where distances are measured by means of invar wires. The relative measurements should be integrated into the geodetic scheme.

<u>The absolute displacement measurements</u> include geodetic deformation methods which refer locations of the examined points of the structure to the motionless stations located out of the pressure zone from which the measurements are made. To determine values of the absolute displacement the following geodetic methods are used:

- trigonometric method (triangulation, trilateration, triangulateriation, trigonometric leveling),

- precise leveling,

- alignment observations.

The geodetic methods furnish the engineers building the dam with a very important set of data:

- the deformations of the foundation under the load of the growing structure,

- deformations due to the first impounding at a time when most other measuring devices cannot be set up as yet,

- magnitude and direction of the deformations of the structure due to variations in hydrostatic pressure.

The selection of methods and instruments and the arrangement and number of observations highly depend on the precision and accuracy deemed necessary as well as on topographical and other circumstances. Geodetic measurements therefore have to be designed individually.

Experienced personnel is required: geodetic engineer, reliable observer and assistants.

Periodic observations should be performed by the same observer always using the same instrument.

For deformation measurements triangulation or trilateration can be used, however in most cases a combination of directions and distances will be of advantage.

GEODETIC SCHEME

Geodetic deformation measurements provide a direct measure of displacement as a function of time.

Deformation measurements on dams must extend over many decades and geodetic scheme for the geodetic measurements must be designed and established to guarantee the continuity.

All measurements have to be planned in such a way to yield a sufficiently precise and accurate deterination of coordinates. The precision of the results depends on the precision of the instruments, methods used and on the disposition of the measurements.

TRIANGULATION

Triangulation is the traditional method of geodesy for determination of coordinates of points. This method enables the determination of absolute displacement of the structure points.

In principle the method consists of measuring the directions to series of targets set rigidly into concrete of the dam from a number of suitably located instrument pillars. From the changes in these directions, measured periodically over a given time, the deformations can be calculated.

The triangulation method was used, for the first time, in 1921 by Zolly and Lang, two Swiss engineers on the Montsalvens arch power dam.

The triangulation scheme should have one or two base lines measured with accuracy of 1/400 000 and should be extended a minimum 500 m down-stream from the dam.

Fixpoints of triangulation are concrete pillars built on solid rock with the flat square tip, 0.5 m x 0.5 m or circle 0.4 m in dia., 1.2 m above the ground, located 30-150 m from the dam depending on topographical configuration and the structure site. It is imperative that for repeated theodolite set-ups each pillar has a special bolt embedded in the center of the top. The bolt must be truly vertical to insure perfect centering of the instrument or target.

The location for fixpoints is chosen so that

the stress caused by the weight of the dam and the water load of the reservoir may be neglected;

the number and layout of the fixpoints will guarantee desired precision.

Fixpoints are presumably unmoving or the displacement of which can be neglected. As it has to be assumed that some fixpoints do not come up to this expectation, the following rules have to be observed:

Triangulation scheme must consist of at least 4 fixpoints;

Control of fixpoints is by reference points;

The relative mutual position of the fixpoints can be determined with sufficient precision by triangulation. (See Fig. 2)

<u>The points on the dam</u> at which locations are to be determined by intersection are arranged in horizontal and vertical rows. It is very important that each vertical row has three points needed to draw a deformation line. The interval between these points depends on the number and arrangements of the structural blocks. Letters are assigned for each horizontal row, vertical rows are numbered from left to right (looking upstream). Points on the dam should be intersected from 3 or, if impossible, from 2 stations. The angle of intersection should be between 60-120^{o}.

<u>Orientation points</u> are located outside of the pressure area and far enough from observation station so that an error of orientation caused by possible slight displacement of the stations may be neglected in view of an error of observation.

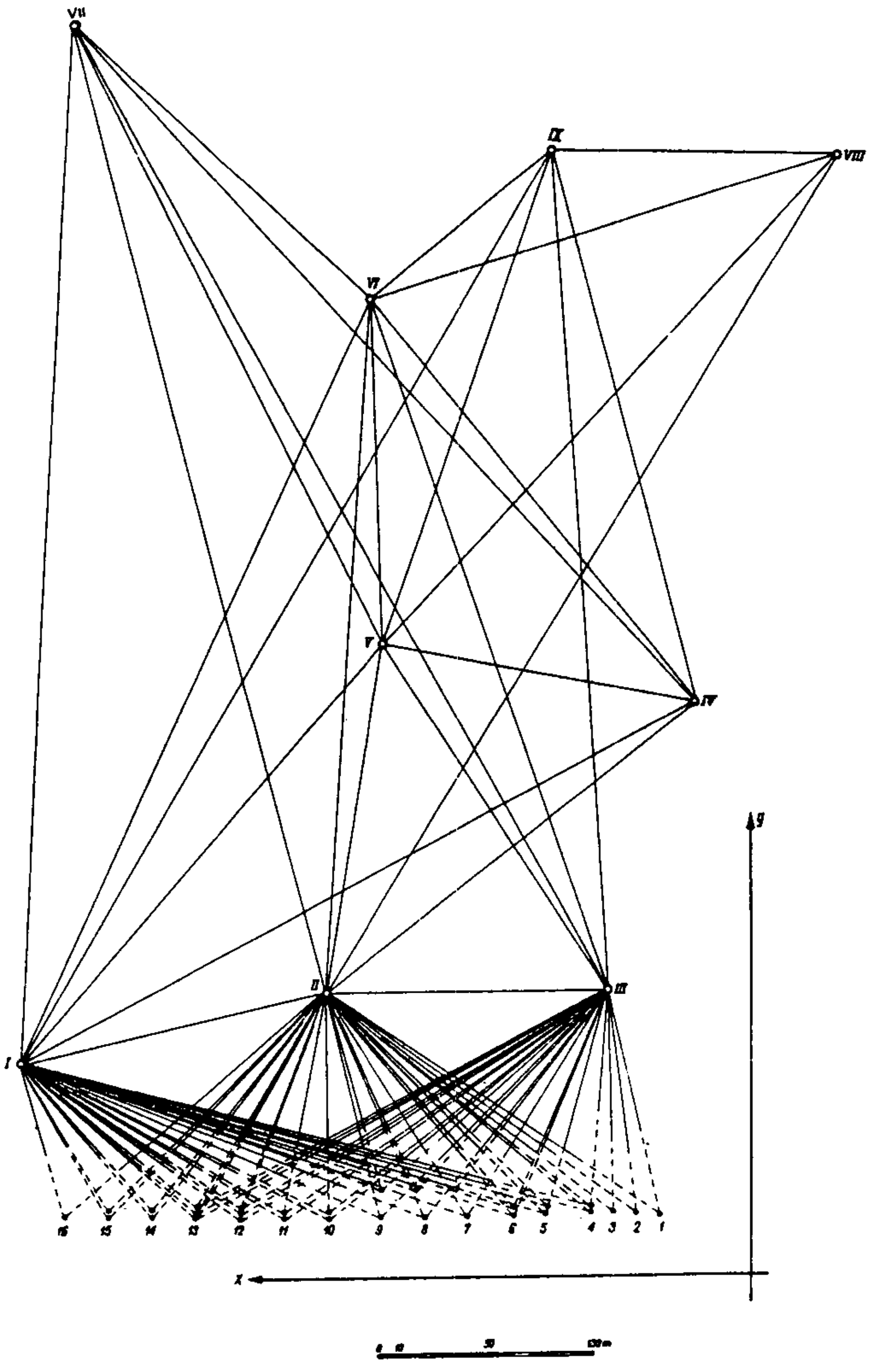


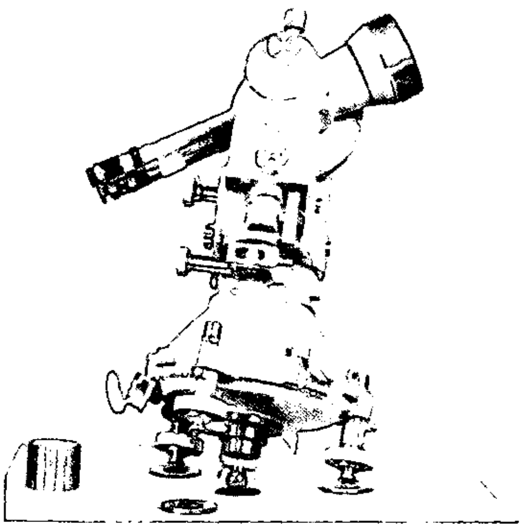
Fig. 2 Triangulation scheme

Three to four orientation points are assigned to each theodolite station in order to guard against any possibility of errors in the orientation of the sets of directions. Their very purpose makes it clear that these points must be of very well determined and fixed position, preferably mounted on rock. The size of the marks of targets to be used for them will be determined by the sighting distance.

Reference points - as some of the fixpoints (observations pillars) could suffer very slight changes in their positions, they must be controlled by reference points, situated on bedrock and located out of the pressure zone.

OBSERVATION OF DIRECTIONS AND VERTICAL ANGLES

Directions are observed in 4 sets with a high precision theodolite WILD T-3 (Fig. 3) or KERN DKM-3. The highest manification of 40 or 45 is commonly chosen for observations. Observations are made from concrete pillars.



Wild T3 Precision Theodolite with ball-centring for accurate setting-up on pillars for deformation measurement.

Fig. 3

The three theodolite foot screws are set on metal disks, while the theodolite centering ball pivot is seated into the pillar bolt (See Fig. 4). The centering ball fits precisely into a socket cemented into the top of the pillar, so that theodolite is always exactly in the same position. In order to maintain the height of the instrument constant, the centering ball pivot is provided with two small slides, mounted at the extremities of a diameter, each of which carries an index. By operating the footscrews as needed, the index on the aligning stud can be brought to coincidence with the indices on the slides, which rest on the edges of the pillar bolt. Since sightings at a considerable vertical angle are inevitable in the course of work, attention should be paid to perfect leveling of the theodolite for that the collimation level ay be used and the leveling of the theodolite should be checked if necessary, improved after every half series.

Single direction measurement readings in each half of series consists of double sighting by using of azimuth fine motion screw, double coincidence and micrometer reading.

For determination of elevation of some not accessible points on the dam, the method of trigonometric leveling is used. Vertical angles are measured by the same theodolite used for directions.

When taking observations on a power dam, a well trained observer can obtain an average accuracy, for a direction on twice-measured sets, of about $\pm$ 0.5" and $\pm$ 0.9" for a vertical angle, or about $\pm$ 0.3 mm and 0.5 mm over a 100 m aiming distance.

Fig. 4 Deformation equipment. From left: three-foot screw discs, centering socket, cover for centering socket, permanent target bolt for embedding in dam wall, insert target.

TRILATERATION

Deformation measurements on large dams can be made in two dimensions using trilateration method.

A network of fixpoints must be established so that points on the structure can be related to those points which have been selected for stability, and at some distance from the structure itself. Concrete pillars should be placed in geologically stable positions, their lay-out should guarantee geometry for trilateration measurements. For the best results the angle of intersection should be 90°, but in any case it should be between 30° - 150°. In trilateration, lengths to the surveyed point from each of two fixpoints will give the position of the surveyed point in two dimensions. Measurements from three fixpoints will give three positions of the surveyed point, which may be used as a check of survey accuracy. Figure 5 shows a perfect net for the measurements of a dam.

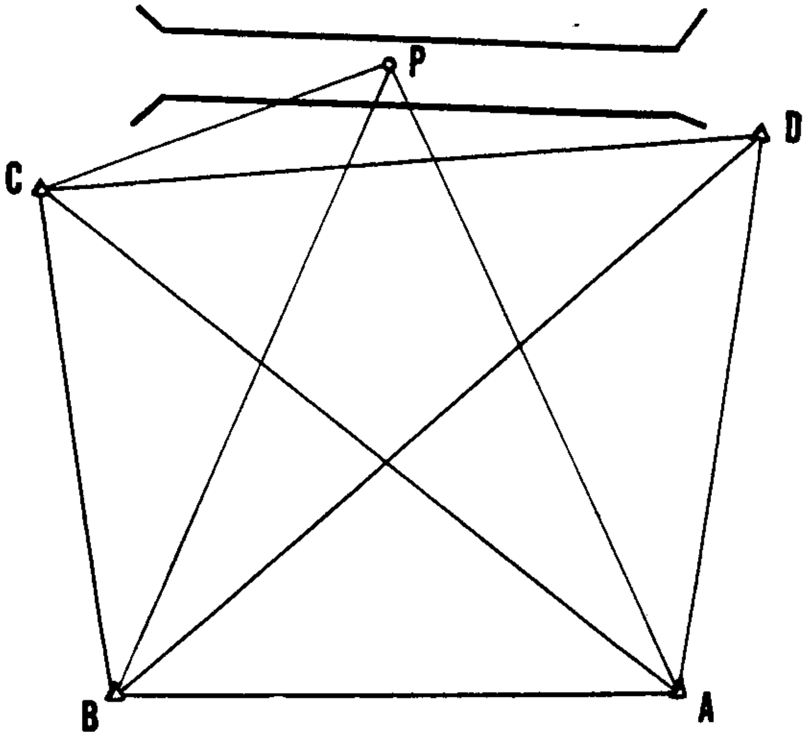


Fig. 5

Points A,B,C and D are concrete pillars. All are intervisible. Point P is the surveyed point on the dam and is measured from fixpoints A,B and C. Positions of point P are calculated form measurments A and B, from measurements B and C, and from measurements A and C.

Point D has been selected as a reference point. Its position was chosen on sound rock, to be visible from the other fixpoints, and so that the lines to it from the other fixpoints would pass through similar atmospheric conditions to those from fixpoints to the surveyed points on the dam. All fixpoints are occupied with the DME, and measurements are made to all fixpoints. The distance to the reference fixpoint D should be read twice, in order to check for drift in the instrument or changes in the atmosphere. For many single station the lines should be measured within one half an hour. When all the fixpoints have been occupied and all the possible lines measured, the figure may be reduced to a series of triangles. The next step is to reduce each line to a chord distance on the spheroid so the angles can be calculated and geometric checks applied to figures. The observed and the corrected spheriod distances are recorded. The corrected spheroid distances are used to form corrected ratios, which in turn are used to calculate the angles of triangles in the control net. After the control net has once been measured, using temperature and pressure corrections, possible movements of fixpoints may be monitored by comparing observed ratios. When positions have been established for the fixpoints in the control net, the length from the fixpoint to the reference point is used to monitor the refractive index for lines to points on the dam. This may be done by assuming the distance to the reference to be correct. This correct distance to the reference point together with the observed distance provide a correction factor for refractive index that may be applied to other lines form the fixpoint to points on the dam.

Temperature and pressure measurements are made only on the control lines and only the first time a study is made at the particular dam. The next time the dam is visited, perhaps 6 or 12 months later, it will not be necessary to measure refractive index. Possible movement in the control figure may be checked at that time by a comparison of ratios of observed distances.

PRECISE LEVELING

Vertical movements may be detected through a series of precise levels. The points to be determined by leveling are chosen in the cross-section-planes which already contain the points to be determined by intersection. For levelng purposes, the crown of the dam is the most accessible place. Leveling points at the foot of the power dam also is of interest, but they are far less accessible.

No matter what the structure size and number of structure points for which elevation is to be determined, at least 3 primary bench marks are to be established. Primary bench marks should be set-up 50-100 m one from another at suitable locations out of the pressure zone, possibly in the same level that points on the structure of which location is to be determind. The precision leveling rod with graduation of Invar should be used to eliminate all errors associated with invar leveling. Levels recommended for this work are: Zena NI 002, Zeiss Ni 1, Wild N3.

ALIGNMENT OBSERVATION

When this method is to be used, a convenient straight base line must be selected along the structure or section of concern, not impaired visually, free from possible damage to survey points, and not affecting efficient usage of the facility, etc. Concrete observation pillars are erected at each end of the base line. Survey points (alignment points) are installed along the base line and are usually placed at each side of monolith joints in concrete sections. The accuracy in offset measurements during actual surveys, depends on the distance between the theodolite and the Micromanipulator. Offsets can be measured with Micromanipulator, with accuracy 0.2 mm. In this method:

Distances and angles are measured between observation points and reference points to check for movements;

Distance measured between observation pillars at each end of base line (average used in computations);

Distances measured to all survey points, distances measured twice and the average used in computations;

Theodolite is set on concrete pillar and foresight obtained on the terminal pillar at the far end of the base line. Micromanipulator is set on the first survey point and deviation measured with the direction of movement noted. Minus (-) indicates movement down-stream and plus (+) indicates movements up-stream. Micromanipulator is then moved to the next point and the same procedure followed. When the distance from sighting pillar to the Micromanipulator becomes shorter, the Theodolite is reset and procedure followed;

Elevations are obtained on all survey points;

Survey data are prepared for and reduced by Automatic Data Processing Equipment;

Comparison, plotting, and analysis of survey results.

GIGERWALD DAM

The application of geodetic methods for the determination of the deformations can be shown on the example of recently built the Gigerwald Dam.

The Gigerwald dam is a double curvature arch dam of medium size, situated in the eastern part of Switzerland. The example of the Gigerwald dam is used here because on this dam geodetic deformations measurements were designed in virture of the application of Mekometer ME 3000 and new method of measurements of traverses by Distometer.

MEKOMETER ME 3000

The most important recent advance in surveying has been the introduction of modern optical distance measuring instruments. For deformation measurements Kern Mekometer ME 3000 is recommended. (Fig. 6).

Mekometer is an electro-optical distance measuring instrument with high resolution at short and medium distances. The internal accuray of the Mekometer is $\pm$ (0.2mm + 1 10^{-6} D). It is obtained by means of the high modulation frequency and the internal electronic frequency tuning. The opto-mechanical determination of the phase difference is almost free from instrumental errors.

In contrast to conventional amplitude modulation, the polarization of an optical carrier is modulated. The high modulating frequency of about 500 MHz provides a high absolute distance resolution and permits an optical phase measurement by means of a variable light path instead of an electronic phase meter. The modulation wave length is fixed by the resonance of a relatively small microwave resonator containing air under atmospheric conditions. Thereby the distance scale is affected by atmospheric changes much less than with instruments operating with fixed modulation frequencies and therefore with variable modulation wave lengths. The determined distance is automatically corrected for the atmosphere at the instrument station.

The ME 3000 is at present the most accurate distance measuring instrument available. Accuracies of a few tenths of a millimeter can be obtained without great effort or outlay of instrument.

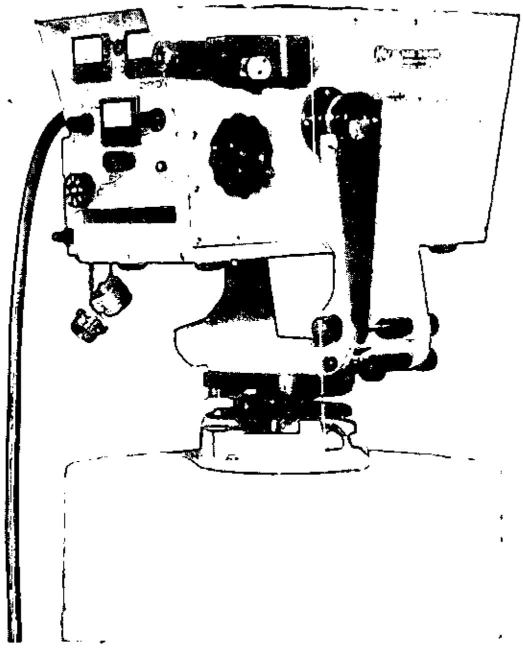


Fig. 6 Mekometer ME 3000

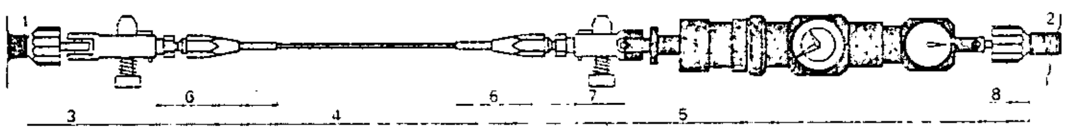
DISTOMETER ISETH

The Kern Distometer (Fig. 7) is a precision instrument for measuring distances by means of invar wires. It is particularly suited to determine precisely variations of distances and lengths when investigating structural deformations and shifts. The instrument was designed by the Institute for Road-, Railroad- and Rock Engineering of the Federal Institute of Technology, Zurich, Switzerland.

The entire equipment consists of mechanical components only. Therefore, it is extremely reliable and its operation is independent of power sources. Distometer measurements are simple, quick and are obtained with a minimum of personnel. An observation team consists of 2-3 persons where 1 or 2 of them may manipulate the invar wires. The length of the invar wire may vary between 1 m and 50 m. The measuring range for length deviations is 100 mm.

The accuracy of a measurement is -0.02 mm for wire lengths up to 20 m. For longer wires, it is ± 1 10^{-6} if the distance (m.s.e.).

Equipment for measuring length deviations with invar wires consists of three main components: the tension gauge, the displacement gauge and the invar wire.



Operational Distometer Equipment
1 Setting bolt, welded to object
2 Setting bolt, set in concrete
3 Swivel joint with adapter to wire coupling
4 Invar wire
5 Distometer ISETH
6 Wire coupling
7 Adapter to wire coupling at Distometer
8 Swivel joint at Distometer

Fig. 7

Tension Gauge

During measurement, this device holds the invar wire under the required tension. It consists mainly of a precision steel spring whose elongation is a measure of the tension affecting the invar wire. The elongation of the spring can be preset to a required value on the gauge.

Displacement Gauge

A second gauge serves as displacement meter and measures the distance between the Distometer and the attached end of the invar wire.

Invar Wire

Under a constant preset tension, the invar wire provides a uniform length stability largely independent of temperature. The wire is equipped with couplings which provide a precise connection to the Distometer on one end and to a secured point on the other. The equipment is complimented by setting bolts to secure the reference points at the objects to be measured and two swivel couplings which are inserted between the setting bolt and invar wire on one side and setting bolt and Distometer on the other. The use of an adapter plate provides for measurements between points whose position is fixed by Kern centering plates on pillars or consoles. The invar wire will be cut to the required length as needed in each individual case and fitted with couplings on both ends. The individualized wires are stored on drums so that they can be used again on subsequent measurements. A tension clamp holds the free end of the invar wire to the drum.

