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31 December 2010

**US Army Corps
of Engineers**

ENGINEERING AND DESIGN

Standards and Procedures for Referencing Project Elevation Grades to Nationwide Vertical Datums

ENGINEER MANUAL

CECW-CE

DEPARTMENT OF THE ARMY
U.S. Army Corps of Engineers
Washington, DC 20314-1000

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Manual
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Engineering and Design
STANDARDS AND PROCEDURES FOR REFERENCING PROJECT ELEVATION GRADES TO NATIONWIDE VERTICAL DATUMS

1. Purpose. This manual provides technical guidance for referencing project elevation grades to nationwide vertical datums established and maintained by the U.S. Department of Commerce. It supplements ER 1110-2-8160 (*Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*) that requires controlling elevations and local datums on USACE projects shall be properly and accurately referenced to nationwide spatial reference systems used by other Federal, state, and local agencies responsible for flood forecasting, inundation modeling, water control, flood insurance rate maps, navigation charting, and topographic mapping.

2. Applicability. This manual applies to all USACE commands having responsibility for the planning, engineering, design, construction, operation, maintenance, and regulation of flood risk management, coastal storm damage reduction, hurricane protection, multi-purpose water supply/control, hydropower, regulatory, ecosystem restoration, and navigation projects.

3. Distribution. This publication is approved for public release; distribution is unlimited.

4. Discussion. ER 1110-2-8160 requires that the designed, constructed, and maintained elevation grades of USACE projects shall be reliably and accurately referenced to a consistent nationwide framework, or vertical datum—i.e., the National Spatial Reference System (NSRS) or the National Water Level Observation Network (NWLON) maintained by the U. S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA). This manual provides technical and procedural guidance for establishing the relationships for designed, constructed, or maintained project grades relative to these nationwide frameworks.

FOR THE COMMANDER:

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CHAPTER 1

Introduction

1-1. Purpose. This manual provides technical guidance for referencing project elevation grades to nationwide vertical datums established and maintained by the U.S. Department of Commerce. It supplements ER 1110-2-8160 (*Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*) that requires controlling elevations and local datums on USACE projects shall be properly and accurately referenced to nationwide spatial reference systems used by other Federal, state, and local agencies responsible for flood forecasting, inundation modeling, water control, flood insurance rate maps, navigation charting, and topographic mapping.

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1-3. Distribution. This publication is approved for public release; distribution is unlimited.

1-4. References. Referenced USACE publications and related bibliographic information are listed in Appendix A. Where applicable, primary source material for individual chapters may be noted within that chapter.

1-5. Discussion. ER 1110-2-8160 requires that the designed, constructed, and maintained elevation grades of USACE projects shall be reliably and accurately referenced to a consistent nationwide framework, or vertical datum—i.e., the National Spatial Reference System (NSRS) or the National Water Level Observation Network (NWLON) maintained by the U. S. Department of Commerce, National Oceanic and Atmospheric Administration (NOAA). This manual provides technical and procedural guidance for establishing the relationships for designed, constructed, or maintained project grades relative to these nationwide frameworks.

1-6. Background. In the aftermath of Hurricane Katrina in 2005, a study by the "Interagency Performance Evaluation Taskforce" (IPET 2007) found a number of project elevation and reference datum issues that had Corps-wide impact. Subsequent Corps-wide reviews revealed that flood protection and water control structure elevation grades were often referenced to uncertain or superseded terrestrial-based geodetic vertical datums instead of hydraulic/water-level referenced datums from which the structural protective elevations were designed. In some cases, long-term land subsidence, seasonal tidal fluctuations, and sea level change were not always fully compensated for in flood protection structure design or periodically monitored after construction. In addition, navigation projects in tidal regions were often defined to a vertical reference datum that was not based on the latest tidal model for the region, or were defined relative to a datum that was inconsistent with recognized national or international maritime datums. The technical variations and uncertainties between geodetic, satellite-based (ellipsoidal), and water level datums, and their proper application on engineering and construction projects, were often misunderstood.

a. Datum uncertainty impacts. The IPET study found that inadequate or uncertain geodetic and water level datums can lead to the design and construction of deficient flood protection structures. In areas experiencing subsidence or post-glacial rebound, the relationship between the water surface and the project structures changes through time. In coastal districts, sea level change adds to the dynamic nature of this relationship. Reference datum relationships and elevation uncertainties must be factored into the overall risk analysis and design of flood protection structures and navigation project grades. The hydraulic and geodetic elevation relationships must be verified during construction, and periodically monitored after construction to account for subsidence, settlement, periodic nation-wide reference datum redefinitions and readjustments, sea level change, and other factors.

b. Relationships between hydraulic and geodetic datums. Establishing a solid relationship between hydraulic/tidal datums and geodetic datums is critical in relating measurements of wave heights and water level elevations, high-resolution hydrodynamic conditions, water elevations of hydrostatic forces and loadings at levees and floodwalls, elevations of pump station inverts, and related elevations of flood inundation models deriving drainage volumes or first-floor elevations in residential areas. This is best illustrated by the following excerpt from a report "*A National Vertical Datum Transformation Tool*" (Parker 2003).

> *"... the land-water interface depends on how water levels change in both space and time. To combine or compare coastal elevations (heights and depths) from diverse sources, they must be referenced to the same vertical datum as a common framework. Using inconsistent datums can cause artificial discontinuities that become acutely problematic when producing maps at the accuracy that is critically needed by Federal, state, and local authorities to make informed decisions."*

The relationship between the geodetic and hydraulic datums may or may not be easily defined. More often than not, the relationship is complex and requires field survey observations or extensive modeling to quantify. These relationships are especially critical on coastal hurricane protection and navigation projects where accurate hydrodynamic tidal modeling is essential in relating water level elevations to a datum that varies spatially and is time varying due to subsidence or sea level changes. Datums in other parts of the country may be subject to post-glacial rebound. Thus, there is no consistent, non-varying, vertical datum framework for many areas—periodic survey updates and continuous monitoring are required for projects experiencing vertical reference variations.

c. Flood mapping studies. The requirement for accurate vertical datums is emphasized in a National Research Council study "*Mapping the Zone—Improving Flood Map Accuracy*" (NRC 2009). This report concluded that "... the accuracy of elevation data has an enormous impact on the accuracy of flood maps. Ensuring that future flood studies are based on the most accurate and consistent foundation possible requires (1) continuation of a suite of agency elevation programs and (2) acquisition of accurate, high-resolution elevation data." The NRC study found that "... the greatest effect by far of any variant on the BFE [Base Flood Elevation] is from the input data for land surface elevation ... [and that] the base flood elevation profile is significantly more influenced by whether the National Elevation Dataset or LIDAR terrain data are used to define land surface elevation than by any variation of methods for calculating channel hydraulics." A 2009 FEMA report to Congress "*Risk Mapping, Assessment, and Planning (Risk MAP) Multi-Year Plan: Fiscal Years 2010-2014"* (FEMA 2009) outlined that agency's plan for enhancing and maintaining the quality of flood hazard data and flood maps, with particular emphasis on expanded use of LIDAR technology to measure accurately referenced elevations in flood hazard areas.

1-7. Scope of Manual. Chapter 2 provides an overview of geodetic, hydraulic, and tidal datums used to define grades on USACE civil works projects, in both CONUS and OCONUS regions. Chapter 3 contains recommended survey procedures and accuracy standards for referencing project grades to federal frameworks. The remaining chapters (4 through 9) contain detailed guidance for referencing datums on specific types of civil works projects. The appendices to this manual contain application examples of civil works projects that have been adequately referenced to the federal datum frameworks.

1-8. General Background on the Definition and Use of Vertical Datums. Vertical datums typically represent a terrestrial or earth-based surface to which geospatial coordinates (such as heights, elevations, or depths) of project grades are referenced. The vertical datum is the base foundation for nearly all civil and military design, engineering, and construction projects in USACE—especially those civil projects that interface with water.

a. USACE vertical datums. In general, there are five types of vertical datums that are used to reference grades on USACE civil works projects.

(1) Geodetic (or Orthometric) Datums (e.g., North American Vertical Datum of 1988—NAVD88, National Geodetic Vertical Datum of 1929—NGVD29)

(2) Hydraulic Datums (e.g., Low Water Reference Planes—LWRP, IGLD, Pool stages)

(3) Tidal Datums (e.g., Mean Sea Level-MSL, Local Mean Sea Level-LMSL, Mean Lower Low Water-MLLW, Mean High Water-MHW)

(4) Local or Legacy Datums (e.g., Mean Low Gulf-MLG, Chicago City Datum, Memphis Datum, Cairo Datum, local river gage stage, US Engineer Datum-USED, COEMLW)

(5) Global Navigation Satellite System (GNSS) Earth-Centered Datums (e.g., GRS80, WSG84)

b. Multiple datums. Most USACE projects interfacing with water are referenced to at least two of the above datums. Some may require reference to all five. Increasing emphasis and eventual dependence on GNSS satellite positioning for primary construction stakeout and machine control will necessitate that all projects be eventually referenced to satellite-based datums, resulting in a minimum of three reference datums for most projects. Given these multiple reference datums, it is critical that the relationship among the datums be firmly established, maintained, and well documented.

c. Legacy or local datums. Many USACE navigation, flood protection, and water control projects are still referenced to superseded datums, such as Mean Low Water, Mean Sea Level, NGVD29, Sea Level Datum of 1929 (SLD29), etc. Projects referenced to these superseded or legacy datums are, in effect, actually referenced to a "Local Datum." The relationship between this legacy datum, the current federal orthometric datum, and the local hydraulic reference plane is often highly uncertain. Projects referenced to legacy datums must, at minimum, be related to the current federal orthometric or tidal datum. This does not preclude the continued use of these legacy datums for navigation, flood risk management, or water control purposes; only that the relationship between the legacy datum and the current federal datum is established, documented, and maintained.

(1) Local navigation project datums. Navigation projects are usually referenced to an established low water reference plane—a tidal low water on coastal projects and a hydraulic-based reference plane on rivers, pools, lakes, and reservoirs. Tidal navigation project grades that were constructed and maintained to an older local low water datum may need to be updated for subsequent sea level or subsidence changes that have occurred since the project was authorized and/or the legacy datum was established. The current relationship between the legacy datum and the current federal tidal reference datum must be clearly indicated on all project documents.

(2) OCONUS local orthometric datums. NAVD88 or NGVD29 is not applicable to OCONUS projects in the Pacific Ocean region, the Caribbean, and in portions of Alaska where established local datums have been defined by NOAA, USACE, or another agency.

d. Project life-cycle variations in datums. The relationship within and between the reference datums listed above often may be complex given they can deviate spatially over a region, due to a variety of reasons. They may also have temporal deviations due to land subsidence or uplift, sea level changes, crustal/plate motion, project reconstruction, periodic readjustments to the datum origin, or to redefined points on the reference surface. Figure 1-1 illustrates the time-dependent datum variations that may be encountered during the life cycle of a project—in this case a hurricane protection project. It also illustrates the uncertainties and risk assessment factors associated with reference datums that must be considered in designing protection elevations over the life cycle of a project.

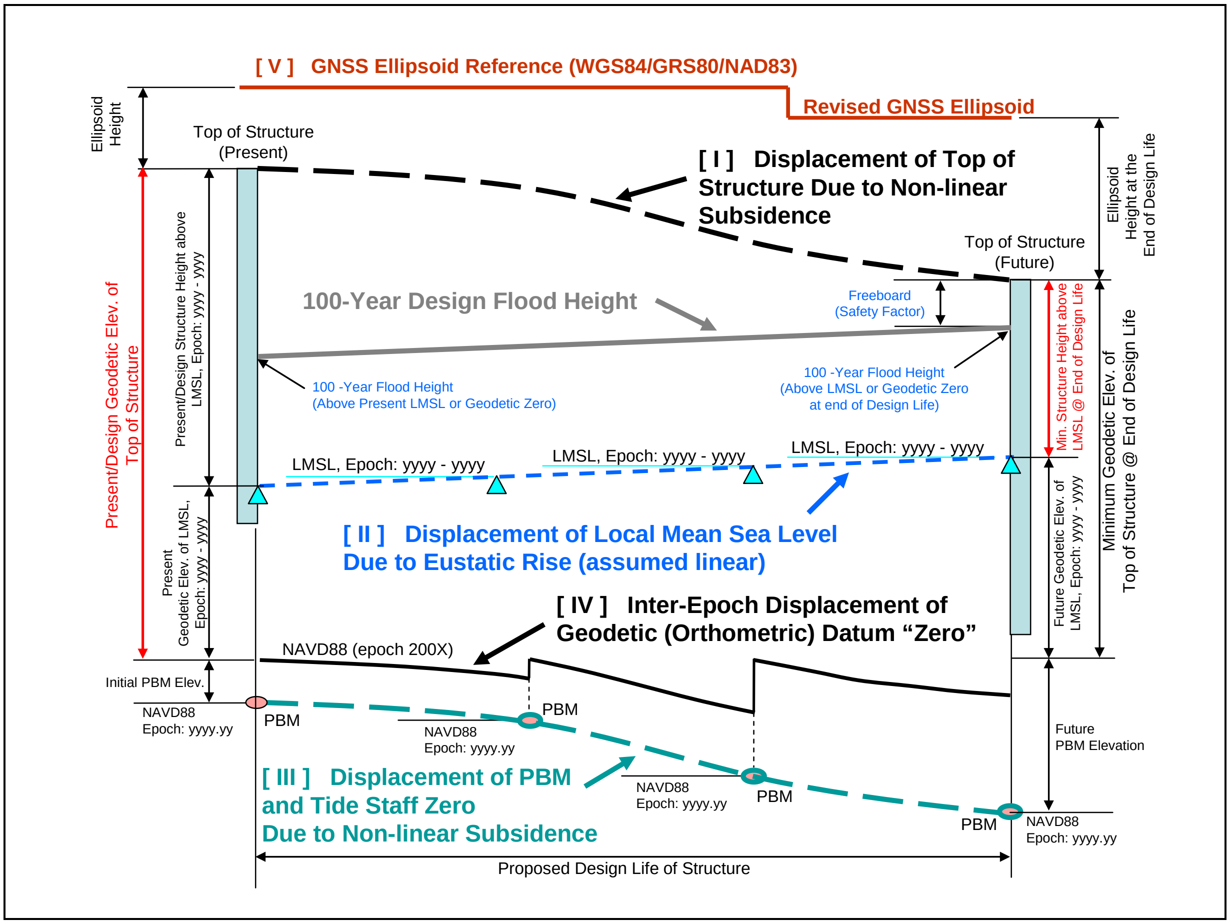


Figure 1-1. Structure design and protection height considerations and uncertainties due to vertical reference datum variations over the life cycle of a project.