In order to obtain absolute measurement results, the entire Distometer system, i.e., invar wires and couplings have to be calibrated on an appropriate base line in the conventional way. However, the calibration unit provides for proper calibration where relative measurements are required, assuming that the length of the invar wires remain constant. In special cases, e.g. (for example), when very little redundancy is obtained in the measurements, it is recommended to measure each distance with several wires. With this method, the stability of the wires can be checked. Geodetic measurements of the highest accuracy are best executed with invar wires of 1.65 mm diameter. These are lightly more cumbersome to work with, however, they have a greater stability.

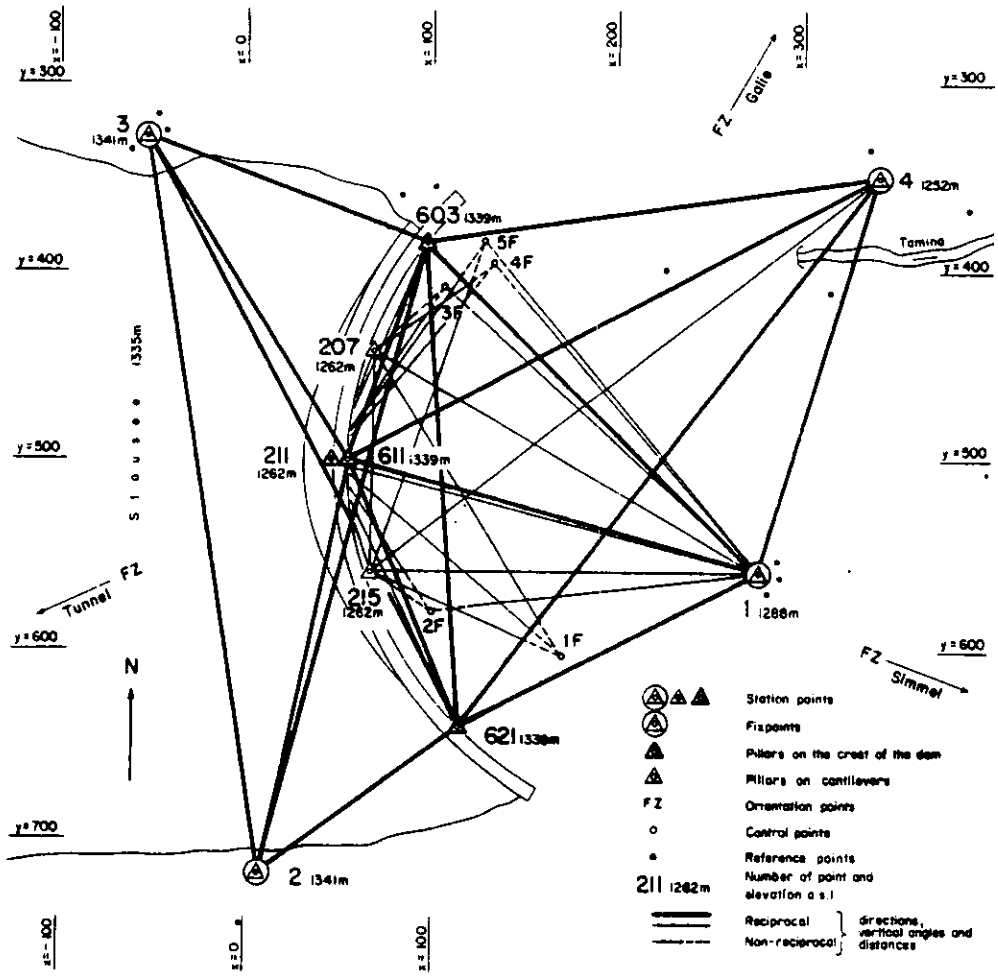


Fig. 8 Triangulateration scheme: complete measurement

The Gigerwald scheme and the extent of a complete measurement are shown in Fig. 8.

The pillars 603, 611 and 621 on the crest of the dam as well as the pillars 207, 211 and 215 relate the traverses in the control galleries of the dam to the triangulateration. The cantilevered pillars 207, 211 and 215 at the elevation of control gallery G2 are at exits of the dam. The lower control gallery G1 is already below the earth fill in front of the dam. Each target 1F to 5F in the rock is observed with three vectors. To enable the measurment of distances t the points in the rock, the placement of reflectors was made possible on these points.

For observations of directions the theodolite Kern DKM 3 was used with the collimation level. The same instrument was used for observation of vertical angle. Instrument and target numbers were recorded in order to take into account the individual heights of instruments for the evaluation.

With deformation measurements on dams very often great differences in elevation are experienced. To be able to determine unobjectionable coordinates on the basis of slope distances precise elevations are a necessity, even if these elevations are of no further interest. It will hardly be suitable to determine differences in elevation by leveling. The trigonometric determination of elevations is therefore of great importance.

The distances were observed by means of a Mekometer ME 3000. For each station the observations were carried out in the chronological order: distances, vertical angles and directions. This arrangement had the advantage that observations of distances forth and back took place mostly at different days. The forth and back measurements were continuously compared. At the begining of the measurement, to allow the standard cavity to adapt the temperature of the ambient air, instrument was left for 30 min. switched on, i.e. with the ventilator running. The temperature variations in the whole scheme were recorded.

For the evaluation of the triangulateration scheme for the Gigerwald dam the following procedure was used:

- Three-dimensional adjustment; x,y coordinates in a local projection system, z ellipsoidal elevations,

- Additional unknowns in the adjustment were the components of the deviation of the vertical,

- The triangulation net was adjusted as a free net without constraints. Scale, position and orientation of the net were determined by means of fixpoints, i.e. by Helmert transformation with regard to the coordinates y and x and by suitable conditions with regard to elevations and deviations of the vertical.

As a result of an adjustment computation, adjustment corrections were obtained and added to the actual observations to get adjusted observations. The calculated coordinates, elevations a.s.o. together with the adjusted observations form a consistent system of correlated values.

The evaluation was based on the following mean errors a priori:

Directions 2^{cc}
Vertical angles 3^{cc}
Distances (Mekometer) 0.3 mm + $1.5_{10}{-6}$ * D

The ratio mean error a posteriori/mean error a priori was 0.74, accidentally for both adjustments.

Even with the highly reduced scheme very good results can be obtained if a Mekometer is used.

The scheme of the reduced measurements is shown below in Figure 9.

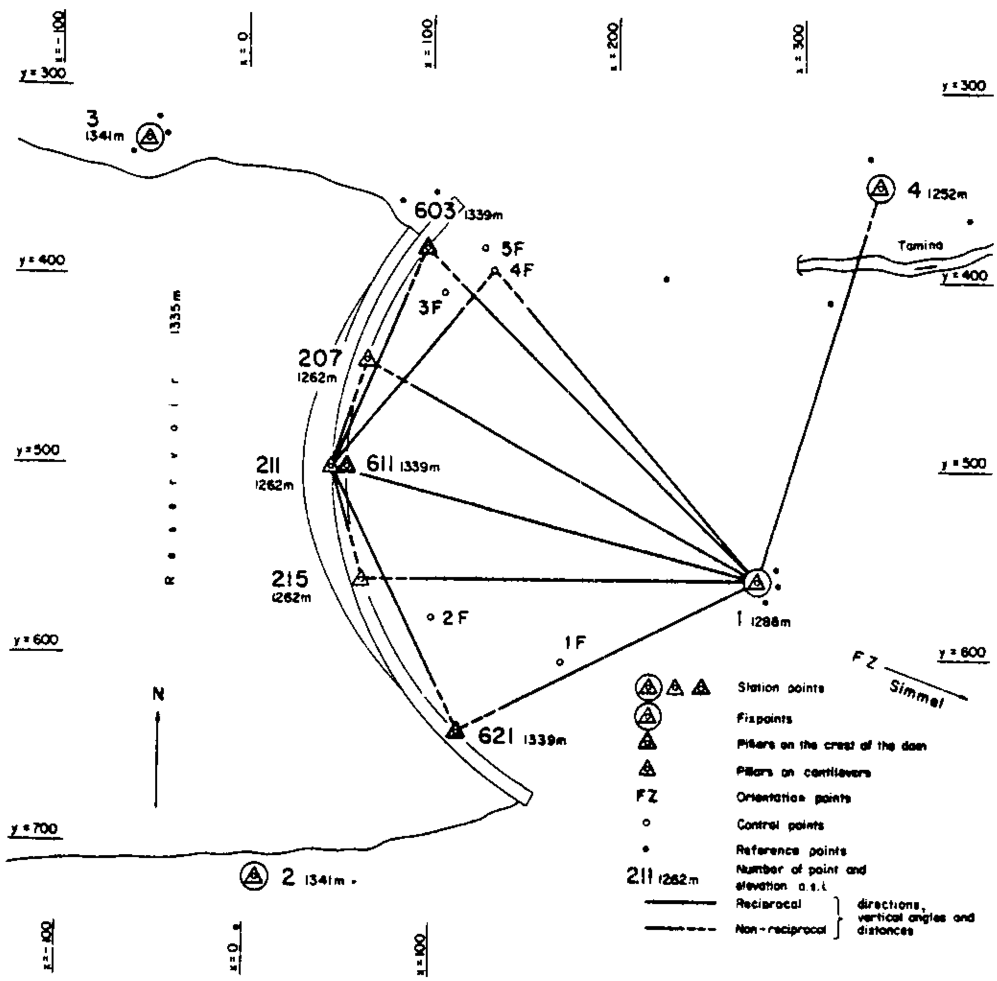


Fig. 9 Reduced measurement

The displacements of the points 207, 211, 215 and 621 may be deduced from pendulum readings assuming that the anchor points of the inverted plumb-lines are stable. It was one of the main purposes of the reduced measurements to check this assumption. Furthermore the points 603 and 4F are observed to control the left abutment. The observations were carried out according to the information giver above. The evaluation took place in the field with the help of a pocket calculater HP67.

The example of the Gigerwald dam shows how relative displacement measurements for the entire structure can be obtained by application of traverses.

The linear arrangement of the traverse points and the bench marks is a result of the layout of the control galleries in the dam. In view of the distance measurement with invar wires the distances between the points are equal and depend on the length of the blocks of the dam. For Swiss double curvature arch dams this is, as a rule, 16 m. Contingent on the curvature of the dam traverse sides of 32 to 96 m length were chosen at Gigerwald. Between successive traverse points one or two alignment points were inserted.

For the continuous monitoring of the Gigerwald dam pendulums are installed in 4 blocks. These pendulums are used to relate the traverses that are situated one upon the other. For that purpose the pendulum wires are observed from the traverse points by direction and distance. Pendulums and traverses form a kind of three-dimensional traverse net. Included are particularly the traverse points at the end of the control galleries, partly situated in the rock, and the anchor points of the inverted pendulums situated in the rock, and the anchor points of the inverted pendulums situated in up to 40 m deep bore holes. The displacements of these points in the abutments and the foundation are of very great interest. They are normally small and require therefore a great precision.

The traverse net has to be related in a suitable way to the triangulation. Absolute displacements referring to the fixpoints of the triangulation net are then obtained for all points of the traverse net. At Gigerwald the two following methods were used to relate the traverse net to the triangulation:

- Relation of the pillars on the crest of the dam (A in Fig. 10); The method is shown in Fig. 11

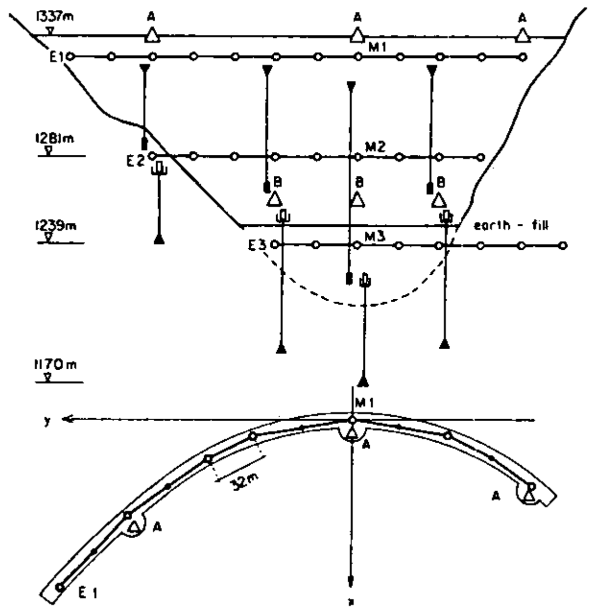


Fig. 10 Traverse net: traverse- and alignment points, pendulums, connecting points

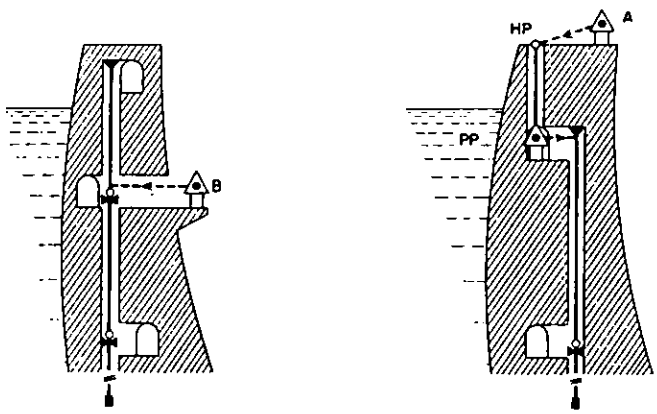


Fig. 11 Relation of traverse net to triangulation

For the optical plumbing PP-HP, a theodolite with elbow eyepieces or a precise vertical collimator may be used. The center of the optical plumbing target HP will be determined from a pillar A by a vector.

- Relation to the pillars on the cantilevers (B in Fig. 10): Fig.11 shows the method. The plumb-line is determined from pillar B by means of a vector.

The monuments of the traverse points have to guarantee precise measurements during a long time and in spite of adverse conditions. A galvanized steel beam was attached to the wall with two or three anchors. On it, a precisely tooled centering plug of stainless steel was fixed. The washers serve for the precise leveling of the centering plug. The centering plug can be exchanged if necessary: its position is reproducible by means of the drill hole in the steel beam and the elevation is given by the known thickness of the washers.

The elevation of traverse- and alignment points as well as additional bench marks are leveled. For the connection of the various levelings the differences in elevation between the crest of the dam and the various control galleries have to be determined by means of invar wires. Fig. 12 shows the arrangement of elevation transfer at Gigerwald.

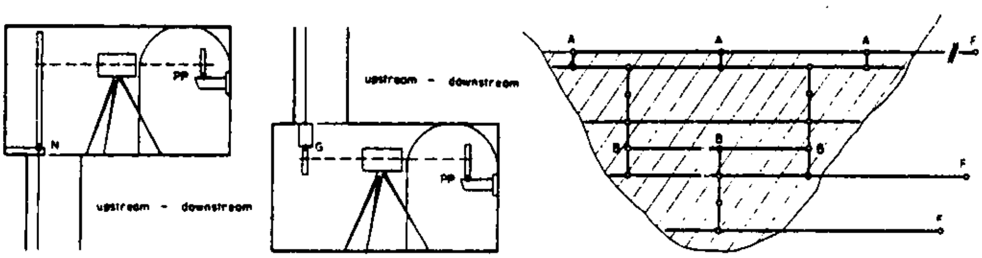


Fig. 12

The leveling net can either be related to the points A and B of the triangulation or to fixpoints F deep in the rock formation. Permanently installed wires serve for the elevation transfer.

On all traverse points the directions are observed to the adjacent traverse- and alignment points as well as to the pendulum wires. The KDM2-A was used for this sort of work. Target were made of plastic in cone shape illuminated from inside. Due to possible refraction, the observation of directions was repeated on another day. For the measurement of distances invar wires were used. At Gigerwald all distances were observed twice two different wires, using a total of 4 wires.

The differences in elevation were determined by means of a high precision leveling. A 30 cm long scale of invar was set up on the traverse and alignment points. At sights of 16 m (half distance between leveled points) the standard deviation of a mean difference in elevation is only 0.04 mm.

Coordinates of the traverse points were calculated by means of a two-dimensional simultaneous adjustment. The coordinates of the connecting points (603, 611, 621, 207, 211 and 215) were taken over from triangulation. The plane coordinates of all traverse points as well as those of the different pendulum wires were introduced as unknowns into the adjustment. This computation, as well as the one of the triangulation, was done with the same program on a large computer. The observed slope distances of the traverse were reduced by means of approximate elevations. As the slopes of the observed distances are small throughout, the approximate elevations do not have to be very precise. According to the two different lengths of wires two scale factors were introduced as unknowns into the adjustment.

Instead of a rigorous adjustment of the leveling net provisional elevations were calculated for each section of the leveling net, based on leveled differences in elevation and the elevation of an arbitrary initial point. Subsequently, corrections were calculated for each section as unknowns of an adjustment so that the discrepancies for all elevation transfer and the connections to the fixpoints F (Fig. 12) became a minimum.

Figure 13 shows the displacement between first and second measurement of the traverse points in the uppermost control gallery. The first measurement was executed before the start of the filling of the reservoir. At the second measurement the reservoir level was 10 m below the scheduled maximum.

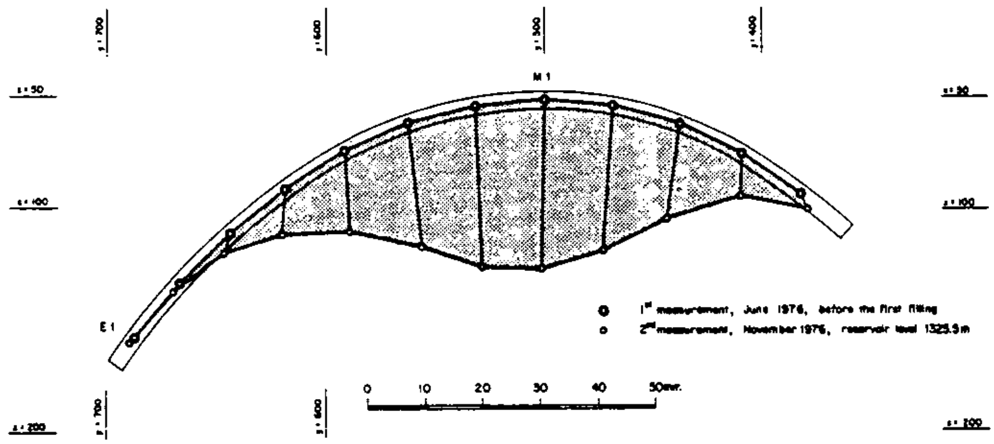


Fig. 13 Displacements of traverse- and alignment points in the uppermost control gallery

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CALIBRATION BASE LINES
FOR ELECTRONIC
DISTANCE MEASURING INSTRUMENTS (EDMI)
IN NEW JERSEY AND THEIR USE

by

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INTRODUCTION

During the early 1970's, the number and types of electronic distance measuring instruments (EDMI) dramatically increased. Their use was expanded to cover almost every conceivable surveying problem. Quality assurance became a pressing concern. But, unlike tape or wire standardization, no recognized agency or organization was responsible for calibration standards for EDMI.

The calibration of electronic distance measuring instruments involves the determination or verification of instrument constants and the assurance that the measured distances meet accuracy specifications. Although it is not necessary to utilize a measured distance to determine or verify instrument constants, the verification effort is reduced when an accurately measured distance can be used. However, to assure that an instrument is measuring properly, a known distance of high accuracy or, preferably, a sequence of distances forming a calibration range or base line is required. Experience shows that a base line consisting of four on-line monuments spaced at intervals of 150 m, 400 to 430 m, and 1,000 to 1,400 m will meet the needs of the users.

In 1974, the National Geodetic Survey (NGS) of the National Ocean Survey (NOS) began establishing a series of calibration base lines for this purpose. This article was prepared in conjunction with this program and is directed to the land surveyors of New Jersey who use EDMI. General observing procedures are outlined, and an analysis of the observations is developed. Detailed formulas are given for determining the geometric transformation of distances. An analysis is made of error sources affecting the ambient refractive index.

Surveyors have, for the most part, obtained excellent results from EDMI. This leads to the temptation to accept the instrument on faith. Such an approach, however, must be tempered with some systematic plan to ensure that a minimum accuracy requirement is maintained throughout the life of the instrument and, equally important, to provide legal documentation against possible lawsuits arising from its use.

The surveyor will always be held accountable for assuring that the EDMI provides acceptable results. Calibration base lines provide one method of monitoring the accuracy of EDMI.

Finally, the location of established calibration base lines in New Jersey is given for the convenience of the users. Seven calibration base lines were established in 1978 with the help of N. J. Geodetic Control Survey (see article G. Halasi-Kun: Geodetic Survey Activities in New Jersey in this volume) and they will be calibrated by the National Geodetic Survey in 1979. Their locations are apparent from the Fig. 1 of this article.

SUGGESTED GUIDELINES FOR USING CALIBRATION BASE LINES

The solution to most complex problems can only be obtained by a thorough investigation of all its various facets. This approach is certainly true for the problem of calibrating EDMI. Because numerous variables, ranging from human intervention to atmospheric deviation, influence the effectiveness of EDMI, the theoretical basis and operation of each particular instrument should

1. ANDOVER TOWNSHIP

2. PARSIPPANY-TROY TOWNSHIP

3. SKILLMAN TRAINING CENTER, MONTGOMERY

4. MERCER COUNTY COLLEGE, WEST WINDSOR

5. BURLINGTON AIRPORT, LUMBERTON

6. MILLVILLE CITY AIRPORT, MILLVILLE

7. MIDDLESEX COUNTY COLLEGE, EDISON

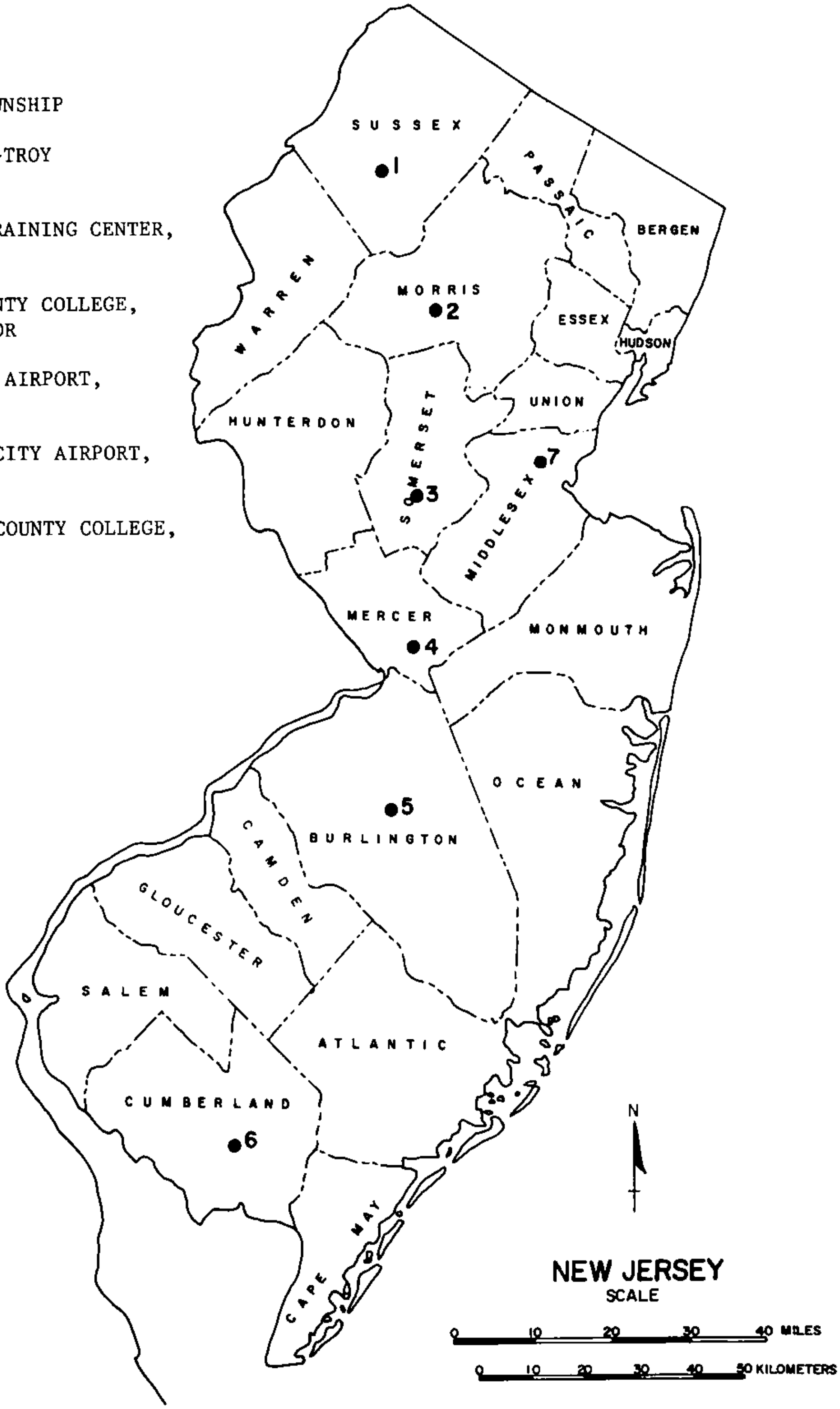

FIGURE 1.--Location of calibration base lines for EDMI in New Jersey, 1978.

be fully understood. This document gives only the outlines of general procedures applicable to most EDMI. For detailed instructions, various professional papers, textbooks, and manufacturers' manuals should be consulted. (See bibliography.)

The calibration process can be considered as having two phases: (1) the acquisition of distance observations, and (2) the analysis of the observations. Valid observational procedures can be invalidated by a distorted analysis and vice-versa. Therefore, the full potential of the calibration base line can be realized only if great care is exercised in performing both phases of this process.

ACQUISITION OF OBSERVATIONS

Because observational procedures lay the foundation for acceptable results, this phase must be investigated and prepared for in detail. Accessory equipment, such as thermometers, barometers, psychrometers, tribachs, and tripods, should be checked and, where applicable, checked against a standard. In addition, the functional relationship of each accessory device to the distance measurement must be understood.

Perhaps the most important element in determining the operational limit and overall accuracy of EDMI is the maintenance of an accurate log of the entire observational procedure. Also, a continuous log provides a history of the instrument that may be used later either to isolate changes in instrument characteristics or for legal verification purposes.

It is suggested that the following information be recorded at the time each observation is made:

1. The names (or numerical designation) of the stations from and to which the observations are made.

2. Instrument/tape model and serial number.

3. Reflector model and serial number.

4. Date and time of observation (Local time - 24 hour-clock).

5. Instrument/reflector constants*.

6. Height of instrument/reflector above marks*.

7. Station elevations*.

8. Instrument/reflect : eccentricity*.

---

*Units of measurement and, if applicable, the reference datum should always be shown.

9. Atmospheric observations*.

    a. Temperature
    b. Pressure
    c. Psychrometer readings.

10. Weather conditions (clear, cloudy, hazy, rain, snow, fog, etc.).

11. Any unusual or problematic conditions, e.g., dust blowing across line or measuring across a gulley 30 m wide and 3 m deep.

Suggested Procedures for Using Calibration Base Lines

The present configuration for calibration base lines has monuments located at 0 m, 150 m, 430 m, and 1,400 m; some variations may occur because of topographical restrictions at the base-line site. This configuration provides six distinct distances for testing EDMI. For cases where additional marks have been set, the number of distinct distances can be determined by $n(n-1)/2$ where "n" is the number of monuments.

Before designing the calibration test, two questions must be answered:

1. For what order of work is the instrument going to be used?

2. Do the manufacturer's specifications indicate it is possible to obtain that order of work?

If most of the work falls into the second-order classification, then test procedures should be developed accordingly. If the manufacturer claims an accuracy of 1:10,000, then, regardless of the effort expended on the test, it is unlikely accuracies of 1:20,000 can be obtained.

For a complete calibration test, the recommended procedure is to perform distance observations both forward and backward over each section of the base line on two separate days. Care should be taken to obtain an as wide as possible range of weather conditions. For example, this can be done by starting observations in the early morning on one day and in the afternoon on the next day. The preferred method is to perform the observations on two successive days: once during daylight and once during the night.

A less accurate test, but one which is sufficient for most needs, consists of measuring all sections of the base line in every combination both forward and backward, i.e., 12 distances would be observed for a four-mark base line. This is recommended as the standard calibration test; the resulting higher confidence in the results far outweighs the extra effort involved.

If it is decided to observe fewer lines than is required for the standard calibration test, it is perhaps more orderly to begin the observing scheme at the "0 m" mark. Measurements should then be made to each of the other monuments in turn. However, regardless of which monument is chosen, the absolute minimum observing scheme is to measure the distances to all other points in the base line; i.e., for a four-mark base line, a minimum of three measurements must be made.

Observing Procedures

1. Set up the instrument and reflector directly over the points to which the published measurements are referred. Care must be taken to assure not only that the instrument and reflector are centered over the points, but also that the tripods are firmly set. Careless centering will defeat the entire purpose of using the base line. In general, there should be little difficulty in centering the equipment to 1 mm or less.

Note: If a quick test of an instrument is to be performed, it may be expedient to set the heights of the instrument and reflector (or slave unit) at approximately the same height. If the difference between the heights of the instruments above the marks is less than $(0.001s/\Delta H)$ m, where s = horizontal distance beween marks, and ΔH = difference of elevation between marks, then no geometric corrections need be applied to compare the measured distance with the published mark-to-mark distance. For example, if $s = 1{,}650$ m, $\Delta H = 10$ m, then the allowable difference between the heights of the instrument is 0.165 m.

2. Initial warmup of the instrument should be performed according to manufacturer's instructions.

3. Measure and record heights of instruments and reflectors above the marks.

4. Read and record meteorological observations (dry and wet bulb temperatures and barometric pressure). Since ambient meteorological conditions have a direct bearing on the results of the distance observations and the near-topography atmosphere is the most turbulent, all precautions should be taken to secure accurate meteorological observations. Ideally, temperatures and pressures should be observed along the entire line during the observation sequence. In most cases, this will not be feasible, so some compromise must be made. In decreasing order of preference, the following measurements should be made:
(1) temperatures and pressures at both ends of the line, both prior to and following the distance observation, and (2) the temperature and pressure at the instrument site.

If the deviations in dry bulb (Δt) and wet bulb ($\Delta t'$) temperatures are 1°C (1.8°F) and the deviation in barometric pressure (Δp) is 3 mm (0.1 in)* of Hg, then the following table gives the error (in parts per million for each component) that will be introduced into a distance observation.

Type of instrument & applicable temperature range	$\Delta t = 1^{o}C$ ppm	$\Delta t' = 1^{o}C$ ppm	$\Delta p = 3$ mm of Hg ppm
Lightwave, including infrared ($0^{o}C-30^{o}C$)	1	0	1
Microwave 0^{o}	4.6	5.5	1
10^{o}	4.5	6.9	1
20^{o}	4.5	9.8	1
30^{o}	4.7	12.5	1

Note: A combination of errors of the magnitude of those given in the previous table may yield significantly erroneous results. (For a thorough discussion of the meteorological effects on the measured distances, see appendix II.)

5. Perform the distance observations. The number of repetitions over each section should follow the manufacturer's recommendations or those suggested in the professional literature. Several instruments have been developed with one or more features that reduce the computational effort usually associated with electronic distance measurements. These features are:

a. A direct input facility for meteorological corrections.

b. A display that optionally gives results in feet or meters, or both.

c. A combination angular and distance instrument that reduces observed slope distances to horizontal distances.

If the instrument being tested has one or more of the above features, additional observations should be taken to ensure the accuracy of the features. For instance, if the instrument being tested permits encoding meteorological data, two complete sets of distance observations should be observed. One set should be observed with the values set at zero and a second set observed with the actual atmospheric data entered into the EDMI. Distances determined using zero meteorological values should then be reduced independently and compared with distances determined when meteorological data were encoded into the EDMI.

MATHEMATICAL REDUCTION

After observations are made, they should be reduced to a common datum. The reduction can be divided in two stages: one dependent on meteorological conditions and the other dependent on geometrical configurations. (This is true only if no corrections were applied during or because of the observing sequence.)

*0.1 in Hg corresponds to a change in altitude of approximately 30 m (~ 100 ft).

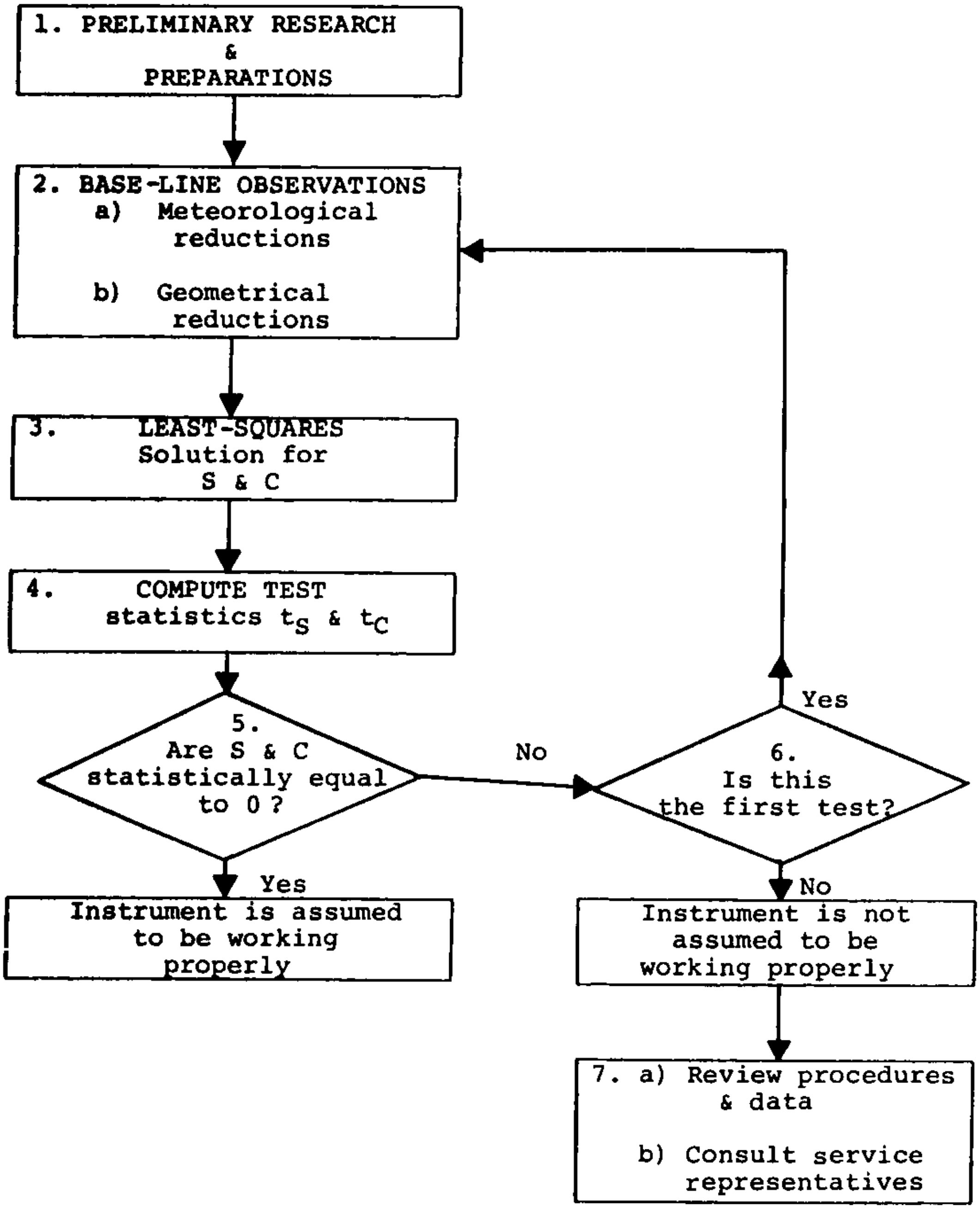

FIGURE 2.--Schematic of text procedure for EDMI at calibration base line.

Reductions for Meteorological Conditions

The correction (ΔD) to the measured distance (D) for actual atmospheric conditions is given by

$$\Delta D = (n-n_a)\ D \qquad (1)$$

and the corrected distance by

$$D_o = D + \Delta D \qquad (2)$$

where

n = nominal index of refraction as recommended by the nanufacturer,

n_a= actual index of refraction, and

n_a is dependent on whether the EDMI has a lightwave source (including infrared) or a microwave source.

Various computational and mechanical methods have been used for determining the refractive index for ambient conditions of the atmosphere. The National Geodetic Survey currently (1977) uses the following equations for the computation of n_a.

Lightwave (including infrared) source

The group refractive index (ng) for modulated light in the atmosphere at 0^o Celsius, 760 mm of mercury (Hg) pressure, and 0.03% carbon dioxide is:

$$n_g = 1 + \left(2876.04 + \frac{48.864}{\lambda^2} + \frac{0.680}{\lambda^4}\right) \times 10^{-7} \qquad (3)$$

where λ is the wavelength of the light expressed in micrometers (μm).

The index of refraction of the atmosphere at the time of observations due to variations in temperature, pressure, and humidity can be computed from:

$$n_a = 1 + \frac{n_g-1}{1+\alpha t} \cdot \frac{p}{760} - \frac{5.5\ e}{1+\alpha t} \times 10^{-8} \qquad (4)$$

where

$\alpha = 0.003661$

e = vapor pressure in mm of Hg

p = atmospheric pressure in mm of Hg

t = dry bulb temperature in degrees Celsius (^{o}C).

Microwave source

The refractive index of the atmosphere for radiowaves differs from that of lightwaves. This is given by:

$$n_a = 1 + \left[\frac{103.49p}{(273.2+t)} + \frac{495,882.48\ e}{(273.2+t)^2} - \frac{17.23\ e}{(273.2+t)}\right] \times 10^{-6} \quad (5)$$

where all variables are as defined for lightwaves.

A modified form of this equation is:

$$n_a = 1 + \left[\frac{103.46\ p}{273.2+t} + \frac{490,814.24\ e}{(273.2 + t)^2}\right] \times 10^{-6} \quad (6)$$

Tables for e may be found in the Smithsonian Meterological Tables (List 1963). For the temperature range usually encountered in actual practice, the following equations provide sufficiently accurate results:

$e = e' + de$

where

$e' = 4.58 \times 10^{a}$
$a = (7.5\ t')/(237.3 + t')$
$de = -0.000660(1+0.00115\ t')\ p\ (t-t')$
t' = wet bulb temperature in ^{o}C.

Note: See Meade (1972) for a comprehensive discussion of various equations for computing refractive index. This article also contains tabular values for e' at $1^{o}F$ intervals.

Reductions for Geometric Configurations

After applying the meteorological correction, the observed distance should be corrected for any eccentricities of the instrument or reflector (or slave unit) and their constants. In the following analysis, distance d_1 should then be reduced to the horizontal distance by:

$$D_H = \left(D_1^2 - \Delta h^{\ 2}\right)^{1/2} \quad (7)$$

where $\Delta h = (H_j = \Delta H_j) - (H_i + \Delta H_i)$

H_i = elevation of station i

ΔH_i= height of instrument/reflector above station i

H_j = elevation of station j

ΔH_j= height of instrument/reflector above station j.

ANALYSIS OF CALIBRATION BASE-LINE OBSERVATIONS

A prerequisite to analyzing the observations is an awareness of the numerous possibilities for introducing errors into the distance observations. Some of these sources are:

1. Centering errors.

2. Improper pointing, voltage, or readings.

3. Errors in height of instruments or reflectors.

4. Measuring under extreme conditions or in areas where external factors unpredictably affect the instrument.

5. Unfamiliarity with the operating condition of the EDMI.

6. Incorrect meteorological data.

7. Improper alignment of optics.

8. Incorrect values for the constants of the reflectors or instruments.

9. Changes in the frequency of the instrument.

Of the above, most can be minimized by following proper procedures and exercising care in obtaining the observations. The others are predominantly attributable to natural aging or to mechanical changes in the structure of the instrument.

These latter errors can be determined only by frequent and periodic observations over a calibration base line and then only through proper evaluation of those observations.

There are no hard and fast rules that govern the analysis of calibration base-line observations. Almost every case must be treated individually. Of prime importance is the original intent for making these observations.

In the introduction, we stated that the surveyor will always be held accountable for providing acceptable results. Therefore, acceptability must be the goal. However, to prove a measurement is acceptable it must be demonstrated that the measuring instrument is reliable and accurate. Tests for reliability and accuracy are not easy. Such conclusions at best are based on arbitrary methods.

Most EDMI manufacturers routinely attribute certain accuracies to their instruments. Although these accuracies should reflect the instrument's ability to measure a "true value," they may, in fact, indicate only the repeatability (precision) of the instrument or test results performed under laboratory conditions. Theoretically, if the accuracy statistic is given in terms of a standard error (σ), 68.3% of the differences between a "true value" and an observed value should fall within the stated specification. Therefore, this value could be used for decision purposes, i.e., as a test statistic. However, the above is true only for large samples and for known standard errors. Both of these requirements are rarely satisfied. In addition, by using this test statistic for rejection purposes another type of error may be committed, i.e., the rejection of valid observations. To reduce the possibility of rejecting a valid observation, a limit of 3σ (three times the standard error value) is usually chosen for deciding if an observation is acceptable or not acceptable. Theoretically, 99.7% of the differences should fall within the 3σ range.

The sequence of operations to perform an analysis of the base-line observation is:

1. Compute the differences between observed values and published values.

2. Analyze these differences. If 99.7% of the observations fall within three times the manufacturer's stated accuracy and 68.3% fall within the manufacturer's stated accuracy, the instrument can be accepted as working accurately and reliably.

If the differences do not agree within above specifications, then a different method must be used to determine an instrument's acceptability. Various approaches can be designed for this purpose.

One such approach is to examine the differences between observed values and published values and determine if the difference is a constant or is proportional to the distance being measured (scale error).

If the differences appear systematic, the instrument constant can be redetermined over the 150-m length and the distances recomputed. If the comparison now shows agreement with the published values (within the above specifications), the solution is considered to be complete and the instrument accepted.

If the differences become significantly larger or smaller as the distances increase, the proper approach is to determine this scale correction. Caution should be exercised in applying the scale correction to other measured distances. Tests have shown that atmospheric sampling techniques in near-topographic situations (i.e., at ground level) can introduce errors in the range of 5 to 6 parts per million.

Therefore, a scale correction should be applied only when an instrument has historically shown a similar scale error under various meteorological conditions.

THE LEAST-SQUARES METHOD

Most calibration tests do not show a pattern of differences as clear as those outlined above. Also, many methods rely on a hit-or-miss approach. The preferred approach is a least-squares solution that simultaneously determines a scale and a constant correction. This solution is based on the supposition that the differences can be attributed either to a scale correction or to a constant correction, or both. The basic equation for this solution is:

$$V = D_A - D_H - SD_A - C \tag{8}$$

where

S = a scale unknown

C = a constant unknown

D_A = the published horizontal distance corresponding to the distance observation

D_H = the observed distance reduced to the horizontal

V = the residual to the observed horizontal distance.

One equation of the above type is written for each observation.

The solution to this system of equations is very similar to the fit of a straight line in a series of points. The theory behind this process is given in many elementary statistics and calculus texts, and will not be presented here.