(1) In Figure 1-1, the absolute protection elevation of the top of the floodwall is lessened over the project's design life cycle due to local land subsidence [I]. This subsidence is independent of any structure settlement. In addition, apparent (or local) sea level change [II] results in a change of protection, in this example, a loss due to apparent sea level rise. Concurrently, the local reference permanent bench mark(s) (PBM) used to establish constructed grades on the structure, and tide gage reference PBMs used to measure MSL, may be subsiding differently from the structure [III]. In addition, NOAA may make periodic adjustments to the geodetic reference datum [IV], the tidal MSL reference datum [II], or even redefine the datums at some point in the future. The reference ellipsoid datum [V] for satellite GNSS (e.g., GRS80 and WGS84) could also be revised at some point in the future.

(2) Of critical importance in Figure 1-1 is the relationship between the local water surface [II] and the local geodetic reference datum [IV]. This changing relationship must be monitored throughout the life cycle of a project (e.g., 50 + years). Protection elevations are referenced to the local water surface level (e.g., Local Mean Sea Level—LMSL, which in Figure 1-1 is shown rising). The current geodetic reference datum—NAVD88 [IV]—is not based on the design water surface nor is it related to the water surface—it is, in effect, an arbitrary reference system. However, this geodetic datum is used for site plan mapping during design, construction stake out

and grading, and flood plain mapping and related hydrological studies (e.g., resultant Flood Insurance Rate Maps—FIRMS). Future vertical reference datums are proposed on or after 2018 that will align with the gravity/geoid surface and may best fit to hydraulic-dynamic based surfaces.

e. Coastal and inland navigation project references. Referenced depths of navigation projects must likewise have a firmly established relationship between the water surface datum and the geodetic datum. On coastal navigation projects, the reference datum (e.g., Mean Lower Low Water) varies temporally as does Local Mean Sea Level [II] shown in Figure 1-1. It also varies spatially due to the dynamics of tidal ranges in a region, and therefore must be modeled. The modeled MLLW reference plane defines constructed and maintained dredging grades and must be continuously updated for tidal epoch, sea level, and subsidence changes as illustrated in Figure 1-1. On inland navigation projects, the reference datum is normally defined relative to hydraulic parameters (e.g., stages) at local river or pool gages. The geodetic datum is not based on the hydraulic water surface profile; therefore, the relationship between the hydraulic river stages and the geodetic datum must be developed and maintained.

f. Inland flood risk management and water control projects. Elevations of inland flood and water control structures (levees, dams, floodwalls, etc.) are designed, constructed, and maintained relative to hydrologic and hydraulic gage data in the project area. Flow profiles and other computed data from river gages may be referenced to various datums, as shown in Figure 1-2. Computed or modeled flood stage profiles may be referenced to the gage reference zero, the low water reference plane, and/or one of the geodetic datums shown in the figure. The geodetic and satellite reference datums vary spatially between the gages (i.e., they are non-parallel) and contain uncertainties that factor in to the overall uncertainty of a computed flood stage; and thus into the uncertainty and risk analysis estimates of the protection elevation of an adjacent levee, floodwall, or dam—reference ER 1105-2-101 (*Risk Analysis for Flood Damage Reduction Studies*). Figure 1-3 illustrates the elevation uncertainties on a flood protection structure resulting from propagated errors in both reference datums and survey connections. Accurate modeling of water surface profiles on a river system depends (in part) on having a consistent reference datum at the primary gages. Chapters 3 and 6 provide survey procedures and standards for connecting these various reference datums on flood risk management and water control projects. Estimated datum uncertainties are outlined in Chapter 9.

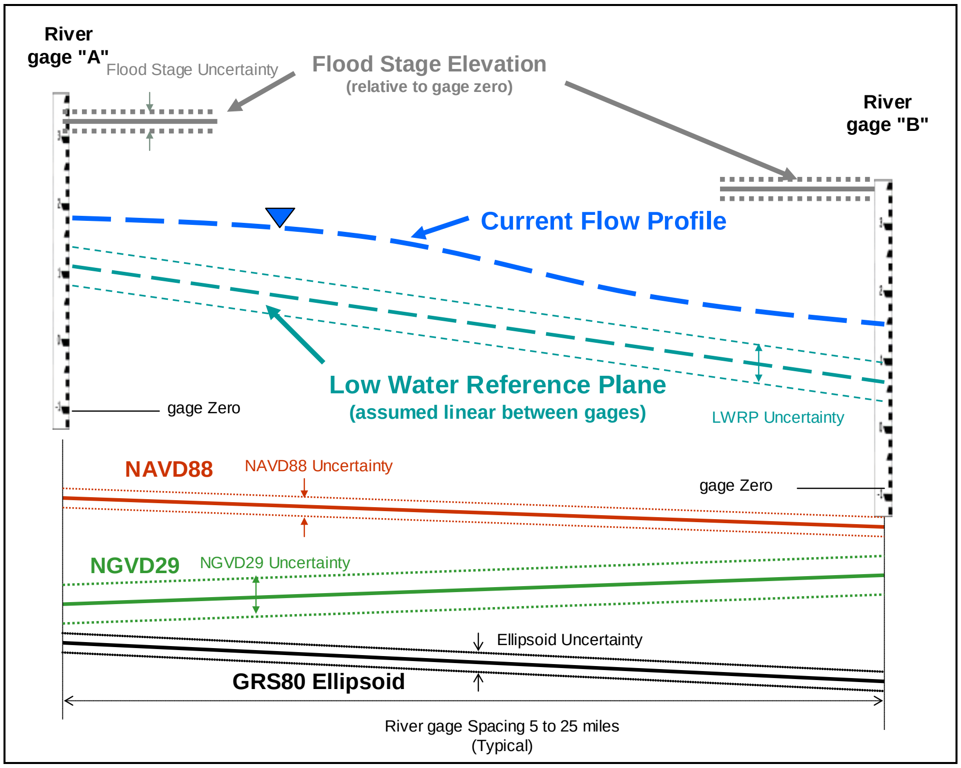


Figure 1-2. Design and protection height considerations and uncertainties due to vertical reference datum variations at gages on an inland river system. (Not to scale)