The solution is given by:

$$S = \frac{n\ \Sigma(D_A\Delta) - \Sigma\ D_A\ \Sigma\ \Delta}{n\ \Sigma(D_A)^2 - (\Sigma\ D_A)^2} \tag{9}$$

$$C = \frac{\Sigma(D_A)^2\ \Sigma\Delta - (\Sigma\ D_A)\ \Sigma(D_A\ \Delta)}{n\ \Sigma(D_A)^2 - (\Sigma\ D_A)^2} \quad \text{or} \tag{10}$$

$$C = \bar{\Delta} - S\ \bar{D}_A \tag{10a}$$

where

n = number of distances observed

Δ = $D_A - D_H$ = difference between the published horizontal distance and the observed horizontal distance

Σ = the summation of values. For example ΣD_A = the sum of the published distances involved

$\bar{\Delta} = \Sigma \Delta / n$

$\bar{D}_A = \Sigma D_A / n.$

In addition to solving for S and C, it is also useful to compute four additional statistics to assist in analyzing the acceptability of the test: the estimated standard error of S ($\hat{\sigma}_S$), a test statistic t_S, the estimated standard error of C ($\hat{\sigma}_C$), and a second test statistic t_C. These values can be computed using the following:

$$\hat{\sigma}_S = \left[\hat{\sigma}_0^2 \quad \frac{n}{n \Sigma D_A^2 - (\Sigma D_A)^2}\right]^{1/2} \tag{11}$$

$$\hat{\sigma}_C = \left[\hat{\sigma}_0^2 \quad \frac{\Sigma D_A^2}{n \Sigma D_A^2 - (\Sigma D_A)^2}\right]^{1/2} \tag{12}$$

where

$$\hat{\sigma}_0^2 = \frac{\Sigma V^2}{n-2} \quad \text{or} \tag{13}$$

$$\hat{\sigma}_0^2 = \frac{\Sigma(\Delta - \bar{\Delta})^2 - \frac{S}{n}\left[n \Sigma(D_A \Delta) - \Sigma D_A \Sigma \Delta\right]}{n-2} \tag{13a}$$

Eq. (13a) gives results that are computationally more correct. However, eq. (13) will give equal results if sufficiently significant digits are carried throughout the computations.

$$t_S = \frac{S}{\hat{\sigma}_S} \tag{14}$$

$$t_C = \frac{C}{\hat{\sigma}_C} \tag{15}$$

It can be shown that t_S and t_C follow the Student's t distribution, which is useful in analyzing small sample tests. For a more thorough explanation of the t statistic, see Mendenhall (1969, pp. 189-220).

Using eqs. (8) through (15), the following procedure may be used to analyze the calibration base-line test results:

(1) Compute S and C from (9), and (10) or (10a).

(2) Compute the residuals (V) from (8).

(3) Compute $\hat{\sigma}_0^2$ from (13) or (13a).

(4) Compute $\hat{\sigma}_S$ and $\hat{\sigma}_C$ from (11) and (12).

(5) Compute t_S and t_C from (14) and (15).

(6) Test the significance of S and C. For this we test the hypothesis (or supposition) that S and C are statistically equal to 0 by comparing the values of t_S and t_C against the critical values of $t_{0.01}$ d.f. (d.f. = degrees of freedom = n-2) as given in table 1. There are four possible results:

(a) The absolute value of t_S is less than $t_{0.01}$ d.f. Then it can be said that S is statistically equal to 0, and S need not be applied.

(b) The absolute value of t_S is greater than $t_{0.01}$ d.f. this implies S is statistically not equal to 0. However, because the determination of the refractive index at ground level is very difficult, the instrument should be retested at another time under considerably different atmospheric conditions.

(c) The absolute value of t_C is less than $t_{0.01}$ d.f. As above for S, C is statistically equal to 0 and need not be applied.

(d) The absolute value of t_C is greater than $t_{0.01}$ d.f. The value of C should be applied to all observations made with the instrument. Note: the constant determined by means of these procedures should not be confused with an instrument constant. For example, the observations could contain a constant error from the instrument, the reflector, or a miscentering. This error source cannot be specifically identified or divided into individual components. For these reasons, without additional independent observations, the constant determined should more properly be called a <u>system constant</u>.

Table 1.—Critical values of t for "degrees of freedom" (d.f.) at 0.01 significance level.

d.f. = n-2	$t_{0.01}$	d.f. = n-2	$t_{0.01}$	d.f. = n-2	$t_{0.01}$
1	63.657	7	3.499	13	3.012
2	9.925	8	3.355	14	2.977
3	5.841	9	3.250	15	2.947
4	4.604	10	3.169	20	2.845
5	4.032	11	3.106	25	2.787
6	3.707	12	3.055		

The preceding can best be illustrated by a few examples. However, before proceeding to the examples, a brief description of the published data will be given.

DESCRIPTION OF PUBLISHED DATA

As stated earlier, the present (1977) recommended configuration for calibration base lines consists of four monuments located at 0 m, 150 m, 430 m, and 1,400 m. This layout provides six distinct distances, as listed in the following format. (Where additional monuments are set, the number of distinct distances can be determined by the formula n(n-1)/2 where "n" is the number of monuments.)

FROM STATION	ELEVATION (M)	TO STATION	ELEVATION (M)	ADJUSTED DISTANCE HORIZONTAL (M)	ADJUSTED DISTANCE MARK-MARK (M)	S.E. (MM)
XXX......XXX	XXX.XX	XXX....XXX	XXX.XX	XXXX.XXXX	XXXX.XXXX	X.XX

The following should be noted:

1. The FROM and TO station names have been arbitrarily assigned and may not agree with the stamping on the disk.

2. Although the differential elevations are considered to be sufficiently accurate for the reduction of the measured distance, the elevations will not be integrated into the National Vertical Control Network, and therefore, should not be treated as bench marks.

3. The adjusted distances listed are the horizontal distances and the mark-to-mark distances. These distances are defined as the distance at the mean elevation of the two stations and the spatial chord distance between the centers of the disks, respectively.

4. The standard error is an estimated value determined from the adjustment and may be more of an indication of the repeatability of the instruments used for measuring the base line than of the actual accuracy of the base line. In this sense, the standard error may be optimistic.

*******FINAL*******

U.S. Dept. of Commerce - NOAA
NOS - Natl. Geodetic Survey
Rockville, Maryland 20852

CALIBRATION BASE LINE DATA
Beltsville Base Line
Source No. 13204

QUAD - 390763
State - Maryland
County - Prince Georges

List of Adjusted Distances

From Sta. Name	Elev.(m)	To Sta. Name	Elev.(m)	Adj. Dist.(m) Horizontal	Adj. Dist.(m) Mark - Mark	Std. Error (mm)
BELTSVILLE 150	47.44	BELTSVILLE 300	46.21	149.9929	149.9979	0.2
BELTSVILLE 150	47.44	BELTSVILLE 600	44.38	449.9990	450.0094	0.2
BELTSVILLE 150	47.44	BELTSVILLE 1800	50.54	1649.9959	1649.9988	0.2
BELTSVILLE 300	46.21	BELTSVILLE 600	44.38	300.0061	300.0117	0.3
BELTSVILLE 300	46.21	BELTSVILLE 1800	50.54	1500.0030	1500.0093	0.3
BELTSVILLE 600	44.38	BELTSVILLE 1800	50.54	1199.9969	1200.0128	0.3

FIGURE 3.--An example of the adjusted results.

EXAMPLES OF EDMI CALIBRATION TESTS

Example #1. The following example is an actual set of test observations performed by a private surveyor over the National Geodetic Survey's calibration base line at Beltsville, Maryland (see fig. 3 for the published data). The instrument to be tested was a short-range infrared EDMI with $n = 1.0002782$ and $\lambda = 0.9100$ μm. The instrument and reflector constant were assumed to be equal in magnitude but opposite in algebraic sign. The resultant system constant is thus assumed equal to zero. (Only one set of prisms was used throughout the test.) The manufacturer's stated accuracy for this instrument is $\pm$ 0.01 m $\pm$ D x 10^{-5} m. The observed distances and corresponding meteorological data are given below. The estimated accuracy of the temperature observations is "within a few degrees."

From Sta.	Height of Inst. (m)	To Sta.	Height of Inst. (m)	Mn Temp. (ºC)	Mn Pressure (mm of Hg)	Obs. Distances D (m)
150	0.20	300	1.53	20.0	760.7	149.9892
300	1.58	150	0.145	21.7	760.7	149.9897
150	0.20	600	1.56	20.0	760.7	449.9927
600	1.61	150	0.145	21.1	761.0	449.9851
150	0.20	1800	3.23	20.0	760.7	1649.9635
1800	3.24	150	0.145	18.9	760.7	1649.9783
300	1.58	600	1.56	21.7	760.7	300.0041
600	1.61	300	1.53	21.1	761.0	300.0018
300	1.58	1800	3.23	21.7	760.7	1499.9763
1800	3.24	300	1.51	18.9	760.7	1499.9972
600	1.61	1800	3.23	21.1	761.0	1200.0050
1800	3.24	600	1.54	18.9	760.7	1200.0070

The distances were corrected for atmospheric refraction using the following:

From eqs. (3) and (4)

$$n_g = 1 + \left[2876.04 + \frac{48.864}{(0.91)^2} + \frac{0.680}{(0.91)^4} \right] \times 10^{-7}$$

$$= 1.0002936$$

and

$$n_a = 1 + \frac{0.0002936}{1 + \alpha t} \cdot \frac{p}{760} - \frac{5.5\ e}{1 + \alpha t} \times 10^{-8}$$

Then from eq. (1)

$$\Delta D = (1.0002782 - n_a)\ D.$$

The distances were corrected for eccentricities, instrument constant, and reflector constant (the sum of which was equal to zero). They were then reduced to the horizontal distance using eq. (7).

The following equations were written in accordance with eq. (8).

$$V_1 = (149.9929) - 149.9899) - S \cdot 149.9929 - C.$$

$$V_2 = (149.9929 - 149.9905) - S \cdot 149.9929 - C.$$

$$V_3 = (149.9990 - 449.9916) - S \cdot 449.9990 - C.$$

$$V_4 = (449.9990 - 449.4849) - S \cdot 449.9990 - C.$$

$$V_5 = (1649.9959 - 1649.9600) - S \cdot 1649.9959 - C.$$

$$V_6 = (1649.9959 - 1649.9728) - S \cdot 1649.9959 - C.$$

$$V_7 = (300.0061 - 300.0003) - S \cdot 300.0061 - C.$$

$$V_8 = (300.0061 - 299.9984) - S \cdot 300.0061 - C.$$

$$V_9 = (1500.0030 - 1499.9739) - S \cdot 1500.0030 - C.$$

$$V_{10} = (1500.0030 - 1499.9906) - S \cdot 1500.0030 - C.$$

$$V_{11} = (1199.9969 - 1199.9866) - S \cdot 1199.9969 - C.$$

$$V_{12} = (1199.9969 - 1199.9858) - S \cdot 1199.9969 - C.$$

Using eqs. (9) and (10), S and C are then solved.

For any computational purposes it may facilitate operations to rearrange the above equations in the tabular form shown below.

(1) Obs.	(2) From	(3) To	(4) D_A (m)	(5) D_H (m)	(6) Δ (m)	(7) $D_A \cdot \Delta$ $(m)^2$	(8) V (m)*
1	150	300	149.9929	149.9899	+ 0.0030	0.44997870	- 0.0007
2	300	150	149.9929	149.9905	+ 0.0024	0.35998296	- 0.0013
3	150	600	449.9990	449.9916	+ 0.0074	3.32999260	- 0.0004
4	600	150	449.9990	449.9849	+ 0.0141	6.34498590	+ 0.0063
5	150	1800	1649.9959	1649.9600	+ 0.0359	59.23485281	+ 0.0119
6	1800	150	1649.9959	1649.9728	+ 0.0231	38.11490529	- 0.0009
7	300	600	300.0061	300.0003	+ 0.0058	1.74003538	0.0000
8	600	300	300.0061	299.9984	+ 0.0077	2.31004697	+ 0.0019
9	300	1800	1500.0030	1499.9739	+ 0.0291	43.65008730	+ 0.0071
10	1800	300	1500.0030	1499.9906	+ 0.0124	18.60003720	- 0.0096
11	600	1800	1199.9969	1199.9866	+ 0.0103	12.35996807	- 0.0076
12	1800	600	1199.9969	1199.9858	+ 0.0111	13.31996559	- 0.0068

The following results are then computed:

ΣD_A = the sum of the elements in column 4.
= 10499.9876 m.

$(\Sigma D_A)^2$ = the square of the above result
= 110249739.6 m^2.

ΣD_A^2 = the sum of the square of the elements in column 4.
= 13454977.32 m^2.

$\Sigma \Delta$ = the sum of the elements of column 6.
= + 0.1623 m.

$\Sigma D_A \Delta$ = the sum of the products of the elements in column 4 and column 6 taken on a row-by-row basis (sum of column 7).
= 199.8148389 m^2.

n = the number of observations.
= 12.

*The residuals (V) are computed after solving for S and C. They may be computed by using the above equations or by using the tabular entries in: col 6 - (S x col. 4) - C. As a check on the computation, the sum of the residuals should also be computed; assuming no round-off error, the result should be equal to zero.

Then

$$S = \frac{n\,\Sigma(D_A\,\Delta) - \Sigma D_A\,\Sigma\,\Delta}{n\,\Sigma D_A^{\,2} - (\Sigma D_A)^2}$$

$$= \frac{12(199.8148389\ m^2) - (10499.9876\ m)\ (+\ 0.1623\ m)}{12(13454977.32\ m^2) - 110249739.6\ m^2}$$

$$= \frac{693.63008\ m^2}{51209988.20\ m^2}$$

$$= 1.354482015 \times 10^{-5}$$

$$\simeq\ 0.0000135,$$

and using eq. (10)

$$C = \frac{\Sigma D_A^{\,2}\ \Sigma\Delta\ - \Sigma D_A\ \Sigma(D_A\,\Delta)}{n\,\Sigma D_A^{\,2} - (\Sigma D_A)^2}$$

$$= \frac{(13454977.32\ m^2)\ (0.1623\ m) - (10499.9876\ m)\ (199.8148389\ m^2)}{12(13454977.32\ m^2) - 110249739.6\ m^2}$$

$$= \frac{85689.488\ m^3}{51209988.20\ m^2}$$

$$= 1.673296 \times 10^{-3}\ m$$

$$\simeq\ +\ 0.0017\ m.$$

Using eq. (10a),

$$C = \overline{\Delta} - S\,\overline{D}_A$$

$$= 0.1623/12 - 0.000013545 \times 10499.9876/12$$

$$= 1.673296 \times 10^{-3} \text{ m}$$

$$\simeq 0.0017 \text{ m.}$$

From eq. (13a),

note: $(n \, \Sigma \, D_A \, \Delta - \Sigma \, D_A \, \Sigma \, \Delta)$ is the numerator from eq. (9),

$$\hat{\sigma}_0^2 = \frac{\Sigma \, (\Delta - \bar{\Delta})^2 - \frac{S}{n}\left[n \, \Sigma \, D_A \, \Delta - \Sigma \, D_A \, \Sigma \, \Delta\right]}{(n - 2)}$$

$$= \left[0.001218442500 - \frac{1.354482015 \times 10^{-5}}{12} \times 693.63008\right] \div 10$$

$$= 4.355191077 \times 10^{-5}$$

$$\simeq 0.0000436.$$

From eq. (11),

$$\hat{\sigma}_S = \left[\hat{\sigma}_0^2 \, \frac{n}{n \, \Sigma \, D_A^2 - (\Sigma \, D_A)^2}\right]^{1/2} .$$

Note: The denominator is the same as in eqs. (8) and (9).

$$\hat{\sigma}_S = \left[4.355191077 \times 10^{-5} \, \frac{12}{51209988.20}\right]^{1/2}$$

$$= 3.194602582 \times 10^{-5}$$

$$\simeq 0.0000032.$$

From eq. (12),

$$\hat{\sigma}_C = \left[\hat{\sigma}_0^2 \, \frac{\Sigma \, D_A^2}{n \, \Sigma \, D_A^2 - (\Sigma \, D_A)^2}\right]^{1/2}$$

$$= \left[4.355191077 \times 10^{-5} \times \frac{13454977.32}{51209988.20} \right]^{1/2}$$

$$= 3.382732845 \times 10^{-3}$$

$$\simeq \quad 0.0034 \text{ m.}$$

From eqs. (14) and (15),

$$t_S = \frac{S}{\hat{\sigma}_S}$$

$$= \frac{1.354482015 \times 10^{-5}}{3.194602582 \times 10^{-6}}$$

$$= 4.240$$

$$t_C = \frac{C}{\hat{\sigma}_C}$$

$$= \frac{1.673296 \times 10^{-3}}{3.382732845 \times 10^{-3}}$$

$$= 0.495.$$

Following step 6 of the procedure for analyzing the data, we can now decide the validity of the results. From figure 2, with d.f. = 10, the critical value of t is 3.169. It can be seen that t_S is greater than $t_{0.01,10}$. Therefore, at the 1% significance level, it is possible to reject the hypothesis that S is statistically equal to 0. However, for reasons mentioned previously, a retesting should be performed.

However, t_C is less than $t_{0.01,10}$. Therefore, we cannot reject the hypothesis that C is equal to 0 at the 1% significance level.

Note: If the same sequence and number (n) of observations are performed for each test, then ΣD_A, $(\Sigma D_A)^2$, $\Sigma D_A{}^2$, and n will be constants for a particular base line. The values $\Sigma \Delta$ and $\Sigma D_A \Delta$ only need be computed for each calibration test.

Assume that instead of observing 12 observations, only station 150 or station 1800 was occupied. Both situations are given below. The observations are taken from the previous example.

<u>Example #2.</u>

Station 150

Obs.	From	To	D_A (m)	D_H (m)	Δ (m)	$D_A\ \Delta$ (m^2)	V (m)
1	150	300	149.9929	149.9899	0.0030	0.44997870	0.0010
2	150	600	449.9990	449.9916	0.0074	3.32999260	- 0.0013
3	150	1800	1649.9959	1649.9600	0.0359	59.23485281	0.0003

$\Sigma\ D_A = 2249.9878$ m $\qquad \Sigma\ \Delta = +0.0463 \qquad \Sigma\ D_A\ \Delta = 63.01482411 \qquad \Sigma\ V = 0$

$(\Sigma D_A)^2 = 5062445.1\ m^2 \qquad \Sigma(\Delta - \bar{\Delta})^2 = 6.380066667 \times 10^{-4}$

$\Sigma\ D_A^2 = 2947483.44\ m^2$

As above

$$n\ \Sigma\ D_A\ \Delta - \Sigma\ D_A\ \Sigma\ \Delta = 84.87003720$$

$$\Sigma\ D_A^2\ \Sigma\ \Delta - \Sigma\ D_A\ \Sigma\ D_A\ \Delta = -5.3141022 \times 10^3$$

$$n\ \Sigma\ D_A^2 - (\Sigma\ D_A)^2 = 3.780005220 \times 10^6$$

$S = 2.245235979 \times 10^{-5}$

$\simeq 0.0000225$

$\hat{\sigma}_0^2 = 2.829129700 \times 10^{-6}$

$\hat{\sigma}_S = 1.498445171 \times 10^{-6}$

$\simeq 0.0000015$

$t_S = 14.984$

$C = -1.405845201 \times 10^{-}$

$\simeq -0.0014$ m

$\hat{\sigma}_C = 4.184181198 \times 10^{-3}$

$\simeq 0.0042$ m

$t_C = -0.336$

From table 1, $t_{0.01,1} = 63.657$. Therefore, statistically we cannot reject the hypothesis that both S and C are zero

Example #3.

Station 1800

Obs.	From	To	D_A (m)	D_H (m)	Δ (m)	$D_A\ \Delta$ (m^2)	V (m)
1	1800	600	1199.9969	1199.9858	+ 0.0111	13.31996559	- 0.0021
2	1800	300	1500.0030	1499.9739	+ 0.0291	43.65008730	+ 0.0065
3	1800	150	1649.9959	1649.9728	+ 0.0231	38.11490529	- 0.0043

$\Sigma D_A = 4349.9958$ m $\Sigma \Delta = 0.0633$ m $\Sigma D_A \Delta = 95.0849518\ m^2$ $\Sigma V = 0$

$(\Sigma D_A)^2 = 18922463.5\ m^2$ $\Sigma(\Delta-\bar{\Delta})^2 = 1.68 \times 10^{-4}$

$\Sigma D_A{}^2 = 6412488.0\ m^2$

As given previously,

$$n \Sigma D_A \Delta - \Sigma D_A \Sigma\Delta = 9.900140400$$

$$\Sigma D_A{}^2 \Sigma\Delta - \Sigma D_A \Sigma D_A \Delta = -7.708678300 \times 10^3$$

$$n \Sigma D_A{}^2 - (\Sigma D_A)^2 = 3.15005 \times 10^5$$

$S = 3.142851828 \times 10^{-5}$

$\simeq 0.0000314$

$\hat{\sigma}_0^2 = 6.42844188 \times 10^{-5}$

$\hat{\sigma}_S = 2.47431372 \times 10^{-5}$

$\simeq 0.0000247$

$t_S = 1.270$

$C = -2.447160616 \times 10^{-2}$

$\simeq -0.0024$ m

$\hat{\sigma}_C = 3.617490672 \times 10^{-2}$

$\simeq 0.0362$ m

$t_C = 0.676$

As in table 1, $t_{0.01,1} = 63.657$. Again, on the basis of statistics we cannot reject the hypothesis that both S and C are zero.

EDMI Calibration Base-Line Locations in New Jersey

In 1978, the following calibration base-line locations were selected:

(1) In Andover Township Areoflex Airport, Sussex County sponsored by Robinson Aerial Survey, Inc.

(2) Parsippany-Troy Township, Morris County, sponsored by the Engineers and Surveyors' Association, Inc. Bergen-Passaic Unit.