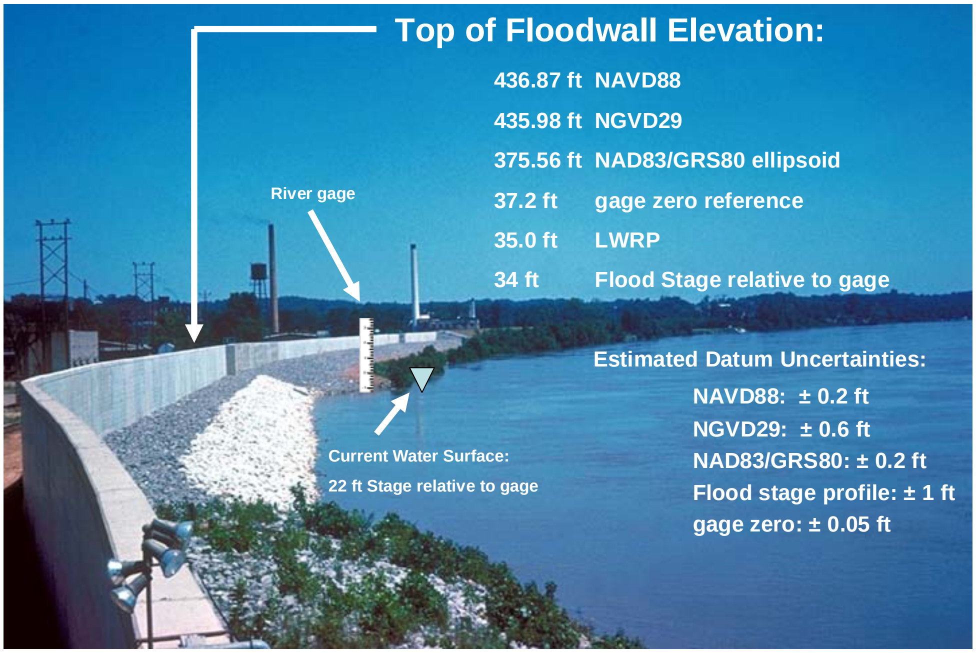


Figure 1-3. Datum and elevation uncertainties on an inland river system floodwall protection height. The reference datum uncertainties propagate to an uncertainty in the floodwall elevation. (see Chapter 9).

1-9. Federal Framework Systems for Referencing USACE Project Grades. It is important that USACE project control and elevation grades conform to the following federal framework systems and guidelines.

a. National Spatial Reference System (NSRS). The NSRS represents an independent framework system for long-term monitoring of the stability of project grades and flood protection elevations. This reference system is maintained by the NOAA National Geodetic Survey (NGS). In addition to USACE, this reference system has been adopted by most Federal agencies, including FEMA, USGS, EPA, and by most state transportation departments (DOT). The NSRS is a national reference framework that specifies latitude, longitude, height (elevation), scale, gravity, and orientation throughout CONUS and most civil works OCONUS locations. It is also the base reference for most GNSS observations. Accordingly, USACE must ensure flood risk management projects and navigation projects are referenced to this NSRS system. This insures consistency in reporting elevations or grades between agencies. In addition, incorporating Corps project control into the NSRS minimizes the need for maintaining independent databases at each District. It also ensures that Corps project control will be automatically updated when future updates to the NSRS are made. Permanent bench marks (PBM), or "*Primary Project Control Points*" (PPCP), on USACE projects shall be firmly connected to the NSRS and submitted to the NGS for inclusion in the published NSRS. Details on these procedures are explained in Chapter 3.

b. National Water Level Observation Network (NWLON). The NWLON is composed of the continuously operating long-term primary and secondary control tide stations established by the Center for Operational Oceanographic Products and Services (CO-OPS), an agency under the NOAA National Ocean Service. This network provides the basic foundation for the determination of tidal datums for coastal and marine boundaries and for chart datums in CONUS and OCONUS regions. The National Water Level Program (NWLP), also administered by the NOAA CO-OPS, includes the NWLON and water level elevation data and bench mark elevation data from historical long-term and short-term gages operated by that agency. NOAA tidal data are referenced to a specific National Tidal Datum Epoch (NTDE). The specific 19-year NTDE period adopted by NOAA is the official time segment over which tide observations are taken and reduced to obtain mean values (e.g., Mean Sea Level, Mean Lower Low Water, etc.) for tidal datums. A common period of observation is necessary because of periodic and apparent secular trends in sea level. Special NTDEs are adopted for local areas with extreme relative sea level change due to significant land subsidence (e.g., Louisiana) or land rebound (e.g., SE Alaska). CO-OPS computes special NTDEs based on recent 5-year modified tidal datum epochs.

c. FEMA National Flood Insurance Program (NFIP) Guidelines. FEMA has issued a number of publications dealing with flood mapping accuracy standards and related elevation datums that are needed for NFIP studies and to certify or accredit levee/floodwall systems. These certifications are referenced to Base Flood Elevations (BFE) shown on Flood Insurance Rate Maps (FIRMs). Design and constructed elevations on floodwalls, and related freeboard allowances, stillwater elevations, etc. must be consistent with the same regional vertical datums specified in NFIP regulations and guidelines—see "*Identification and Mapping of Special Hazard Areas*" (44 CFR 65) and FEMA's "*Guidelines and Specifications for Flood Hazard*

Mapping Partners" (FEMA 2003). FEMA Elevation Certificates require vertical datum designations for FIRM Base Flood Elevations and building first-floor elevations. Metadata associated with the origin of the datum (reference bench marks, FIRM, etc.) are critical in order to reliably relate FEMA BFEs to USACE floodwall protection elevations.

1-10. Implementation Actions. In accordance with ER 1110-2-8160, USACE commands need to ensure all project grade elevations or navigation depths are referenced directly or indirectly to the NSRS or NWLON framework systems described above. All newly authorized and existing projects should be evaluated to ensure that designed and constructed grades are adequately connected and referenced to the NSRS and the applicable tidal or hydraulic network. The hydraulic/tidal and geodetic vertical datum relationships must be assessed, developed and/or verified during the Feasibility and Preconstruction Engineering and Design (PED) phases, during construction, and periodically monitored after construction to account for subsidence, settlement, NOAA reference datum redefinitions and readjustments, sea level change, and other factors.

a. Critical project datum assessment items. Special attention should be made to assess the following critical issues associated with a project's vertical reference:

(1) Primary project control point bench marks. All projects shall have one or more permanent bench marks—i.e., a PPCP—that is directly connected to and published in the NOAA/NGS National Spatial Reference System (NSRS) network.

(2) Water level gage references. Permanent bench marks used to reference river, pool, reservoir, and tide gages shall be connected to and published in the NOAA National Spatial Reference System (NSRS) network.

(3) Protection grade elevations. Flood/hurricane protection structures and water control structure crest elevations shall be referenced to hydraulic flow or NWLON tidal models that are based on reliable water-level gage data that is referenced to the NSRS and reflects adjustments for sea level, settlement, or subsidence/uplift changes.

(4) Coastal navigation project grades. Coastal navigation project depths shall be defined relative to a local Mean Lower Low Water (MLLW) datum defined by the Department of Commerce; as required by Section 224 of the Water Resources Development Act of 1992 (WRDA 1992). This navigation reference datum shall be based on the latest tidal epoch. Depth measurements shall be spatially corrected based on hydrodynamic tidal models developed from and calibrated to up-to-date water-level gage data, and that field survey techniques are adequately compensating for short-term phase and slope variations in the water surface.

b. Corrective actions. Existing projects deemed deficient in any of the criteria outlined above will require corrective field survey actions. The amount of time and expense will vary considerably, depending on the geographical size of the project, risk assessments, the density and reliability of existing water level gages, and various other factors. Project engineers or managers should prepare a cost estimate in sufficient detail to allow programming the corrective action into the next budget cycle for the project. The guidance listed below synopsizes the effort required for various project conditions.

c. Geodetic control survey connections to the NSRS. At least one PPCP on every project must be geodetically connected to the NSRS. A variety of techniques for performing this connection are described in Chapter 3. In most cases, existing NSRS control in the region will suffice, and no significant field survey effort is required. In nearly all cases, PPCPs can be economically accomplished using Differential Global Positing System (DGPS) height transfer methods relative to the NSRS. Conventional differential leveling may be a more economical option, especially over short distances. PPCPs established or reestablished shall be submitted to NGS for inclusion in the NSRS.

d. Water level gage connections to the NSRS. Water level gages that are used to reference elevations of flood risk management, water control, or tidal parameters on navigation, water control, or HSPP projects must be referenced to and documented in the NSRS. A bench mark referenced to each gage shall be surveyed and placed into the NSRS and continuously maintained in that file. In some cases, this gage reference bench mark may serve as the PPCP for the entire project. Additional details are found in Chapter 4.

e. Coastal navigation project reference datums. Navigation projects in tidal areas that were not adequately updated to a current MLLW reference datum, or have outdated or unknown origin tidal modeling regimes (phase and range), or are on superseded tidal epochs, will require field efforts to update the project. This may require setting one or more short-term tidal gages to perform simultaneous comparison datum translations between an existing NWLON station and/or developing a tidal model utilizing NOAA hydrodynamic modeling techniques which can be applied to develop the MLLW datum relationship over a project reach. Minimizing tidal phase errors may require mandated utilization of GNSS differential carrier phase water surface elevation measurements in lieu of extrapolated gage elevations—i.e., Real Time Network (RTN) applications such as Real Time Kinematic (RTK), Virtual Reference Networks (VRN), or Virtual Reference Stations (VRS). Details on these methods are covered in Chapter 4.

f. Coastal navigation or Hurricane and Shore Protection Projects (HSPP) projects on non-standard or undefined tidal datums. Projects on antiquated or non-standard tidal datums must be converted or related to the MLLW datum established by NOAA used for coastal navigation and maritime charting in CONUS or OCONUS waters. This includes those projects that are still referenced to legacy datums such as Mean Low Water (MLW), Mean Gulf Level (MGL), Mean Low Gulf (MLG), Gulf Coast Low Water Datum, Old Cairo Datum 1871, Delta Survey Datum 1858, New Cairo Datum 1910, Mean Tide Level, Corps of Engineers Mean Low Water (COEMLW), U.S. Engineer Datum (USED), etc.

g. Mean Sea Level (MSL) or NGVD datums. Project control elevations or bench marks defined generically to "Mean Sea Level" or "NGVD" without any definitive source data (metadata) probably have no firmly established relationship to the current NSRS and may need to be resurveyed. "NGVD29" was once known as the "Sea Level Datum of 1929." However, neither NGVD29 nor the current NAVD88 datums are equivalent to "mean sea level." Resurveying will entail establishing a hydraulic and a NSRS geodetic reference, as applicable. Details are outlined in Chapter 3.

h. Permanent bench mark control requirements for extensive flood risk management or reservoir projects. Levee projects encompassing large geographic extents may require more than one PPCP to cover the project area. PPCPs should be added as necessary to control the project grades and features using conventional surveying methods, or preferably at a sparser density needed to accommodate GPS real-time kinematic construction survey or machine control methods. These permanent bench marks must be firmly connected to applicable hydraulic gages and regional NSRS datums as described above, and, where required, should be submitted to NGS for inclusion into the NSRS. Requirements for additional NSRS densification on large flood risk management or water control reservoir projects are covered in Chapter 6.

i. Projects subject to high subsidence rates. Projects located in high subsidence areas (e.g., portions of Louisiana, Texas, and California) may require special attention. This also applies to areas on the Northwest coast (e.g., Alaska) and other locations that may be subject to crustal uplift or glacial rebound. Vertical elevations of reference bench marks, water level gages, and protection structures must be continuously monitored for movement and loss of protection. This monitoring can be accomplished using static GPS survey methods or conventional differential leveling. In high subsidence areas, independent local vertical control networks referenced to the NSRS may be established for these purposes. These vertical networks are periodically resurveyed at intervals dependent on subsidence rates. For example, in the New Orleans, LA area, primary control PBMs on these monitoring networks are date-stamped to signify reobservation/readjustment epochs—e.g., "BM XYZ (2004.65)." Additional technical guidance for monitoring subsidence or uplift can be obtained from the USACE Army Geospatial Center (AGC) and the NGS. Details are covered in Chapter 8.

j. Ecosystem restoration projects. In aquatic ecosystem restoration projects, the appropriate vertical datum should be taken into consideration during all project phases. Restoration projects should be based on valid water level measurements, and in some cases, the current NSRS geodetic datum. Often vertical datums for restoration projects need to establish the relationship between historical and current water level and geodetic datums to ensure the ecosystem restoration success. For ecosystem restoration projects in coastal areas, see Chapter 5 for details in defining the appropriate water level datum. See Chapter 6 for details in defining the appropriate datum to be used for non-coastal ecosystem restoration projects.

k. Regulatory permitting actions. Compensatory mitigation projects or regulatory permitting activities that are referenced to tidal or non-tidal datums should be defined to an established datum based on valid water level observations, as appropriate to local, state, and federal statutory requirements. Statutory Mean High Water (MHW), High Tide Line (HTL), etc. boundary demarcations in coastal areas may, in some cases, require direct reference to NOAA NWLON gage networks. Refer to details in Chapter 7.

1-11. Periodic Reassessments of Controlling Reference Elevations. Periodic reevaluations of project reference elevations and related datums covered in this manual should be included as an integral component in the various civil works inspection programs of completed projects. The frequency that these reevaluations will be needed is a function of estimated magnitude of geophysical changes that could impact flood protection or navigation grades. Project elevations and dredging grades that are referenced to tidal datums will have to be periodically coordinated with and/or reviewed by NOAA to ensure the latest tidal hydraulic effects are incorporated and that the project is reliably connected with the NSRS. In all cases, a complete reevaluation of the vertical datum should be conducted at each scheduled periodic inspection. Shallow-draft navigation projects may have different criteria. Any uncertainties in protection levels that are identified during the inspection will also need to be incorporated into any applicable risk/reliability models developed for the project—see EM 1110-2-1619 (*Risk Based Analysis for Flood Damage Reduction Studies*).