(3) Skillman Training Center in Montgomery, Somerset County Sponsored by the County of Somerset.

(4) Mercer County Community College in West Windsor, Mercer County sponsored by the Professional Land Surveyors Association of New Jersey.

(5) Burlington Airpark in Lumberton, Burlington County sponsored by the Surveyor's Association of West Jersey.

(6) Millville City Airport in Millville, Cumberland County sponsored also by the Surveyor's Association of New Jersey.

(7) Middlesex County College, Edison, sponsored by the New Jersey Society of Professional Engineers--Land Surveyors Practice Section.

The location of the calibration base-lines is shown on figure 1.

Description of each calibration base-line of New Jersey is available upon request at the Bureau of Geology and Topography--N.J. Geodetic Control Survey, P.O. Box 1390, Trenton, N.J. 08625.

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APPENDIX I. THE GEOMETRICAL TRANSFORMATION OF ELECTRONICALLY MEASURED DISTANCES

Notation:

α = Mean azimuth of line (clockwise from south).

ϕ = Mean latitude of line.

H_i = Elevation of station above mean sea level.

ΔH_i = Height of instrument (or reflector) above mark.

N_i = Geoidal undulation.

k = Index of refraction (for lightwave instruments) $k \simeq 0.18$, for microwave instruments $k \simeq 0.25$).

D_0 = Observed slope distance corrected for ambient atmospheric conditions and mode of measurements (e.g., eccentricities, instrument constant, mirror (or reflector) constant, etc.).

D_1 = $D_0 \pm$ the correction for second velocity (see Höpcke, W., "On the curvature of electromagnetic waves and its effect on measurement of distance," Survey Review, No. 141, pp. 298-312, July 1966). (See eq. (I-7) on page 27.)

D_2 = Chord distance at instrument elevations.

D_3 = Chord distance at station elevation (mark-to-mark).

D_4 = Geoidal or sea level distance.

D_5 = Chord distance at the sea level surface.

D_6 = Ellipsoidal or geodetic distance.

D_7 = Chord distance at the ellipsoidal surface.

D_H = Horizontal chord distance at mean elevation of instruments.

a = Semi-major axis = 6378206.4, Clarke Spheroid 1866.

b = Semi-minor axis = 6356583.8, Clarke Spheroid 1866.

Classically, observed distances have been reduced to one of two surfaces, either the geoid (sea level) or the ellipsoid. To which surface the distances were reduced depended on available information. Generally, in the United States distances were reduced to the geoid. However, with the acquisition of more accurate information on geoidal undulations, the present trend is to reduce the distances to the ellipsoid.

With the introduction of satellite positioning systems, very long base line interferometry (VLBI) or for special purpose surveys, the term "reduction" will no longer suffice. Therefore, we should think in terms of the transformation of distances.

Generally, this transformation can be divided into two procedures:

1. The transformation of the distance along an arc to a chord distance or its inverse.

2. The transformation of a chord distance at one altitude to a chord distance at another altitude.

The general equations for these transformations are:

Chord Distance to Chord Distance:

$$D_1{}^2 = \frac{D^2 - \left(H_2 - H_1\right)^2}{\left(1 + \frac{H_1}{R}\right)\left(1 + \frac{H_2}{R}\right)} \left(1 + \frac{H_1'}{R}\right)\left(1 + \frac{H_2'}{R}\right) + \left(H_2' - H_1'\right)^2 \qquad \text{(I-1)}$$

where D is the spatial chord distance at elevations H_1 and H_2, and D_1 is the desired spatial chord distance at elevations H_1' and H_2', and R is the radius of curvature.

Arc to Chord:

$$D_1 = 2R \sin \frac{D}{2R} \,. \qquad \text{(I-2)}$$

Here

D_1 is the desired chord distance, and
D is the distance along an arc.

Normally eq. (I-2) is a small correction that amounts to a change in distance of 1.5 mm for a line 10,000 m in length.

The following specific equations for various geometric distances were derived from the above two equations. (See figure I-1 for a graphic representation of the geometric relationships.)

Equations for the transformation of electronically measured distances:

Equations for the transformation of electronically measured distances:

$$e'^2 = \frac{a^2 - b^2}{b^2} \qquad \text{(I-3)}$$

$$c = \frac{a^2}{b} \qquad \text{(I-4)}$$

$$N = \frac{c}{(1 + e'^2 \cos^2\phi)^{1/2}} \qquad \text{(I-5)}$$

$$R = \frac{N}{1 + e'^2 \cos^2\phi \cos^2\alpha} \qquad \text{(I-6)}$$

$$D_1 = D_0 - (k - k^2)\ D_0^3/12\ R^2 \qquad \text{(I-7)}$$

$$R' = \frac{R}{k} \qquad \text{(I-8)}$$

$$D_2 = 2\ R'\ \mathrm{Sin}\left(D_1/2\ R'\ \frac{180}{\pi}\right) \qquad \text{(I-9)}$$

$$H_1' = H_1 + \Delta H_1 \qquad \text{(I-10)}$$

$$H_2' = H_2 + \Delta H_2 \qquad \text{(I-11)}$$

$$\Delta H = H_1' - H_2' \qquad \text{(I-12)}$$

$$D_5 = \left[(D_2^2 - \Delta H^2)/\left\{(1 + H_1'/R)(1 + H_2'/R)\right\}\right]^{1/2} \qquad \text{(I-13)}$$

$$D_4 = 2\ R\left[\mathrm{Sin}^{-1}\ (D_5/2R)\right]\frac{\pi}{180} \qquad \text{(I-14)}$$

$$D_3 = \left[D_5^2\ (1 + H_1/R)(1 + H_2/R) + (H_1 - H_2)^2\right]^{1/2} \qquad \text{(I-15)}$$

$$h_1 = H_1' + N_1 \qquad \text{(I-16)}$$

$$h_2 = H_2' + N_2 \qquad \text{(I-17)}$$

$$\Delta h = h_1 - h_2 \qquad \text{(I-18)}$$

$$D_7 = \left[(D_2{}^2 - \Delta h^2)/\left\{(1+h_1/R)(1+h_2/R)\right\}\right]^{1/2} \qquad \text{(I-19)}$$

$$D_6 = 2R\left[\mathrm{Sin}^{-1}(D_7/2R)\right]\frac{\pi}{180} \qquad \text{(I-20)}$$

$$H_m = (H_1 + H_2)/2 \qquad \text{(I-21)}$$

$$D_H = \left[\left\{(D_3^2 - \Delta H^2)(1 + H_m/R)^2\right\}/\left\{(1 + H_1/R)(1 + H_2/R)\right\}\right]^{1/2} \approx (D_3^2 - \Delta H^2)^{1/2} \qquad \text{(I-22)}$$

Note: In eqs. (I-9), (I-14), and (I-20) the terms $\pi/180$ or $180/\pi$ were added to convert from angular measure to radian measure (or vice versa).

APPENDIX II. THE INFLUENCE OF METEOROLOGICAL DATA ON THE ACCURACY OF ELECTRONICALLY MEASURED DISTANCES

The determination of the refractive index of the ambient atmosphere has a critical influence on the accuracy of distances measured with EDMI. These effects can be evaluated by varying the parameters in the equations for n_a (refractave index) and computing their influence. Alternately, their influence may be computed by evaluating the partial derivatives of the refractive index equation at nominal values. The partial derivatives of the refractive index equation for microwave and lightwave sources are discussed below.

Microwave Source EDMI

From eq. (6) (see page 8) we have

$$(n_a-1) \times 10^6 = \frac{103.46p}{273.2+t} + \frac{490,814.24e}{(273.2+t)^2}$$

where

$$e = e' + 6$$

$$e' \simeq 4.58 \times 10^a$$

$$a = 7.5t'/(237.3+t)$$

$$de = -0.000660\ (1 + 0.00115t')\ P\ (t-t).$$

Then, letting

$$n = (n_a - 1) \times 10^6$$

the partial derivatives with respect to t, t', and p are:

$$\frac{\partial n}{\partial p} \simeq \frac{103.46}{273.2+t} - \frac{323.94}{(273.2+t)^2}\ (1 + 0.00115t')\ (t-t') \qquad \text{(II-1)}$$

$$\frac{\partial n}{\partial t} \simeq \frac{-103.46p}{(273.2+t)^2} - \frac{981628.48e}{(273.2+t)^3} - \frac{323.94}{(273.2+t)^2}\ (1 + 0.00115t')\ p \qquad \text{(II-2)}$$

$$\frac{\partial n}{\partial t'} \simeq \frac{490814.24}{(273.2+t)^2}\left[\frac{4098.026e'}{237.3+t')^2} + \right. \qquad \text{(II-3)}$$

$$\left. 0.00066p\ (1 + 0.00230t' - 0.00115t)\right].$$

The above derivatives when evaluated yield results in units of ppm, when t and t' are in degrees Celsius and pressures are in mm of Hg.

Evaluating eq. (II-1) for $0° C \leq t \leq 30° C$

$$t - t' = 10° C$$

(a) $t = 0°C$: $\frac{\partial n}{\partial p} = 0.34$

(b) $t = 10°C$: $\frac{\partial n}{\partial p} = 0.33$

(c) $t = 20°C$: $\frac{\partial n}{\partial p} = 0.31$

(d) $t = 30°C$: $\frac{\partial n}{\partial p} = 0.30$

If we assume the error of observing pressure is approximately 3 mm (0.1 in.) of Hg, then for a mean value of $\partial n/\partial p$ equal to 0.32, the error introduced into the computation of refraction and, thus, the distance is:

$$\Delta n = 0.32 \Delta p$$

$$\Delta n = 0.32 (3) = 1.0 \text{ ppm.}$$

Evaluating eq. (II-2) for $0° C \leq t \leq 30° C$

$$t' = t$$

$$p = 760 \text{ mm of Hg}$$

and e' given by the following:

t' (°C)	=	0°	10°	20°	30°
e' (mm of H_g)	=	4.58	9.20	17.53	31.81

(a) $t = 0°C$: $\frac{\partial n}{\partial t} = -4.57$

(b) $t = 10°C$: $\frac{\partial n}{\partial t} = -4.52$

(c) $t = 20°C$: $\frac{\partial n}{\partial t} = -4.52$

(d) $t = 30°C$: $\frac{\partial n}{\partial t} = -4.75$.

Assuming an error in observing dry bulb temperatures on the order of 0.5° C and using the mean value from above, the effect on the refractive index is:

$$\Delta n = -4.58\,\Delta t$$

$$\Delta n = (-4.58)(0.5)$$

$$\Delta n = -2.3 \text{ ppm.}$$

Evaluating eq. (II-3) for $0°\ C \le t \le 30°\ C$

$$t' = t$$

$$p = 760 \text{ mm of } H_g$$

e' as above

(a) $t = 0°$ C: $\frac{\partial n}{\partial t'} = 5.49$

(b) $t = 10°$ C: $\frac{\partial n}{\partial t'} = 6.92$

(c) $t = 20°$ C: $\frac{\partial n}{\partial t'} = 9.08$

(d) $t = 30°$ C: $\frac{\partial n}{\partial t'} = 12.51.$

Again one can assume an error in determinations of the wet bulb temperature to be approximately 0.5° C. However, a mean of the above values would not be very indicative. Therefore, the range of the effect will be given.

For 0° C: $\Delta n = 5.49(0.5) = 2.74$

For 30° C: $\Delta n = 12.51(0.5) = 6.26$

or for $0°\ C \le t' \le 30°\ C$

$$2.7 \text{ ppm} \le \partial n \le 6.2 \text{ ppm.}$$

It should be noted that previously some authors have stated that a change of 1° C in t produces a change of 1 ppm in the distances. From the evaluation of eq. (II-2) above, the effect is approximately 5 ppm. Perhaps the confusion arises because of a failure to evaluate the third term in this equation or because of an alternate approach to these differentials. If the partial derivatives are taken with respect to p, t, e (instead of p, t, t'), consider the following:

$$\frac{\partial n}{\partial p} = \frac{103.46}{273.2+t} \tag{II-4}$$

$$\frac{\partial n}{\partial t} = \frac{-103.46p}{(273.2+t)^2} - \frac{981628.48e}{(273.2+t)^3} \tag{II-5}$$

$$\frac{\partial n}{\partial e} = \frac{490814.24}{(273.2+t)^2} \ . \tag{II-6}$$

Comparing eqs. (II-1) and (II-2), the difference is the second term of (II-1). This term evaluated for nominal values contributes less than 0.1 ppm and thus has no real effect.

Evaluating eq. (II-5) for values as in eq. (II-2) we have:

(a) $t = 0^\circ$ C: $\frac{\partial n}{\partial t} = -1.27$

(b) $t = 10^\circ$ C: $\frac{\partial n}{\partial t} = -1.38$

(c) $t = 20^\circ$ C: $\frac{\partial n}{\partial t} = -1.59$

(d) $t = 30^\circ$ C: $\frac{\partial n}{\partial t} = -1.98$.

However, e (the vapor pressure) is determined from observations of t, t', and p. From

$$e = e' + de$$

$$e' = 4.58 \times 10^a$$

$$a = (7.5t')/(237.3+t')$$

$$de = -0.000660\ (1 + 0.00115t')\ p\ (t-t')\ ,$$

the following partials are determined:

$$\frac{\partial e}{\partial p} = -0.000660\ (1 + 0.00115t')\ (t-t') \tag{II-7}$$

$$\frac{\partial e}{\partial t} = -0.000660\ (1 + 0.00115t')\ p \tag{II-8}$$

$$\frac{\partial e}{\partial t'} = \frac{4098.764e'}{(237.3+t')^2} + 0.00066p\ (1 + 0.00230t' - 0.00115t). \tag{II-9}$$

Combining with eq. (II-6) and evaluating eqs. (II-8) and (II-9) for $0^{o}C \leq t \leq 30^{o}C$

$$t' = t$$

$$p = 760 \text{ mm of Hg}$$

Then

$$t = 0° \text{ C}: \quad \Delta n = -3.29\Delta t + 5.46\Delta t'$$

$$t = 10° \text{ C}: \quad \Delta n = -3.12\Delta t + 6.89\Delta t'$$

$$t = 20° \text{ C}: \quad \Delta n = -2.91\Delta t + 9.14\Delta t'$$

$$t = 30° \text{ C}: \quad \Delta n = -2.78\Delta t + 12.50\Delta t'.$$

From eq. (II-5) the impression is given that the effect of $1^{o}C$ change in dry bulb is in the magnitude of 1 ppm. However, when combined with the above, the results are similar to those obtained using eqs. (II-1) through (II-3).

Lightwave source EDMI

From eq. (4) (see page 7) we have

$$(n_a - 1) \times 10^6 = \left[\frac{n_g - 1}{1+\alpha t} \times \frac{p}{760} - \frac{5.5e10^{-8}}{(1+\alpha t)^2}\right] \times 10^6$$

Again, letting

$$n = (n_a - 1) \times 10^6$$

the partial derivatives with respect to p, t, and t' are:

$$\frac{\partial n}{\partial p} = \frac{(n_g - 1)}{(1+\alpha t)\cdot 760} \times 10^6 + \frac{0.0000363}{(1+\alpha t)^3}(1 + 0.00115t')\ (t-t') \tag{II-10}$$

$$\frac{\partial n}{\partial t} = \frac{-\alpha(n_g - 1)p \times 10^6}{(1+\alpha t)^2\ 760} + \frac{0.11\ e\alpha}{(1+\alpha t)^3} + \tag{II-11}$$

$$\frac{0.0000363\ (1 + 0.00115t')\cdot p}{(1+\alpha t)^2}$$

$$\frac{\partial n}{\partial t'} = \frac{-0.055}{(1+\alpha t)2}\left[\frac{4098.764}{(237.3+t')2} + \right. \tag{II-12}$$

$$\left. 0.00066p\ (1 + 0.0023t' - 0.00115t)\right].$$

Remembering

$$(n_g-1) \times 10^6 = \left[2876.04 + \frac{48.864}{\lambda^2} + \frac{0.680}{\lambda^4}\right] \times 10^{-4}$$

then for $\lambda = 0.6328$ µm

$$(n_g-1) \times 10^6 = 300.2308$$

and for $\lambda = 0.9300$ µm

$$(n_g-1) \times 10^6 = 293.3446.$$

Evaluating eq. (II-10) for 0° C $\leq t \leq 30^\circ$ C

$$t-t' = 10° \text{ C}$$

$$\text{and } \lambda = 0.6328\ \mu\text{m}.$$

we have

(a) $t = 0°$ C: $\frac{\partial n}{\partial p} = 0.40$

(b) $t = 10°$ C: $\frac{\partial n}{\partial p} = 0.38$

(c) $t = 20°$ C: $\frac{\partial n}{\partial p} = 0.37$

(d) $t = 30°$ C: $\frac{\partial n}{\partial p} = 0.36$.

For $\lambda = 0.9300$ µm

(a) $t = 0°$ C: $\frac{\partial n}{\partial p} = 0.39$

(b) $t = 10°$ C: $\frac{\partial n}{\partial p} = 0.37$

(c) $t = 20°$ C: $\frac{\partial n}{\partial p} = 0.36$

(d) $t = 30°$ C: $\frac{\partial n}{\partial p} = 0.35$.

Using the mean value of n/ p equal to 0.37 and an error of 3 mm (0.] in) of Hg, the error introduced into the refractive index is:

$$\Delta n = (0.37)(3)$$
$$= 1.1 \text{ ppm}.$$

Evaluating eq. (II-11) for 0° C $\leq t \leq 30^\circ$ C

$t' = t$
$p = 760$ mm of Hg
$\lambda = 0.6328$ µm
e' (see values on page 30).

then

(a) $t = 0° C$: $\frac{\partial n}{\partial t} = -1.07$

(b) $t = 10° C$: $\frac{\partial n}{\partial t} = -1.00$

(c) $t = 20° C$: $\frac{\partial n}{\partial t} = -0.93$

(d) $t = 30° C$: $\frac{\partial n}{\partial t} = -0.86$.

For $\lambda = 0.9300$ μm:

(e) $t = 0° C$: $\frac{\partial n}{\partial t} = -1.04$

(f) $t = 10° C$: $\frac{\partial n}{\partial t} = -0.97$

(g) $t = 20° C$: $\frac{\partial n}{\partial t} = -0.90$

(h) $t = 30° C$: $\frac{\partial n}{\partial t} = -0.84$.

The mean from above is 0.95. Using an error in t of 0.5^{o} C, then the effect on the refractive index is:

$$\Delta n = (0.95)(0.5)$$
$$= 0.5 \text{ ppm} .$$

Evaluating eq. (II-12) for $0^{o}\ C \leq t \leq 30^{o}\ C$

$$t' = t$$

$$p = 760 \text{ mm of Hg}$$

$$\lambda = 0.6328\ \mu m \text{ and } 0.9300\ \mu m$$

(a) $t = 0°$ C: $\frac{\partial n}{\partial t'} = -0.05$

(b) $t = 10°$ C: $\frac{\partial n}{\partial t'} = -0.06$

(c) $t = 20°$ C: $\frac{\partial n}{\partial t'} = -0.08$

(d) $t = 30°$ C: $\frac{\partial n}{\partial t'} = -0.10$.

From the above, it can be seen that the effect of nominal errors in the wet bulb temperature on the determination of refractive index is minimal.

In addition to errors in temperature and pressure, the refractive index of light is affected by errors in the assigned angstrom rating of the light source. From eqs. (II-3) and (II-4),

$$\frac{\partial n}{\partial \lambda} = \frac{-9.7728}{\lambda^3} - \frac{0.272}{\lambda^5} \cdot \frac{p}{(1+\alpha t)(760)} \quad . \qquad \text{(II-13)}$$

Evaluating eq. (II-13) for $0° \text{ C} \leq t \leq 30° \text{ C}$

$p = 760$ mm of Hg .

For $\lambda = 0.6328$ μm

(a) $t = 0°$ C: $\frac{\partial n}{\partial \lambda} = -41.25$

(b) $t = 10°$ C: $\frac{\partial n}{\partial \lambda} = -39.79$

(c) $t = 20°$ C: $\frac{\partial n}{\partial \lambda} = -38.43$

(d) $t = 30°$ C: $\frac{\partial n}{\partial \lambda} = -37.17$.

For $\lambda = 0.9300$ μm

(e) $t = 0°$ C: $\frac{\partial n}{\partial \lambda} = -12.54$

(f) $t = 10°$ C: $\frac{\partial n}{\partial \lambda} = -12.10$

(g) $t = 20°$ C: $\frac{\partial n}{\partial \lambda} = -11.68$

(h) $t = 30°$ C: $\frac{\partial n}{\partial \lambda} = -11.30$.

An error of 0.01 μm in λ introduces a change in the refractive index of 0.4 ppm for instruments having a light source in the range of 0.6328 μm and 0.1 ppm for instruments having a light source in the range of 0.9300 μm.

For instruments using a red laser light, the light source wavelengths are around 0.6328 μm. Infrared wavelengths are around 0.9 μm.

APPENDIX III. TABLE OF SELECTED CONVERSION FACTORS

Temperature:

$^{o}C = 5/9\ (^{o}F - 32)$
$^{o}F = 9/5\ ^{o}C + 32$

where

^{o}C = degrees Celsius
^{o}F = degrees Fahrenheit

Pressure:

1 in. of mercury (Hg) = 33.86389 mb = 0.3386389 kPa
1 mm of Hg = 1.333224 mb = 0.0133224 kPa
1 in. of Hg = 33.86389 mb
1 mb = 0.02952998 in of Hg
1 mb = 0.7500616 mm of Hg
1 in. of Hg = 25.4 mm of Hg

Pressure in mm of Hg = $25.4 \times e^{a}$

where

$a = 3.3978 - \text{Alt}\ (3.6792 \times 10^{-5})$

and

Alt = altimeter reading in feet

e = base of natural logarithm

= 2.718281828 . . .

Note: If, as in some altimeters, zero feet does not equal sea level, then the altimeter reading will have to be modified accordingly.

Length:

1 m = 39.37 in
1 m = 3.28083333 ft
1 ft = 0.30480061 m
1 in. = 25.400051 mm

SETTLEMENT CONTROL SURVEY REPORT
AND
METHOD OF MEASUREMENT
FOR
THE VERRAZANO NARROWS BRIDGE

by

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I. AUTHORIZATION

Pursuant to the authorization of the Triborough Bridge and Tunnel Authority, we performed a settlement control survey on the Verrazano Narrows Bridge, anchorages and the towers to determine the possible settlement in comparison to a similar settlement control survey performed in 1970.

II. SCOPE OF WORK

The control project involved the recovery of the control bench marks and the settlement control points set in 1970 and establishment of level runs between the control bench marks to determine any possible movement within the control bench mark network.

Taking of settlement readings on the existing and newly established settlement control points on the anchorages and the towers.

Balancing of the loops, established by differential leveling, with the condition equation method of least squares, computation of elevations of each settlement control point to ten thousandths of a foot accuracy and determining the amount of movement of each settlement control point.

ALL ELEVATIONS REFER TO THE U.S. COAST AND GEODETIC SEA LEVEL DATUM OF 1929

III. VERTICAL CONTROLS

A. Staten Island Side - The following control bench marks were recovered, and accepted for control measurements.

DESIGNATION	COMPUTER NO.	ELEVATION
US D-344	(2)	103.7760
TIDAL - 5 STA. I-36A	(14)	12.2324
T.B.T.A. - 4	(12)	30.0169
T.B.T.A. - 5	(3)	47.7957
T.B.T.A. - 6	(13)	43.5211

T.B.T.A. - 7 (1), T.B.T.A. - 8 (5), T.B.T.A. - 9 (4), indicated slight movements, so these control points were included in the level loops, new elevations computed, but not used to control settlement readings.

The following settlement points were recovered and used in settlement control measurements:

DESIGNATION ANCHORAGES	COMPUTER NO.
6-S1	(10)
8-S1	(7)
12-S1	(9)
14-S1	(8)
15-S1	(6)
17-S1	(11)

TOWER

A-1	(28)
G-1	(17)
ST-1	(18)
ST-2	(19)
ST-3	(26)
ST-4	(27)
ST-5	(20)
ST-6	(22)
ST-7	(23)
ST-8	(25)
A-12	(24)
G-12	(21)

The following new settlement points were established:

DESIGNATION	COMPUTER NO.	LOCATION
22-S1	(54)	Anchorage
24-S1	(55)	Anchorage

B. Brooklyn Side - The following control bench marks were recovered and accepted for control measurements:

DESIGNATION	COMPUTER NO.	ELEVATION
US-183	(43)	12.6000
US-J-342	(45)	27.1920
T.B.T.A. - 3	(46)	42.7997

T.B.T.A. - 1, brass plug set in 1970 in the base of a road sign was destroyed by removal of the sign.

T.B.T.A. - 2, brass plug set in 1970, in the concrete base of a sign near the southeasterly corner of the anchorage, in the center mall area, between the entrance and exit lanes to the bridge from Belt Parkway, indicates a significant and unexplainable movement. It was included in the level loop and a new elevation was computed.

The following settlement points were recovered and used in settlement control measurements:

DESIGNATION	COMPUTER NO.
ANCHORAGE	
2-B	(41)
8-B	(48)
9-B	(50)
12-B	(49)
29-B	(51)
31-B	(52)

DESIGNATION	COMPUTER NO.
TOWER	
A-1	(35)
G-1	(34)
BT-1	(33)
BT-2	(32)
BT-3	(37)
BT-4	(36)
BT-5	(31)
BT-6	(30)
BT-7	(39)
BT-8	(38)
A-12	(40)
G-12	(29)

The following new control bench marks were established to replace T.B.T.A. - 1.

T.B.T.A. - 1A	(55)
T.B.T.A. - 1B	(56)
USCE	(57) set by others included in level loops

The following new settlement point was established on the south side of the anchorage to replace 8-B (48) which was damaged.

8-C	(58) brass plug

IV. PROCEDURE

During the course of all leveling work, heretofore described, a WILD - N3 (No. N3-341020) precision level, with parallel plate micrometer was used, utilizing the WILD GPL invar leveling staves with supporting poles.

All turnpoints were stakes with tack or portable heavy cast iron turn points (WILD).

Initially, all level routes were measured, instrument and turn point station laid out according to first order standards, so the length of backsight and foresight were equal and did not exceed 150 ft.

Except where the level line was carried over water surface between anchorages and towers, the method of reciprocal leveling was applied.

All measurements were double readings on both sides of the invar leveling staves. Using the means of the actual readings a difference of elevation was computed for each link between two bench marks.

All reciprocal readings were corrected for refraction and curvature.

Measurements were only taken in early morning hours with no wind more than 5 miles per hour, to avoid refraction and vibration. The level was protected with an umbrella against deformation caused by the heat of the sun.

V. CONTROL NETWORKS

Before the actual settlement measurements took place, a level line was made between the ground control points to determine any possible movements, and the reliability of the U.S.C. & G.S. and T.B.T.A. control bench marks.

A. STATEN ISLAND SIDE - A direct line was made between U.S. Tidal 5, T.B.T.A. - 6, T.B.T.A. - 4, T.B.T.A. - 5, US D-344, T.B.T.A. - 7, T.B.T.A. - 8, T.B.T.A. - 9, closing back to US TIDAL 5. Comparing the relative elevation differences between each bench mark with the measurement done in 1970, it clearly indicated that US TIDAL 5, T.B.T.A. 6, T.B.T.A. - 5, T.B.T.A. - 4, and US D-344 did not move and we can accept them as reliable controls, holding the same elevations as used in 1970. T.B.T.A. - 7, T.B.T.A. - 8, T.B.T.A. - 9, did show some movements so they were not used as control points, but were included in the level loops and new elevations computed.

Eight adjoining loops were formed through the control and settlement points, closed, balanced and elevations computed.

The maximum error of closure was 0.00244 in loop 4. The largest correction applied was 0.00130. All other corrections were in the 0.00001 to 0.00069 range.

B. BROOKLYN SIDE - A level line made through ground control points T.B.T.A. - 3, US-J-342, TIDAL - 5,183, T.B.T.A. - 2, clearly indicated that if compared to elevation differences measured in 1970, T.B.T.A. - 3, US J-342, and 183 did not move, so the elevations held in 1970 are still reliable values. U.S. TIDAL - 5 moved .0075 ft. higher, this bench mark set in a large base stone of the U.S. Army pier could have been moved by the force of a storm. The movement of T.B.T.A. - 2, a brass plug in the concrete base of a sign is still unexplainable. It moved 0.7706 ft. higher. By visual investigation it seems to be tilted somewhat to the east. This control bench mark was included at this time in the same loop as it was in 1970. At both times the loop error of closure was well within limit. A new elevation was computed.

Ten adjoining loops were formed through the control and settlement points, loops closed, balanced and elevations computed.

The maximum error of closure was 0.01972 in loop #4. Loop #4 includes the measurements across the water from the anchorage to the tower. Each line of sight was separately measured by E.D.M. Beetle 500. One line of sight was from 31-B anchorage to BT-7 tower 1,253 ft. The other line of sight was from 2-B anchorage to BT-4 tower, 1,241 ft. long.

In both cases the method of reciprocal leveling was applied. The foresight was made on a target, which is adjusted manually on the rod, to the horizontal line of sight, and the reading is taken on the target so the accuracy of the readings is limited compared to the invar rod and the parallel plate micrometer. The invar rod can not be used at such distance, due to the difficulty in distinguishing the graduations.

The largest corrections were 0.01139 between US J-342 and USCE on a distance of 4,374 ft. between 31-B and BT-7 0.00945 on 1,253 ft. and between 2-B and BT-4 0.00936 on 1,241 ft., all other corrections were between 0.00003 and 0.00139 range.

VI. COMPUTED ELEVATIONS AND SETTLEMENT

The summary of loop closures before balancing were shown on computer output sheets, (see that data on pages I-7 (Staten Island side) and I-14 (Brooklyn side) respectively).

Pages I-8 and I-9 (Staten Island) and I-15 and I-16 (Brooklyn) show the actual elevation differences as measured, the corrections applied and the corrected elevation differences.

Pages I-10 and I-11 (Staten Island) and I-17 and I-18 (Brooklyn) are the tabulations of the computed elevations, showing each bench mark with the computer number, the new elevations computed, the designation of the bench mark, the elevations measured in 1970 and the actual changes in elevation.

For better visual representation, we have included diagrams for both sides, to show the actual amount of movement of each settlement point on the anchorages and towers. (See pages I-12 and I-13 (Staten Island) and I-19 and I-20 (Brooklyn) for the loop layouts and elevation changes.)

The Staten Island anchorage shows the biggest amount of settlement at the southeasterly corner point #11, -0.0171 ft.: the northeasterly corner point #6, -0.0104 ft.; the northwesterly corner #8, moved upwards +0.0028 ft.; the southwesterly corner #9 can be considered stable with +0.0002 ft.; the northerly center point #7 moved upwards +0.0024 ft.; the southerly center point #10 settled -0.0043 ft.

The Staten Island tower indicates the biggest settlement at the southeasterly corner point #28 set in the granite block, -0.0154 ft., and it varies through the easterly row of control points to -0.0106 at the northwesterly corner point #24. The westerly row of control points vary from -0.0037 for point #21 to -0.0105 at point #18.

On the Brooklyn anchorage each control point shows settlement in the range from -0.0074 to -0.0166. The two control points on the westerly side show -0.0151 (#52) and -0.0166 (#41) as compared to the easterly control points -0.0074 (#50) and -0.0077 (#49).

The Brooklyn tower north base settled almost evenly on all control points from -0.0209 ft. to -0.0244 ft.; the south base from -0.0097 to -0.0168. In comparing the two tower bases the north base indicates somewhat larger settlement than the south base.

As a summary of our report, the smallest amount of movement was measured at control point #9, +0.0002 ft. on the Staten Island anchorage and the largest measured movement was at control point #31, -0.0244 on the Brooklyn tower base plate.

SUMMARY OF LOOP CLOSURES BEFORE BALANCING

LOOP NO.	ERROR OF CLOSURE
1	.00067
2	-.00083
3	-.00195
4	.00244
5	.00036
6	-.00010
7	-.00011
8	.00075

VERAZZANO NARROWS BRIDGE SETTLEMENT REPORT   STATEN ISLAND SIDE   1976

LOW BM.	HIGH BM.	LINK NO.	DIST.	ELEVATION DIFF.	CORRECTION	CORRECTED EL. DIFF.	LOOP NUMBERS +	+	-	-	COMMENTS.
14	13	1	0	31.28872	.00000	31.28872	3	0	0	0	
12	13	2	0	13.50424	.00000	13.50424	1	0	3	0	
12	3	3	0	17.77887	.00000	17.77887	3	0	0	0	
3	2	4	0	55.98027	.00000	55.98027	4	0	0	0	
1	2	5	1912	5.00909	.00130	5.01039	3	0	4	0	
7	53	7	154	.68980	.00005	.68985	3	0	5	0	
6	7	8	126	1.58241	.00004	1.58245	3	0	5	0	
6	5	9	300	4.69160	-.00010	4.69150	5	0	3	0	
5	4	10	241	6.42162	-.00016	6.42146	4	0	3	0	
3	4	11	1032	57.15079	.00069	57.15148	3	0	4	0	
14	2	13	0	91.54360	.00000	91.54360	0	0	3	0	
11	5	15	450	23.24127	-.00015	23.24112	4	0	5	0	
11	10	17	148	12.22162	.00005	12.22167	5	0	4	0	
10	54	18	170	9.53439	.00006	9.53445	5	0	4	0	
54	1	19	42	1.72483	.00003	1.72486	3	0	4	0	
8	9	20	134	.03350	.00030	.03380	2	0	5	0	
17	18	24	24	2.17628	.00000	2.17628	6	0	0	0	
18	19	25	41	.02524	.00005	.02529	6	0	1	0	
19	20	26	89	.03015	.00003	.03018	7	0	1	0	
21	20	27	44	2.07916	.00013	2.07929	1	0	8	0	

VERAZZANO NARROWS BRIDGE SETTLEMENT REPORT STATEN ISLAND SIDE 1976

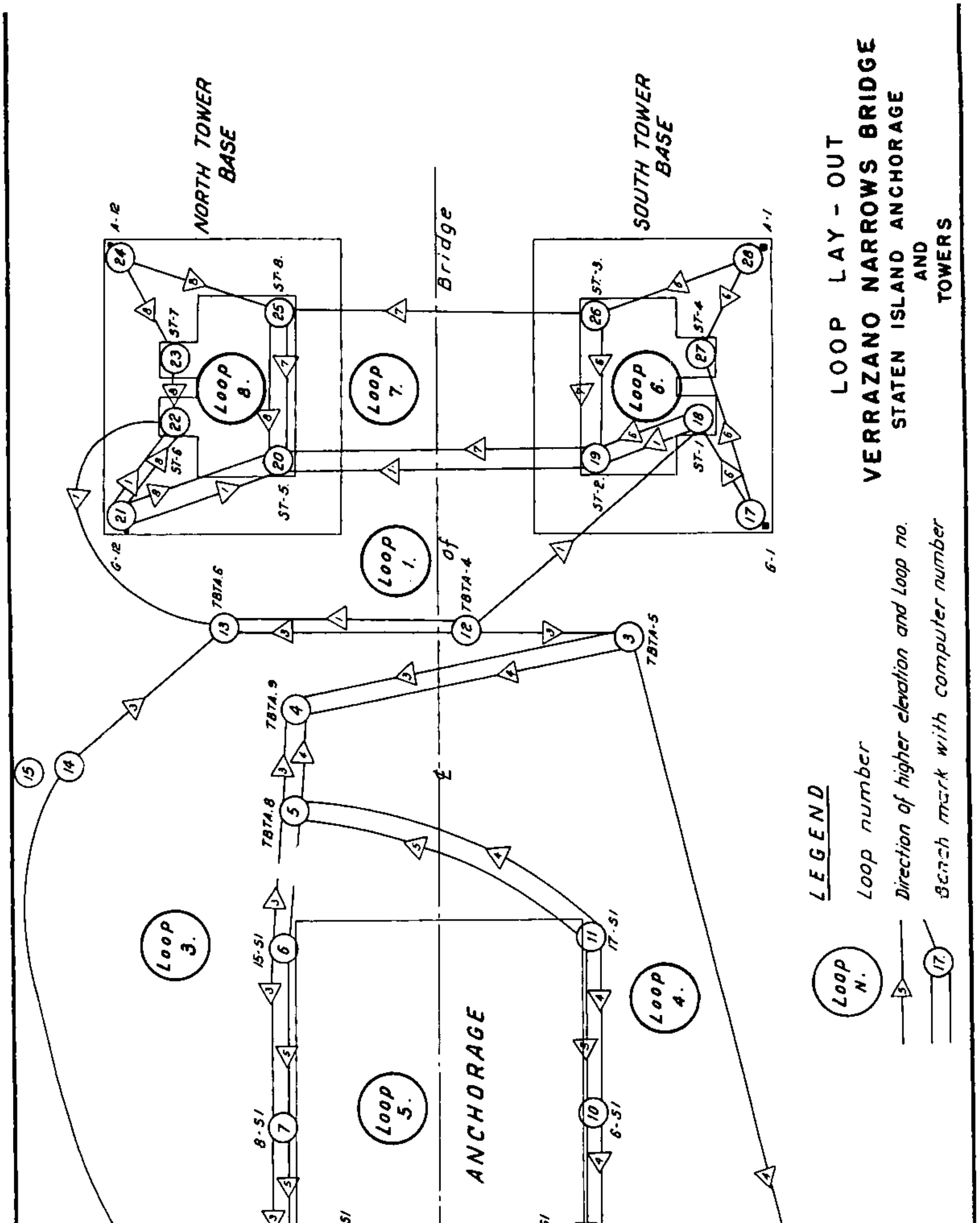
LOW BM.	HIGH BM.	LINK NO.	DIST.	ELEVATION DIFF.	CORRECTION	CORRECTED EL. DIFF.	LOOP NUMBERS +	+	-	-	COMMENTS.
21	22	28	22	2.07975	-.00006	2.07969	8	0	1	0	
23	22	29	29	.03313	.00011	.03324	0	0	8	0	
24	23	30	24	2.24972	.00009	2.24981	0	0	8	0	
24	25	31	49	2.22621	-.00019	2.22602	8	0	0	0	
26	25	32	89	.03332	.00005	.03337	0	0	7	0	
28	26	33	50	2.26823	-.00001	2.26822	0	0	6	0	
28	27	34	24	2.26998	.00000	2.26998	6	0	0	0	
17	27	35	51	2.14351	-.00001	2.14350	0	0	6	0	
22	13	65	376	10.64471	.00037	10.64508	0	0	1	0	
12	18	66	425	2.80288	.00041	2.80329	0	0	1	0	
26	19	67	50	.05986	-.00004	.05982	7	0	6	0	
25	20	68	50	.05680	-.00016	.05664	8	0	7	0	
53	54	69	214	.93460	-.00041	.93419	3	0	2	0	
53	8	70	26	8.10783	.00006	8.10789	2	0	5	0	
54	9	71	24	7.20756	-.00005	7.20751	5	0	2	0	

VERAZZANO NARROWS BRIDGE SETTLEMENT REPORT   STATEN ISLAND SIDE   1976

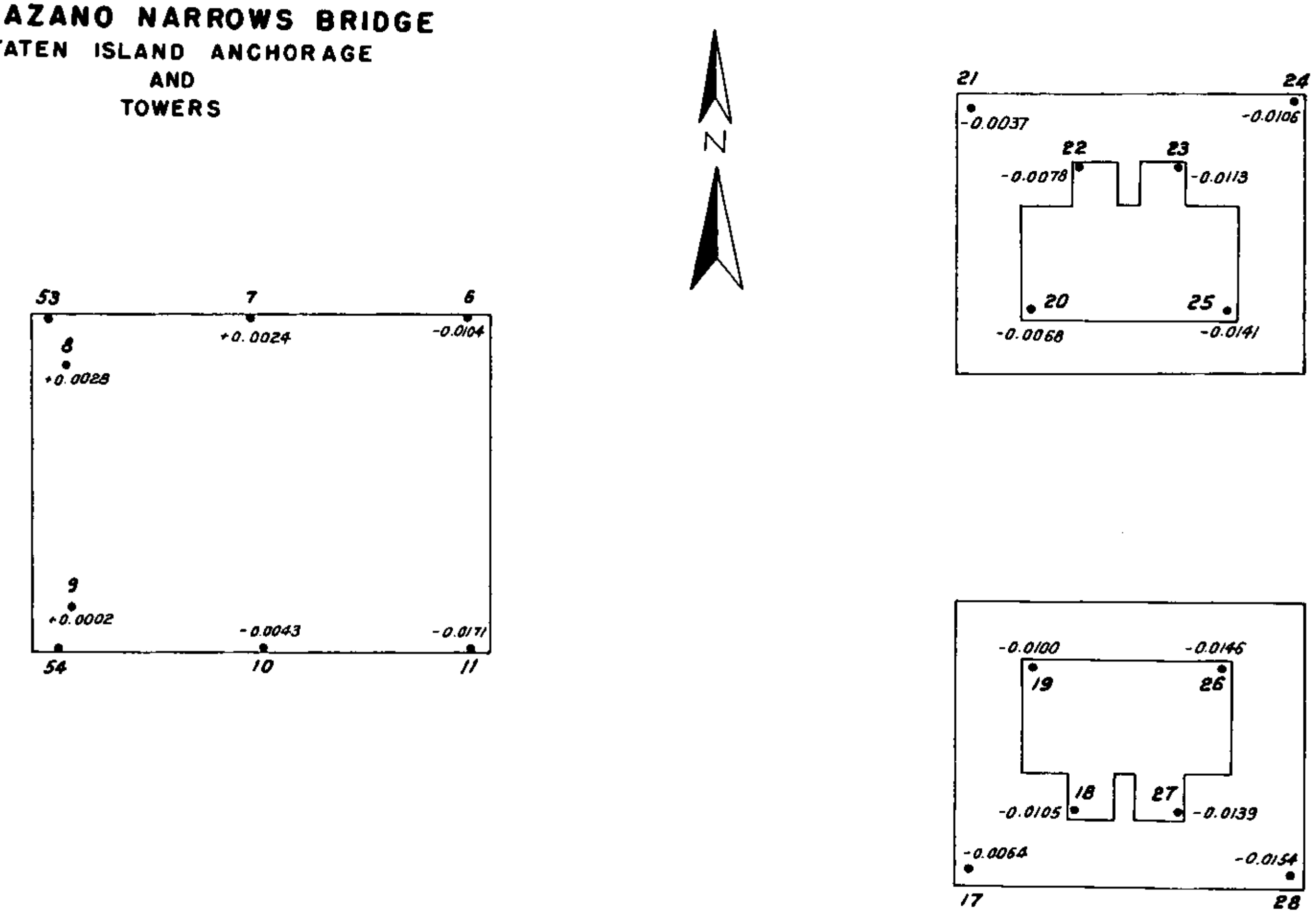
BENCH MARK	NEW ELEVATION	COMMENTS.	OLD ELEVATION	CHANGE
1	98.7656	TBTA-7 CONTROL BM BRASS PLUG	98.7923	-.0267
2	103.7760	US D344 BM BRASS DISK	103.7760	.0000
3	47.7957	TBTA-5 CONTROL BM IRON BOLT	47.7957	.0000
4	104.9472	TBTA-9 CONTROL BM BRASS PLUG	104.9466	.0006
5	98.5258	TBTA-8 CONTROL BM BRASS PLUG	98.5280	-.0022
6	93.8343	15 SI ANCH SETTLEMENT POINT	93.8447	-.0104
7	95.4167	8 SI ANCH SETTLEMENT POINT	95.4143	.0024
8	104.2145	14 SI ANCH SETTLEMENT POINT	104.2117	.0028
9	104.2483	12 SI ANCH SETTLEMENT POINT	104.2481	.0002
10	87.5063	6 SI ANCH SETTLEMENT POINT	87.5106	-.0043
11	75.2846	17 SI ANCH SETTLEMENT POINT	75.3017	-.0171
12	30.0169	TBTA-4 CONTR BM BRASS PLUG	30.0169	-.0000
13	43.5211	TBTA-6 CONTR BM BRASS PLUG	43.5211	.0000
14	12.2324	US TIDAL-5 BRASS PLUG	12.2324	.0000
17	30.6439	G1 SI TOWER OLD SETTLEMENT PT	30.6503	-.0064
18	32.8202	ST 1 SI TOWER BASE PLATE	32.8307	-.0105
19	32.8455	ST 2 SI TOWER BASE PLATE	32.8555	-.0100
20	32.8756	ST 5 SI TOWER BASE PLATE	32.8824	-.0068
21	30.7964	G12 SI TOWER OLD SETTLEMENT PT	30.8001	-.0037