1-12. Metrics and Accuracy Definitions. Both English and metric units are used in this manual. Elevations, depths, and gage data for USACE civil works projects are expressed in English units, following local engineering and construction practices. Exceptions may exist on some OCONUS projects. GNSS observation data and standards are normally in metric units. NOAA geodetic data and water level gage data are published in English and/or metric units. Satellite-derived geographical or Cartesian coordinates are transformed to English units for use in local project reference and design systems, such as State Plane Coordinate System (SPCS) grids or local construction chainage-offset systems. English/metric equivalencies are noted where applicable, including the critical—and often statutory—distinction between the US Survey Foot (1,200/3,937 meters (m) exactly) and International Foot (30.48/100 m exactly) conversions. One-dimensional (1D), two-dimensional (2D), and three-dimensional (3D) accuracy or uncertainty statistics, standards, and tolerances specified in this manual are defined at the 95% RMS confidence level. Cost estimates cited in this manual are in 2010 dollars.

1-13. Trade Name Exclusions. The citation or illustration in this manual of trade names of commercially available products, including supporting surveying equipment, instrumentation, and software, does not constitute official endorsement or approval of the use of such products.

1-14. Abbreviations and Acronyms. Abbreviations and acronyms used in this manual are listed in the Glossary.

1-15. Manual Development, Technical Assistance, and Training. Technical guidance in this manual was developed by the Army Geospatial Center (AGC) in conjunction with the U.S. Army Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory (CHL), the NOAA National Geodetic Survey (NGS), the NOAA Coast Survey Development Laboratory (CSDL), and the NOAA Center for Operational Oceanographic Products and Services (CO-OPS). The AGC (National Datums and Subsidence Program) may be contacted for detailed technical guidance and formal training in evaluating the adequacy of existing project reference datums and survey techniques needed to connect project control with the NSRS. Reference http://www.agc.army.mil/ndsp.

1-16. Proponency and Waivers. The HQUSACE proponent for this manual is the Engineering & Construction Community of Practice. Waivers to this guidance should be forwarded through MSC to HQUSACE (ATTN: CECW-CE).

CHAPTER 2

Geodetic, Tidal, and Hydraulic Reference Datums Used to Define Project Grades on Civil Works Projects

2-1. Purpose. This chapter provides a technical overview of the interrelationship between geodetic (i.e., orthometric) and hydraulic datums that are used to reference various civil works projects. These datums are the baseline reference for designing protection elevations of levees and related water control structures, and the design depths of navigation projects. They are also used for setting grades during project construction and maintenance operations. The Corps uses a variety of orthometric and water level datums to reference coastal and inland navigation projects. Figure 2-1 provides a generalized illustration of the various references used on coastal and inland navigation projects in tidal, free flow, and controlled regimes. These same orthometric and hydraulic datums are also used to reference elevations of inland levees and dams.

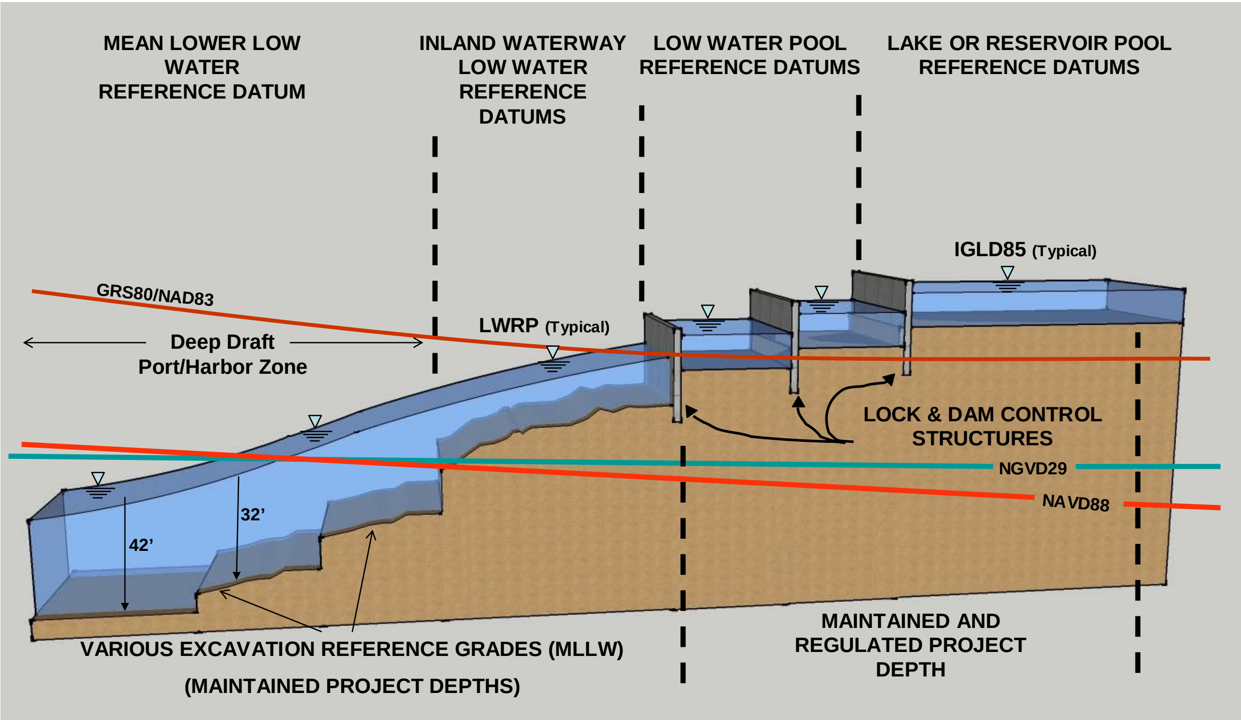


Figure 2-1. Tidal and inland vertical reference datums.

2-2. Geodetic and Hydraulic Vertical Reference Systems**.** The following paragraphs provide a general overview of the types of vertical datums used to define the elevations of USACE flood risk management, water control, HSPP, and navigation projects. Appendix B provides additional information on terrestrial and extraterrestrial geodetic datums and related 2D and 3D coordinate systems. For a more comprehensive discussion on geodesy, vertical datums, and GNSS height measurement, refer to the following publications: "*Physical Geodesy*" (Hofmann 2006) or "*What Does Height Really Mean*" (Meyer 2006).

a. Definition of geodetic datum. The "*Geodetic Glossary*" (NGS 1986) defines a geodetic datum as:

"*A set of constants specifying the coordinate system used for geodetic control, i.e., for calculating the coordinates of points on the Earth*."

(1) The above general definition applies to both horizontal and vertical datums. Vertical datums are normally defined relative to the fixed elevation of some point(s) on the Earth, or in the case of satellite-based ellipsoidal reference systems, a three-dimensional coordinate of a point near the mass center of the Earth.

(2) A more comprehensive definition of vertical datums that directly relates to studies of coastal and inland flooding, and USACE flood risk management project elevations, is outlined in the following excerpts from "*Mapping the Zone: Improving Flood Map Accuracy*" (NRC 2009):

"The data components of a flood study that involve a measurement of height or elevation can be grouped into four general categories:

(a) Elevation reference surface. Before elevation can be measured or the data used in engineering analysis, a measurement system must be established. The location of "zero" and a physical reference for elevation zero (in other words, a vertical datum) must be established on the Earth, where it can be used for all types of height measurements.

(b) Base surface elevation. Two types of base surfaces are important to flood studies: land surface elevation (topography) and its underwater equivalent (bathymetry). Topography is expressed as the height of a location above the geodetic datum and is in most cases a positive value. Bathymetry is expressed as the depth of the land surface below rivers, lakes, and oceans; positive depth is equivalent to negative elevation.

(c) Water surface elevation. The depth of water in rivers, lakes, and streams and the point at which water overtops their banks and spreads across the landscape are the subjects of riverine flood studies. The depth of water in the ocean and the impact of extreme events such as hurricane-induced storm surge or earthquake-induced tsunamis are the subjects of coastal flood studies. The height of water surfaces is measured with stream and tide gages. The location and elevation of the gages themselves must be determined accurately in order to correctly relate water surface measurements to other elevations.

(d) Structure elevation. The vulnerability of buildings and infrastructure to flood damage is directly related to their location with respect to the floodplain and the elevation and orientation of critical structural components with respect to the height of potential floodwaters. In addition, structures within the floodway (such as bridges, dams, levees, and culverts) influence the conveyance of water in a stream channel during a flood event, affecting flood heights."

b. Orthometric elevations and vertical datums. Floodwall and levee protection heights (and related inundation mapping elevations) are normally referenced to "orthometric" vertical datums established by the National Geodetic Survey. Orthometric vertical datums provide a design and construction framework for referencing tidal gages and hydraulic models over a region. Orthometric datums are based on geopotential or equipotential surfaces referenced to some defined terrestrial origin—e.g., a tide gage PBM or the geoid. The two major orthometric datums used in CONUS include:

(1) National Geodetic Vertical Datum of 1929 (NGVD29)—superseded in the early 1990s.

(2) National Geodetic Vertical Datum of 1988 (NAVD88).

Older water control projects may be referenced to superseded vertical datums—e.g., MSL 1912. NAVD88 is the current federally recognized elevation reference system throughout CONUS. Neither NGVD29 nor NAVD88 are true "sea level datums." They are not equivalent to "Local Mean Sea Level" in CONUS. For example on the West Coast, NAVD88 is about 3 ft below LMSL. Failure to account for these orthometric datum anomalies in the design of a floodwall elevation can have significant adverse consequences since orthometric height differences in NAVD88 only approximate actual energy head differences. A redefinition of the NAVD88 is anticipated on or after 2018. This revision using gravity observations is expected to better approximate actual dynamic head elevation differences.

c. Hydraulic datums. These datums are found on inland river, lake, or reservoir systems, typically based on a low water pool or discharge reference point. Examples are the Mississippi River Low Water Reference Plane (e.g., LWRP74, LWRP93, or LWRP07) and the International Great Lakes Datum (IGLD55 or IGLD85). Low water reference planes may be defined relative to 97% discharge flows at river gages, from which river stage elevations are derived. River gage stage elevations are usually referenced to an orthometric datum such as NGVD29 or NAVD88. Hydraulic-based datums in inland waterways directly reference flood profiles and related elevations of flood protection levees or floodwalls and navigation clearances. A variety of inland reference planes are used in controlled pools and reservoirs, such as normal pool level, minimum regulated pool level, and flat pool level, low flow regulation pool, seasonal pool, and conservation pool. Dynamic height differences are often used in relating hydraulic datums. Dynamic heights, unlike orthometric heights, represent geopotential energy (hydraulic head) gradients in water surfaces (canals, rivers, lakes, reservoirs, hydropower plants, etc.) and thus may have application to Corps hydraulic models.

d. Tidal datums. Tidal datums are used throughout all USACE coastal areas and are based on long-term water level averages of a phase of the tide. Mean Sea Level (MSL) datum (or more precisely Local Mean Sea Level--LMSL) is commonly used as a base reference for hydrodynamic storm modeling, wind and wave surge modeling, high water mark observations, stillwater surge elevations, and design of coastal hurricane protection structure elevations. The relationship between these water elevations and the orthometric datum elevation varies spatially and must be computed or modeled. Depths of water in coastal navigation projects in the United States are usually defined relative to Mean Lower Low Water (MLLW) datum, but sometimes a local legacy datum is used. In isolated non-tidal coastal areas (e.g., Pamlico Sound, NC and Laguna Madre, TX) NOAA uses a Low Water Datum (LWD) as a chart datum. Tidal datums are essentially local datums (i.e., LMSL) and should not be extended more than a few hundred feet from the defining gage without substantiating measurements or models. It is essential that the vertical datum plane used in these models use the geoid (or other equipotential surface) and not a geometric plane surface. Tidal datums are periodically updated by NOAA and thus are defined by their National Tidal Datum Epoch (NTDE)—currently 1983-2001.

e. Satellite or space-based datums. Satellite datums are three-dimensional, geocentric, ellipsoidal datums used by Global Navigation Satellite Systems (GNSS), such as the Global Positioning System (GPS)—e.g., ITRF and WGS84. The reference point for these systems is the estimated mass center of the earth. Ellipsoid heights of points in CONUS represent elevations relative to the NAD83/GRS80 mathematically defined ellipsoid. These ellipsoid heights can approximately be transformed to a NAVD88 orthometric elevation using a gravity (geoid) model developed by the National Geodetic Survey. This geoid model approximates the current NAVD88 orthometric reference surface in CONUS.

f. Local or legacy datums. Most USACE civil projects are, in effect, referenced to a local vertical datum. Nearly all construction, boundary, and real estate surveys are aligned to local horizontal datums, e.g., section corners, property corners, road intersections, chainage-offset construction layout, etc. Many local datums are based on arbitrary, unknown, or perhaps archaic origins. Vertical construction datums are often referenced to an arbitrary elevation at a PBM (e.g., elevation 100.00 ft). Some local datums with designated origins may be at distant points from a project—e.g., New Cairo Datum (Cairo, Illinois) projected south to the Gulf Coast in Louisiana. Most hydraulic-based river datums and MSL/MLLW tidal datums are actually local datums when they are not properly modeled or kept updated. Other local or legacy datums encountered in USACE may include Mean Low Gulf (MLG), Mean Gulf Level (MGL), Mean Low Tide (MLT), Old/New Memphis Datums, and Delta Survey Datum. USACE PBMs, TBMs, and floodwall elevations referenced to NGVD29 must be considered a local datum in that relationships to the national NSRS network are no longer maintained. Height differences between points determined from differential leveling are effectively local datums unless orthometric or dynamic height corrections are applied to the observed elevation differences. Geospatial coordinates for such points are primarily used for external reference, such as GIS mapping.