VERAZZANO NARROWS BRIDGE SETTLEMENT REPORT STATEN ISLAND SIDE 1976

BENCH MARK	NEW ELEVATION	COMMENTS.	OLD ELEVATION	CHANGE
22	32.8760	ST 6 SI TOWER BASE PLATE	32.8838	-.0078
23	32.8428	ST 7 SI TOWER BASE PLATE	32.8541	-.0113
24	30.5930	A12 SI TOWER OLD SETTLEMENT PT	30.6036	-.0106
25	32.8190	ST 8 SI TOWER BASE PLATE	32.8331	-.0141
26	32.7856	ST 3 SI TOWER BASE PLATE	32.8002	-.0146
27	32.7874	ST 4 SI TOWER BASE PLATE	32.8013	-.0139
28	30.5174	A1 SI TOWER OLD SETTLEMENT PT	30.5328	-.0154
53	96.1066	22 SI ANCH SETTLEMENT POINT	########	######
54	97.0408	24 SI ANCH SETTLEMENT POINT	########	######


LOOP LAY - OUT
VERRAZANO NARROWS BRIDGE
STATEN ISLAND ANCHORAGE
AND
TOWERS
LEGEND
Loop number
Direction of higher elevation and Loop no.
Bench mark with computer number
NORTH TOWER BASE
SOUTH TOWER BASE
ANCHORAGE
℄ of Bridge
Loop 1.
Loop 2.
Loop 3.
Loop 4.
Loop 5.
Loop 6.
Loop 7.
Loop 8.
Loop N.
US-D-344
TBTA-4
TBTA-5
TBTA-6
TBTA-7
TBTA-8
TBTA-9
A-1
A-12
G-1
G-12
ST-1
ST-2
ST-3
ST-4
ST-5
ST-6
ST-7
ST-8
6-S1
8-S1
11-S1
12-S1
14-S1
15-S1
17-S1
22-S1
24-S1

ELEVATION CHANGES 1970 - 1976
VERRAZANO NARROWS BRIDGE
STATEN ISLAND ANCHORAGE
AND
TOWERS



VERAZZANO NARROWS BRIDGE SETTLEMENT REPORT   BROOKLYN SIDE   1976

BENCH MARK	NEW ELEVATION	COMMENTS.	OLD ELEVATION	CHANGE
29	30.1789	G12 BKLYN TOWER OLD SETTL. PT	30.2031	-.0242
30	32.5690	BT 6 BKLYN TOWER BASE PLATE	32.5927	-.0237
31	32.6195	BT 5 BKLYN TOWER BASE PLATE	32.6439	-.0244
32	32.7109	BT 2 BKLYN TOWER BASE PLATE	32.7277	-.0168
33	32.7446	BT 1 BKLYN TOWER BASE PLATE	32.7591	-.0145
34	30.4497	G 1 BKLYN TOWER OLD SETTL. PT	30.4616	-.0119
35	30.4002	A 1 BKLYN TOWER OLD SETTL. PT	30.4099	-.0097
36	32.7306	BT 4 BKLYN TOWER BASE PLATE	32.7445	-.0139
37	32.6807	BT 3 BKLYN TOWER BASE PLATE	32.6969	-.0162
38	32.5924	BT 8 BKLYN TOWER BASE PLATE	32.6145	-.0221
39	32.5491	BT 7 BKLYN TOWER BASE PLATE	32.5728	-.0237
40	30.0805	A12 BKLYN TOWER OLD SETTL. PT	30.1014	-.0209
41	26.6670	2B BKLYN ANCHOR SETTLEMENT PT	26.6836	-.0166
43	12.6000	183 BE&A MOORING POST	12.6000	-.0000
44	8.8465	US TIDAL-5 BM BRASS PLUG	8.8390	.0075
45	27.1920	US J 342 BM BRASS PLUG	27.1920	.0000
46	42.7997	TBTA-3 CONTROL BM BRASS PLUG	42.7997	.0000
47	54.5321	TBTA-2 CONTROL BM BRASS PLUG	53.7615	.7706
48	34.5932	8B BKLYN ANCHOR SETTLEMENT PT	34.5999	-.0067

SUMMARY OF LOOP CLOSURES BEFORE BALANCING

LOOP NO.	ERROR OF CLOSURE
1	.00106
2	.00140
3	-.00391
4	-.01972
5	.00003
6	-.00112
7	-.00027
8	-.01207
9	-.00136
10	.00000

VERAZZANO NARROWS BRIDGE SETTLEMENT REPORT BROOKLYN SIDE 1976

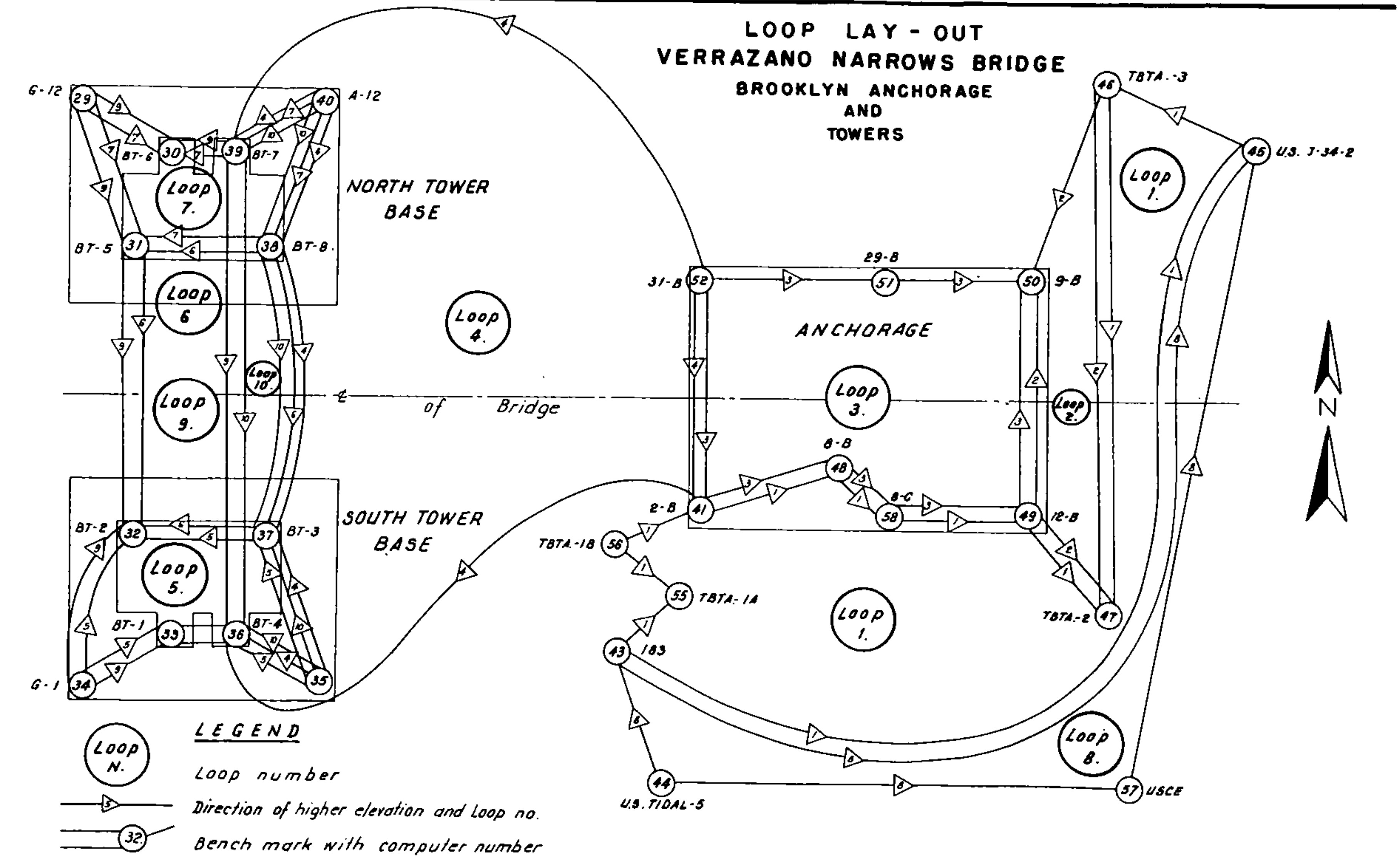
BENCH MARK	NEW ELEVATION	COMMENTS.	OLD ELEVATION	CHANGE
49	52.4752	12B BKLYN ANCHOR SETTLEMENT PT	52.4829	-.0077
50	52.5004	9B BKLYN ANCHOR SETTLEMENT PT	52.5078	-.0074
51	34.7820	29B BKLYN ANCHOR SETTLEMENT PT	34.7933	-.0113
52	25.6038	31B BKLYN ANCHOR SETTLEMENT PT	25.6189	-.0151
55	12.7600	TBTA-1A	########	######
56	18.3415	TBTA-1B	########	######
57	11.2585	USCE	########	#######
58	34.9701	8C BKLYN ANCHOR BRASS PLUG	########	#######

VERAZZANO NARROWS BRIDGE SETTLEMENT REPORT BROOKLYN SIDE 1976

LOW BM.	HIGH BM.	LINK NO.	DIST.	ELEVATION DIFF.	CORRECTION	CORRECTED EL. DIFF.	LOOP NUMBERS +	+	-	-	COMMENTS.
34	33	36	24	2.29500	-.00009	2.29491	0	0	5	9	
34	32	74	50	2.26103	.00019	2.26122	5	9	0	0	
31	32	38	89	.09209	-.00069	.09140	0	0	6	9	
29	30	40	24	2.38995	.00012	2.39007	7	9	0	0	
40	39	42	24	2.46857	.00004	2.46861	4	0	7	10	
29	31	75	50	2.44087	-.00025	2.44062	0	0	7	9	
39	30	76	29	.01999	-.00015	.01984	0	0	7	9	
40	38	77	50	2.51200	-.00008	2.51192	7	10	4	0	
38	37	44	89	.08816	.00010	.08826	6	10	4	0	
35	37	78	50	2.28035	.00015	2.28050	4	0	5	10	
37	32	79	50	.03002	.00020	.03022	6	0	5	0	
38	31	80	50	.02721	-.00013	.02708	7	0	6	0	
35	36	46	24	2.33050	-.00007	2.33043	5	10	4	0	
36	33	81	29	.01387	.01387	.01398	5	9	0	0	
41	36	62	1241	6.05424	.00936	6.06360	4	0	0	0	
52	41	56	168	1.06252	.00067	1.06319	4	0	3	0	
52	39	61	1253	6.95474	-.00945	6.94529	0	0	4	0	
49	50	59	174	.02583	-.00070	.02513	2	0	3	0	
51	50	58	390	17.71700	.00139	17.71839	3	0	0	0	
52	51	57	222	9.17734	.00079	9.17813	3	0	0	0	

LOW BM.	HIGH BM.	LINK NO.	DIST.	ELEVATION DIFF.	CORRECTION	CORRECTED EL. DIFF.	LOOP NUMBERS +	+	-	-	COMMENTS.
43	55	83	140	.15995	.00003	.15998	1	0	0	0	
44	43	53	140	3.75310	.00036	3.75346	8	0	0	0	
44	57	52	120	2.41230	-.00031	2.41199	0	0	8	0	
57	45	72	4374	15.94487	-.01139	15.93348	0	0	8	0	
45	46	51	0	15.60770	.00000	15.60770	0	0	1	0	
46	47	50	538	11.73273	-.00037	11.73236	2	0	1	0	
41	48	48	250	7.92698	-.00083	7.92615	1	0	3	0	
55	56	73	335	5.58143	.00008	5.58151	1	0	0	0	
56	41	55	116	8.32551	.00003	8.32554	1	0	0	0	
49	47	60	184	2.05670	.00013	2.05683	1	0	2	0	
58	49	82	244	17.50592	-.00081	17.50511	1	0	3	0	
46	50	85	478	9.70046	.00021	9.70067	0	0	2	0	
39	36	86	264	.18174	-.00024	.18150	9	0	10	0	
43	45	87	0	14.59200	.00000	14.59200	8	0	1	0	
48	58	88	20	.37700	-.00007	.37693	1	0	3	0	

LOOP LAY - OUT
VERRAZANO NARROWS BRIDGE
BROOKLYN ANCHORAGE AND TOWERS



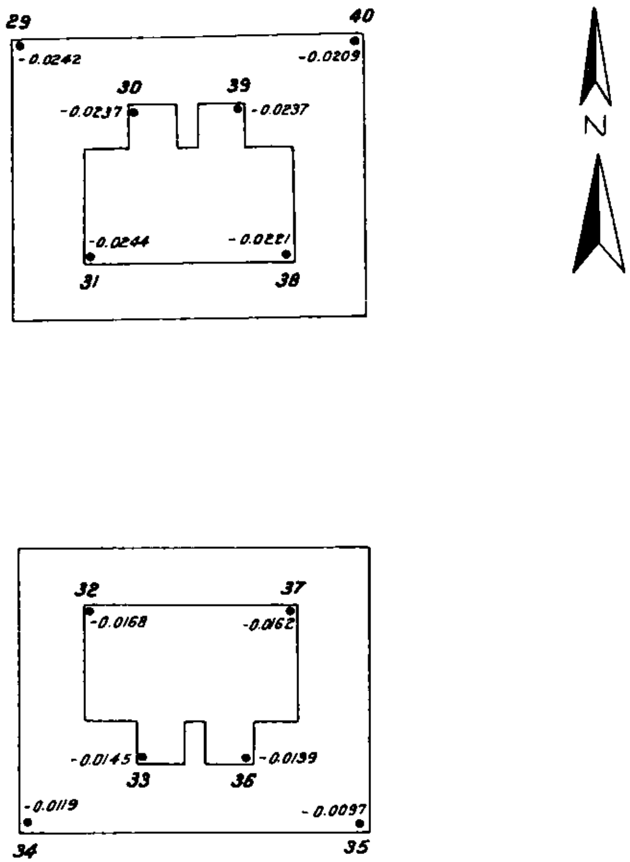
ELEVATION CHANGES 1970 - 1976

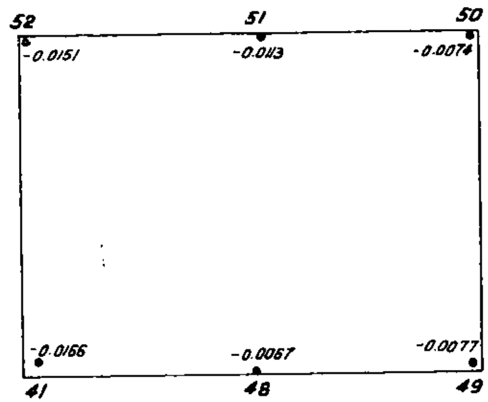
VERRAZANO NARROWS BRIDGE

BROOKLYN ANCHORAGE

AND

TOWERS





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TIDE GAUGING FOR THE 200 MILE FISHERIES LIMIT

by

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TIDE GAUGING
FOR THE
200 MILE FISHERIES LIMIT

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ABSTRACT

Proper demarkation of the 200 nautical mile fisheries limit requires the determination of tidal datums at salient points along the coastline. Because the gauges are placed in the surf zone, the locations are, oftentimes, very difficult in terms of logistics, ocean dynamics, etc.

Under contract to the National Ocean Survey, Pandullo Quirk Associates has installed, and is collecting data on, four (4) gauges established within the North Atlantic Geographical Region of the United States. The installations may be termed unique, requiring some novel technology. The work is fully described, as are the many problems encountered, and recommendations are made for future forays into this field.

The experiences of PQA and the novel solutions to some very difficult problems should prove of interest to anyone interested in marine surveying.

BACKGROUND

The Fisheries Conservation and Management Act of 1976 extends the jurisdiction of the United States to 200 nautical miles offshore. The Act, which was effective as of March 1, 1977, requires the delineation of the low water line from which to establish a base line and, therefore, the 200 mile limit. Accurate demarkation of the 200 mile limit is necessary for enforcement and for the information of all maritime nations as to the limit of U.S. jurisdiction.

The National Ocean Survey (NOS), a part of NOAA and formerly the Coast and Geodetic Survey, has been collecting tidal data since early in the 1800's. Said data is used in coastal charting, prediction of the tides, determination of riparian or littoral boundaries, engineering projects, and the study of tidal phenomena in general. Even though the information on file with NOS is considerable, additional tidal information was necessary to meet the mandate of the Fisheries Act.

To meet its obligation, NOS decided to contract to qualified professionals. They began by soliciting proposals on April 11, 1977, returnable by May 19, 1977, for certain geographical regions: North Atlantic, South Atlantic, Gulf Coast, California and Pacific Northwest. Pandullo Quirk Associates (PQA) responded to the solicitation for the North Atlantic geographical region.1/

1/The National Ocean Survey: "Statement of Work for the Institution, Operation, and Removal of Secondary Tide Stations, North Atlantic" REP No. 7-35221. March 21, 1977.

THE PROJECT

The North Atlantic region required the design, installation, operation, maintenance, and removal of four (4) subordinate tide stations. These tide stations consisted of two (2) secondary stations - 12 valid months of data that should be continuous, and two (2) tertiary stations - 3 valid months of data that should be continuous. The stations were to be located as follows:

Station	Location	Type
Old Harbor Station	Nauset Beach, Cape Cod, Mass.	Secondary
Coast Guard Station	Nantucket Island, Mass.	Tertiary
Surfside	Nantucket Island, Mass.	Tertiary
Shinnecock Inlet	West Bank, Long Island, N.Y.	Secondary

More particularly, each station required the construction of an offshore tide staff supporting structure (all were in the surf zone) and an onshore gauge supporting structure. The gauges, all gas-purged pressure recording (bubbler) types, were government supplied and required an orifice mounted on the offshore structure and a bubbler tube buried under the ocean bottom to the onshore gauge (see Fig. 1).

The contractor was also required to install tidal bench marks and to perform Second Order, Class I leveling. The bench marks were to be set in bed rock, substantial foundations, or atop galvanized iron rods driven 50 feet (or to refusal). Most were of the latter type.

Various government forms and reports were required. A "Tide Station Report" (NOAA Form 77-12) was required at the time of installation, removal, leveling and any construction on the station. NOAA Form 75-29 "Precise Leveling" and NOAA Form 76-83 "Abstract of Precise Levels" were required when leveling was performed and tidal bench mark descriptions, revised bench mark descriptions, recovery notes and bench mark sketches were required when bench marks were established or recovered.

Each month a form was required that listed the daily staff-gauge-time records, as was a monthly report on the condition of the station and the marigram record of the tide.

Primarily, it was of utmost importance to provide NOS with accurate and processable tidal data. Any of several problems could result in a month's data being invalid:

1) a break in the record greater than 72 continuous hours
2) Data that is illegible or obliterated for a period greater than 72 continuous hours
3) Less than five staff-gauge-time readings per week
4) Data that is affected by malfunction of the gauge, clogging of the orifice, etc.
5) Staff movement of more than 0.05 feet which cannot be documented on the record
6) Quality deficiencies that reduce the efficiency of processing by NOS

It was required to provide an additional month's data for each month of invalid data.

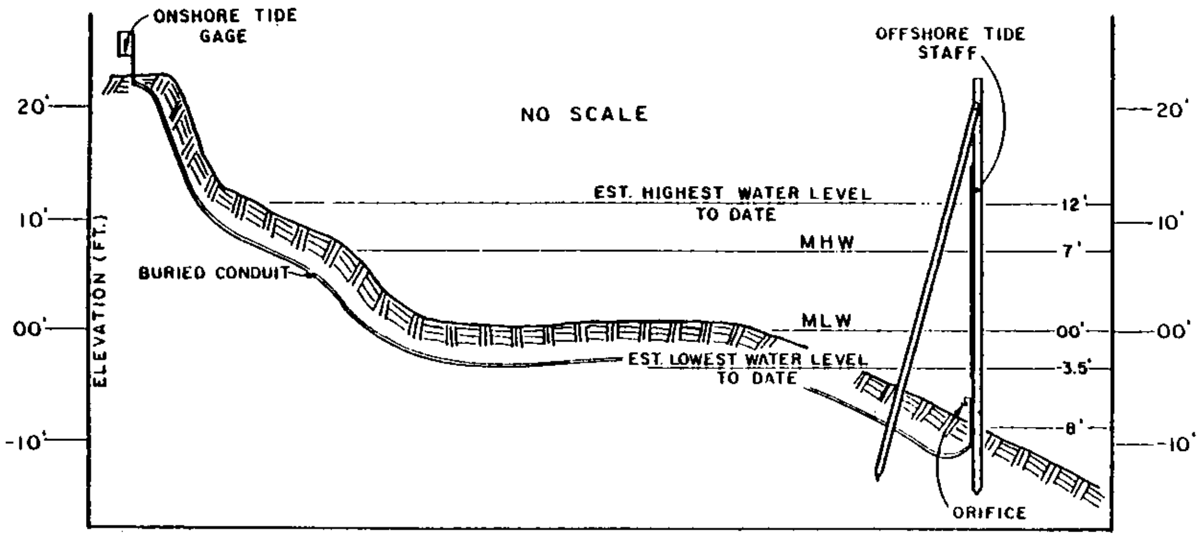

Figure 1. The installation concept.

PQA prepared a proposal for the project, as defined above, integrating a team of in-house Professional Land Surveyors and Oceanographers with a professional consultant in tidal datum determinations, Paul T. O'Hargan, P.L.S. (then of Tri-County Engineering, Naples, Florida), and a marine contractor, David Allen (Chesterfield Associates, Westhampton Beach, N.Y.). The proposal is a considerable document detailing technical qualifications, organization of the project team, available equipment and facilities, resumes of key personnel, applicable experience, work concept and methodology for each station, schedule, and fee.2/

The proposal was evaluated by NOS based on criteria established as follows:3/

1) A maximum of 800 points, broken down in accordance with the following:

Organization, conciseness, and clarity of proposal	50 points
Financial stability	50 points
Reputation of organization	60 points
Related experience of organization	90 points
Experience of personnel	120 points
Professional licenses	50 points
Equipment and facilities	50 points
Suitability of proposed installation	100 points
Method of assuring continuous and accurate data	90 points
Recognition of potential problems and solutions	90 points
Accuracy of project schedule	50 points

2/Pandullo Quirk Associates: Documents entitled "U.S. Dept. of Commerce, National Oceanic and Atmospheric Administration, National Ocean Survey, Washington, D.C. North Atlantic Tide Stations, RFP 7-35221 Part I, Technical Proposal and Part II Cost Proposal. May 18, 1977

3/Contract No. 7-35221 Between U.S. Department of Commerce, NOAA/NOS and Pandullo Quirk Associates, Effective June 30, 1977.

2) And a maximum of 200 points based on cost estimates determined by the formula:

$$\text{Points} = \frac{X}{Y}(200)$$

Where X = lowest price received from an offeror scoring at least 560 points of the 800 possible above.

Y = Price submitted by offeror being evaluated.

Thus the government was assured of competent, professional, contractors, who, in turn, were not "bidding" directly for a professional contract. In the opinion of this author, this procedure represents a sound, rational method of competitive rating of proposals, fair to both the government and the contractor. Many other governmental units and agencies would be well advised to adopt similar procedures.

PQA was successful with its proposal, eventually negotiating a contract with the U.S. Department of Commerce. The work commenced immediately.

PERFORMANCE

Considerable planning had been accomplished by NOS, but PQA found it prudent to do some supplemental planning. U.S.G.S. quadrangles and coastal charts proved worthwhile; historical beach profiles and other hydrographic information was invaluable; aerial reconnaissance was useful in pinpointing actual sites and uncovering potential problems; and interviews with the local population were extremely beneficial.

In planning, it was quickly discovered that the logistics involved in the installations were very considerable. Much equipment and material had to be collected from several suppliers. Coordination between PQA, its consultants and NOS was mandatory. Eventually, everything and everyone had to be transported to the sites, some of which were very remote: Nantucket Island can be reached by air or by ferry, both only at considerable expense and inopportune schedule; the gage site at Old Harbor Station on Nauset Beach, Cape Cod, is 7 miles from the nearest paved road; etc. It as also of concern to be working in seasonal resort areas: in winter, convenient, adequate facilities are difficult to locate while availability and cost were prohibitive in the summer.

Careful planning resulted in the arrival on site, at the proper time, with all the necessary equipment, scientific instruments, materials and miscellaneous supplies (generally). Following a brief reconnaissance to confirm our planning, the actual installation proceeded.

Offshore Structure

An offshore structure was necessary to support the tide staff, stilling well and bubbler orifice. Inasmuch as this structure was to be installed in the surf zone, 300-600 feet offshore, great environmental stress could be expected.

The original NOS concept for the offshore structure consisted of a tripodal structure consisting of one vertical and two battered piles*/ (see Figs. 2a and 2b). The structure was to be set at a water depth of -8.0 feet (MLW) and extend to +15.0 feet. Including penetration into the ocean bottom, piles of approximately 40 feet were necessary.

In its proposal, PQA agreed with the NOS concept. Attempts at installation, however, proved that such a structure could not be constructed without very substantial equipment, wholly outside the scope of the project: a temporary tide monitoring installation. PQA and its marine contractor, Chesterfield Associates, then designed an alternative structure (Fig. 3), which was prefabricated from steel, delivered to the proper position, and fastened to the bottom by means of steel pipe jetted into the bottom. the installation of this alternative was greatly facilitated by use of Chesterfield's amphibious vehicle (a salvaged WWII DUKW amphibious transport).

Results with the new structure were mixed. On Nantucket Island, for example, our installation at the Coast Guard Station proved to be extremely durable, lasting for more than 8 months before it was removed, while at the same time our installation at Surfside, about 2 miles away, was plagued with difficulties: destroyed by a storm shortly after installation, reinstalled only to be damaged by another storm, and ultimately destroyed by a third storm (all within 8 months). Our installation at Old Harbor Station on Cape Cod survived the violent northeaster of February 7 and 8, 1978, even though our gauge and its appurtenances did not, while our Shinnecock Inlet gauge fell prey to the same storm when it had survived the others.

With the benefit of afterthought, it appears that the new structure was probably too light to survive the severe environmental stresses of the surf zone. The NOS concept, however, in this author's opinion, cannot be constructed at reasonable cost (at least not in the North Atlantic zone) within the concept of a temporary installation.

Subsequent designs by NOS, based on their experiences in this and other contracts, modify their original concept. The revisions include replacing the wooden piles with steel pipe and fabricating an assembly for the tide staff and stilling well which can be welded to the main pipe. Although the pipe is easier to jet into the bottom than a wooden pile, PQA's experience continues to indicate that large and expensive equipment is necessary for installation.

At Shinnecock Inlet, PQA eventually received NOS approval to bottom mount the bubbler orifice without the extensive structure previously required. This was made possible by mounting the tide staff and stilling well on an existing wooden piling in the general vicinity. As you might expect, bottom mounting eliminates much of the environmental stress, hence a more lasting and stable installation. As of this writing (January, 1979), our bottom mounted orifice at Shinnecock Inlet has survived 8 months, without a single flaw.

*/The National Ocean Survey: "Statement of Work for the Institution, Operation, and Removal of Secondary Tide Stations, North Atlantic" REP No. 7-35221. March 21, 1977.

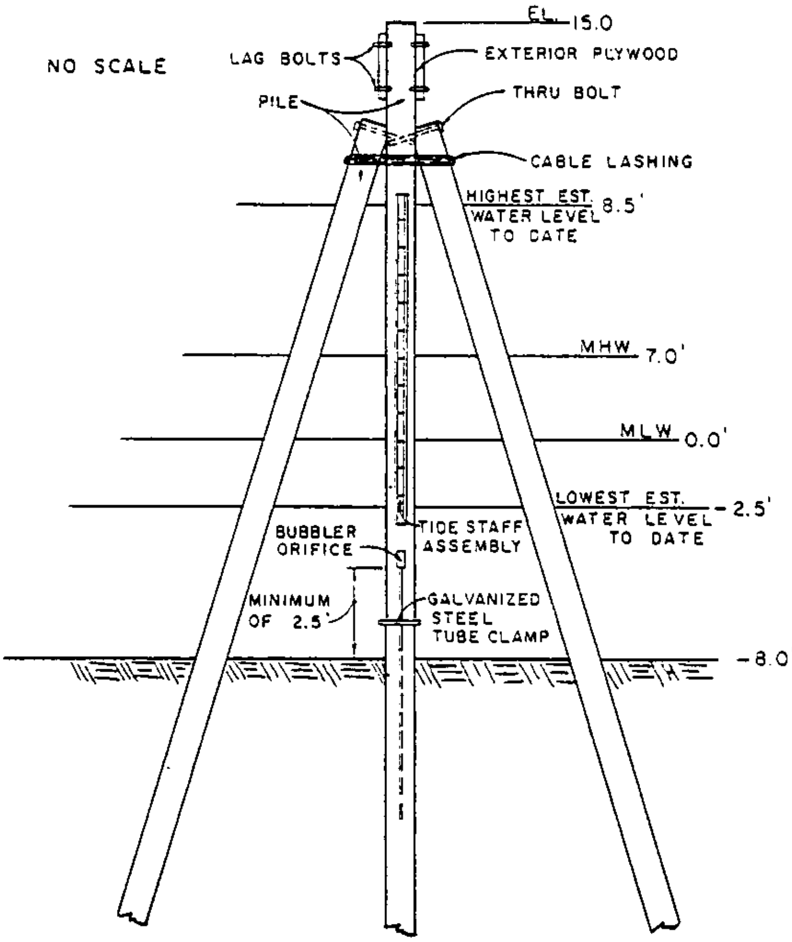

Figure 2a. NOS Offshore structure, shore view.

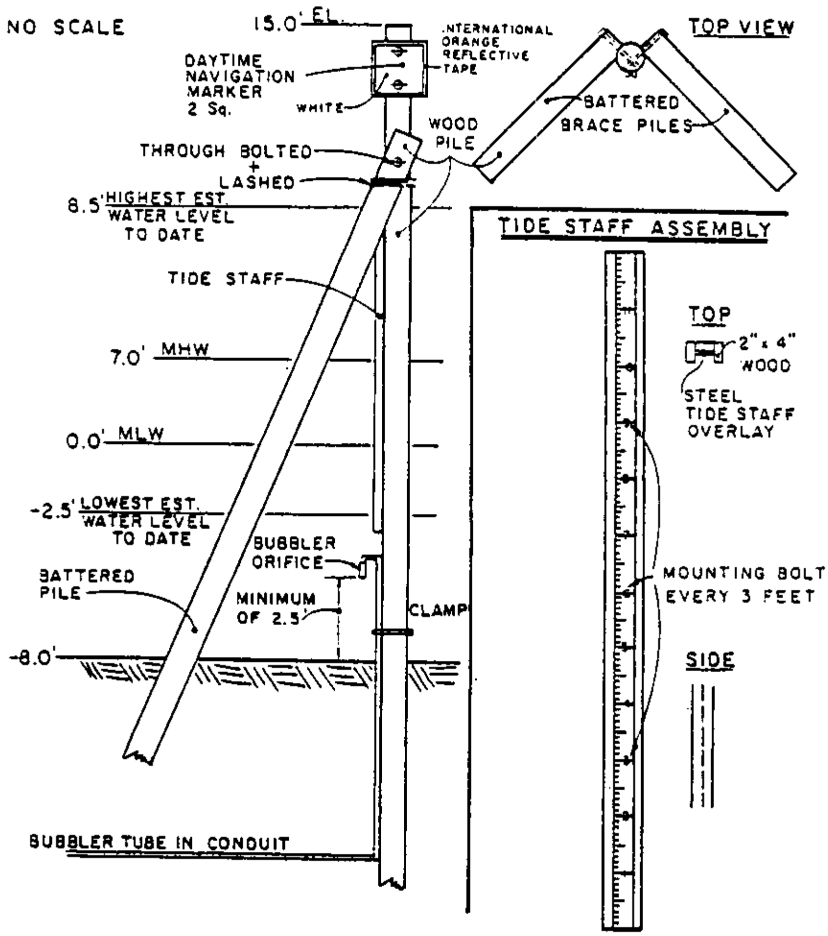

Figure 2b. NOS Offshore structure, side view.

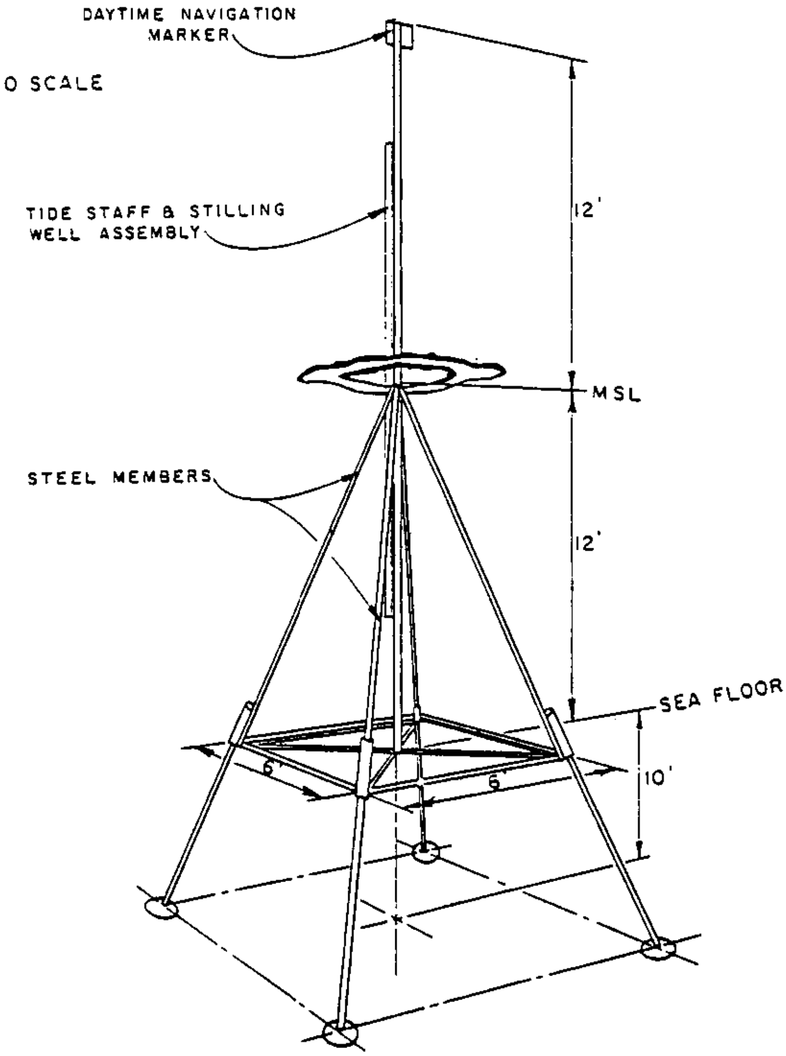

Figure 3. PQA Offshore structure.

Several miscellaneous problems deserve mention. Offshore structures in areas where swimming is prevalent will attract climbers. In addition to the obvious danger to the swimmers themselves, the climbers tear free the stilling wells, navigational markers, and similar items, which cause more than normal maintenance. Another problem, especially in the summer months, is marine growth on the staff and stilling well. This requires maintenance on a monthly basis, which PQA included with its regular gauge maintenance.

Tide Gauges and Appurtenances

At all four sites, NOS methodology called for nitrogen bubbler gauges.*/ PQA was provided with new pressure type 0-10 and 0-20 gauges and appurtenances. Each gauge was mounted on an onshore panel along with a security cover for the nitrogen supply tank (80 CF tanks were employed). Care was taken to armor the nitrogen line to prevent vandalism.

Once clear of the onshore panel, the nitrogen line was buried at least 3 feet below the surface of the beach. Under the water, the line was secured to heavy chain by means of plastic electrical tie wraps. It was not necessary to jet the line into the bottom becuase it settled almost immediately. In fact, such treatment was effective against beach profile changes and PQA found it a rare occurrence to have any problems with the nitrogen lines.

An early proposal by PQA, to install an overhead nitrogen line at one of the sites, proved to be unworkable. Permits for such an installation were extremely difficult to secure while our reason for an overhead line, dramatic beach profile changes, became less of a worry with our success in chain weighting. The concept, however, remains an interesting alternative.

There were many problems. In an early installation we were perplexed because we could not get the gauge to operate normally. Following the checklist (see Appendix I) exactly (and several times) resulted in a build up of pressure in the gauge which would not dissipate. Extensive troubleshooting found the problem: a blocked nitrogen line. This turned out to be a manufacturing defect which we found in several other rolls of tubing. A lesson was learned and, thereafter, all equipment and material was checked, insofar as possible, before installation.

Weather conditions seemed to conspire against the project in the early going. Storm after storm lashed the North Atlantic coast, each one causing a problem somewhere. The "final straw" was the northeaster of February 7-8, 1978, which was, by all accounts, one of the worst storms on record. Especially hard hit was the Old Harbor Station gauge which was the site of an overwash: the ocean met the bay. The final toll: 4 buildings destroyed, the tide gauge lost along with all appurtenances, two bench marks destoyed and another buried, and all protective dunes lost for at least 100 metres. The gauge, as luck would have it, washed ashore on the north side of Nantucket Island (25 miles away) three days later, minus the nitrogen tank. It was badly damaged and is probably beyond salvage. Weather has been insignificant (thankfully) since.

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*/The National Ocean Survey: "Statement of Work for the Institution, Operation, and Removal of Secondary Tide Stations, North Atlantic" REP No. 7-35221. March 21, 1977.

Another problem developed in the gauges themselves. Failures were common in the chart drives because of a manufacturing defect in the main springs. Although the manufacturer repaired the chart drives under his warranty to the government, each gauge had to be repaired immediately following a failure, and failures occurred several times in some gauges.

Security was anticipated to be a problem and PQA was not disappointed in this regard. In fact, we once lost an entire onshore panel, thankfully without the gauge and appurtenances, to a beach party bonfire! Other acts of vandalism were also noted but none had any effect on the data collection. Another problem with security was the maintenance of the locks on the security covers. Even though the best hardware was purchased, constant care was necessary to keep the tumblers free. Even so, six months was the longest time that could be expected before replacement.

Bench Marks and Leveling

NOS regulations4/ specify that three deep rod bench marks and two additional bench marks, set in the most suitable monuments for the locality, are required for each secondary station. Stability conditions at the sites, however, mandated that most of the marks be of the deep rod type.

Deep rod bench marks require the driving of a 1/2" galvanized iron rod to a depth of 50 feet, or refusal. A standard disc is crimped to the top of the rod which is protected by a length of PVC pipe encased in concrete. Rods were supplied in 8 foot lengths, with threaded connectors. Driving of the rods was accomplished by employing a gasoline driving hammer (with a rod driving attachment) and the discs were fastened to the rods with a hydraulic crimping tool. For safety, a tripod was employed to support the driving hammer during the installation of the rods. The tripod was one normally used in manhole work, modified to give it greater height.

Leveling between bench marks and the tide staff is required at installation, stability checks (every 6 months, minimum) and removal. Leveling was required to be to Second order, Class I standards. By employing a second order automatic level, a set of matching invar leveling staffs, a 3-wire procedure, and good general practices, it was no problem to meet the required accuracy. PQA did, however, find it expedient to measure out its turning points before beginning the leveling.

There were few problems in this portion of this work. It is required to peg test the instrument before beginning the leveling and it is prudent to reduce and check all notes before leaving the field. As always, accurate and detailed descriptions of the bench marks are necessary to insure their later recovery.

General

There are several other areas which also deserve attention.

4/Bodnar, A. Nicholas, Jr., LCDR, NOAA: "User's Guide for the Establishment of Tidal Bench Marks and Leveling Requirements for Tide Stations." December 8, 1975.

Training of the tide observers is very important. If at all possible, the observer should be present during the completion of the installation and during the time when the gauge is put "on-line." At least, he should be fully briefed. A check list is helpful, as is the name, address, and telephone number where he can get help.

It became obvious, quite early, that NOS methodology for making daily staff-gauge-time readings was inadequate. Observation of the staff and stilling well from shore (up to 600 feet), with binoculars, was not yielding data that appeared to correlate. As a first step, PQA suggested a larger tide staff. Results were not better with the new staffs which were, it should be pointed out, much more expensive. Our second suggestion solved the problem: a spotting scope. These are normally employed to direct rifle fire and come with interchangeable eye pieces in 20X, 30X, 40X, 50X and 60X. With maximum magnification, a little time to study the staff and stilling well (through several swells), and practice, a very high correlation can be achieved.

Another good practice is to make a monthly maintenance trip to the gauge, within the first five days of the month. At this time the marigram is replaced, the tide staff and stilling well cleaned, the nitrogen tank changed (as required), any maintenance performed, leveling conducted (when necessary), and the tide observer interviewed. This will coincide with the monthly reporting to NOS.

RECOMMENDATIONS

Tidal gauging in the surf zone is an extremely difficult undertaking, replete with dangers to the unwary. It should only be attempted by persons with strong constitutions and gambling instincts. However, there are several ways, in this author's opinion, to alleviate some of the more serious problems that exist in this type of work.

Most importantly, a procedure should be devised to eliminate the daily staff-gauge-time readings. This would allow bottom mounting of the bubbler orifice where it is less susceptible to environmental stress and of course, there would be no offshore structure required. Frequent leveling checks could be made on the mounting to get a relationship to the tidal bench marks. It should be pointed out that this will not eliminate daily visits by a tide observer, merely his reading of the staff. Also, conventional NOS methodology is completely appropriate to all its other tide gauging applications. In surf zone gauging, the problems of ocean dynamics and distance from the staff are the primary troubles that must be corrected.

Another suggestion is for NOS to enter into contracts for this type of work with compensation on a basis other than "fixed fee." The work has proven to be highly dynamic and, therefore, detrimental to the government or to the contractor. I suggest a "cost plus fixed fee" contract would be equitable to both parties.

A final suggestion, especially appropriate if the government insists on fixed fee contracts, is for NOS to separate the work into several contracts. Thus the marine construction, gauge installation, maintenance, etc.; and surveying can be contracted for separately. This would tend to insure that the best qualified persons in each area are employed and that the compensation for each portion is the direct responsibility of the party providing the service.

ACKNOWLEDGEMENTS

In all, I must state that NOS has been exceptionally proper in its dealings with PQA. NOS officials have been eager to provide helpful suggestions and advice and NOS personnel have spent long hours processing doubtful data in an effort to salvage whatever was useful and, thereby, save the contractor extra effort. PQA is indebted to NOS for their attitude which we have tried to repay with additional effort and dedication. PQA is proud to be associated with the professionals at NOS.

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