2-3. <u>Background on the Definition of the National Geodetic Vertical Datum of 1929 (NGVD29)</u>. Since 1929, only two official national vertical datums have been established—NGVD29 and NAVD88. Prior to 1929, the reference surface for a vertical datum has been some approximation of local mean sea level, but this was not a strict requirement. By 1900, the vertical control network for the United States had grown to 21,095 km of geodetic leveling. A reference surface was determined in 1900 by holding elevations referenced to local mean sea level (LMSL) fixed at five tide stations. Data from two other tide stations indirectly influenced the determination of the reference surface. Subsequent readjustments of the leveling network were performed by the US Coast and Geodetic Survey (USC&GS) in 1903, 1907, and 1912.

a. The first of these national datums was the Sea Level Datum of 1929 (SLD29). SLD29 was created by the US Coast and Geodetic Survey (USC&GS) as the datum to adjust all vertical control to in North America. The SLD29 is defined by 26 tide stations (held fixed to Local Mean Sea Level)—21 tide stations in the United States and five tide stations in Canada. When it was established in 1929, SLD29 was believed to be a "mean sea level" datum although mean sea level was not the same at each gage. Mean sea level was not developed using the same epoch or period of record at each of the gages. Each gage was, in effect, a "local mean sea level" (LMSL) reference datum. However, over time, with sea level rise and other factors, it was no longer considered a "mean sea level" datum. In 1973, the name of SLD29 was changed to the National Geodetic Vertical Datum of 1929 (NGVD29).

b. In 1929, the international nature of geodetic networks was well understood, and Canada provided data from its first-order vertical network to combine with the US network. The two networks were connected at 24 locations through vertical control points (bench marks) from Maine/New Brunswick to Washington/British Columbia. Although Canada did not adopt the "Sea Level Datum of 1929" determined by the United States, Canadian-US cooperation in the general adjustment greatly strengthened the 1929 network. Table 2-1 lists the kilometers of leveling involved in the readjustments and the number of tide stations used to establish the datums.

Table 2-1. Legacy Vertical Datums in CONUS. (IPET 2007)

Year of Adjustment	Kilometers of Leveling	Number of Tide Stations
1900	21,095	5
1903	31,789	8
1907	38,359	8
1912	46,468	8
1929	75,159 (U.S.) 31,565 (Canada)	21 5

c. Holding Local Mean Sea Level (LMSL) heights fixed at these tide stations did not mean that the geodetic vertical datum and the LMSL were the same at any location outside of the 26 tide gages. Immediately after the 1929 adjustment, the relationship between NGVD29 and LMSL began to deviate due to apparent sea level change. NOAA updated LMSL and MLLW datums in the US with every change in National Tidal Datum Epoch (NTDE) starting with 1941-1959, 1960-1978, and 1983-2001. NGVD29 was not adjusted to account for sea level change during this time period. There were several later adjustments to the NGVD29 datum, but no change in the national geodetic datum until 1991, when NGS established the NAVD88. Adjustments to the datum are noted by the year (or epoch) in parentheses after the datum name, i.e., NGVD29 (19xx) where 19xx is the year the NGVD29 datum was readjusted in a region or local area based on either new data or releveling of an existing level line. It is noted that this is only an adjustment of the network and not a new datum.

Additionally, the local mean sea level at each of the 26 gages did not lie on the same equipotential surface due to local factors such as prevailing winds, ocean currents, etc. These differences were introduced into the national network as errors during the network adjustment.

2-4. Background on the Definition of the Current North American Vertical Datum of 1988 (NAVD88). Unlike the multiple points which define the "zero level" of NGVD29, NAVD88 is defined by a single tidal bench mark at Father Point/Rimouski, an International Great Lakes Datum of 1985 (IGLD85) water level station at the mouth of the Lower St. Lawrence River, in Quebec, Canada. Its elevation was held fixed in a minimally constrained, least squares adjustment, which is not distorted by constraints of local mean sea level in different areas, as in NGVD29. The warping of NGVD29's reference surface means that the heights determined in that datum are not strictly "orthometric." Conversely, NAVD88's reference surface is equipotential, and therefore heights in that datum are nearly orthometric. The reason they are not truly orthometric is that the reference surface of NAVD88 was not specifically chosen as the geoid. In fact, most estimates of the difference between the NAVD88 reference surface and the geoid put the difference at the level of a few decimeters.

a. In support of NAVD88, the NGS converted the historic height difference links involved in the 1929 general adjustment to computer-readable form. The 1929 general adjustment was recreated by constraining the heights of the original 26 coastal stations. Free-adjustment results were then compared with the general adjustment constrained results. Several differences exceeded 50 cm. A large relative difference, 86 cm, exists between St. Augustine, Florida, and Fort Stevens, Oregon. This is indicative of the amount of distortion present in the 1929 general adjustment.

b. NAVD 88 combined 1,300,000 kilometers of leveling surveys held in the NGS data base, into a single least squares adjustment to provide users with improved heights for over 500,000 vertical control points distributed throughout the United States, on a common datum. There had been approximately 625,000 km of leveling added to the National Geodetic Reference System (NGRS) since NGVD29 was created. (The NGRS has been superseded by the NSRS). An extensive inventory of the vertical control network resulted in the identification of lost bench marks, several affected by crustal motion associated with earthquake activity, postglacial rebound (uplift), and subsidence. Other problems (distortions in the network) were caused by forcing the 625,000 km of leveling to fit previously determined NGVD29 height values. Some observed changes, amounting to as much as 9 m, were noted.

c. The NAVD88 datum adjustment formally began in October 1977 with releveling much of the first-order NGS vertical control network in the United States. The nature of such a network required a framework of newly observed height differences to obtain realistic, contemporary height values to form the readjustment. To accomplish this, NGS identified 81,500 km (50,600 miles) for releveling to be completed by NGS field crews. In addition to the NGS releveling, other federal agencies such as the USACE, many state agencies such as state Departments of Transportation, Departments of Natural Resources, etc. provided NGS with approximately another 20,000 km (32,400 miles) of new and releveled surveys. Replacement of disturbed and destroyed monuments preceded the actual leveling. This effort also included the establishment of "deep-rod" bench marks, which provided reference points for future "traditional" and GPS leveling techniques. Field leveling of the 81,500 km network, and the 20,000 km submitted by state agencies, was accomplished to Federal Geodetic Control Committee (FGCC) First-Order, Class II specifications, using the "double-simultaneous" method. NGS worked closely with both Canada and Mexico to ensure sufficient connections were made along both borders of the United States. NGS field crews also worked closely with both countries to carry the vertical control into both countries and make connections to their vertical network and both countries ran into the United States making connections. Both Canada and Mexico provided NGS with their leveling data so that the NAVD88 would be more extensively “North American” than NGVD29 had been. The general adjustment of NAVD88 was completed in June 1991.

d. The leveling observations used in NAVD88 were corrected for rod scale and temperature, level collimation, and astronomic, refraction, and magnetic effects. All geopotential differences were generated and validated, using interpolated gravity values based on actual surface gravity data. Geopotential differences were used as observations in the least-squares adjustment, geopotential numbers were solved for as unknowns, and after the adjustment was complete, orthometric heights were computed using the well-known Helmert height reduction. The weight of an observation was calculated as the inverse of the variance of the observation, where the variance of the observation is the square of the a priori standard error multiplied by the kilometers of leveling divided by the number of level sections.

2-5. <u>Satellite-Based Vertical Reference Systems</u>. The current and rapidly expanding use of satellite-based ellipsoidal reference systems provides a mechanism for establishing an external reference framework from which orthometric, tidal, hydraulic, and local vertical datums can be related spatially and temporally. GPS along with other expanding global navigation and positioning systems (i.e., GNSS or Global Navigation Satellite Systems) such as GLONASS (Russian Federation), Galileo (European Community), Compass (China), etc. will further the use of satellite-based systems as the primary measurement reference for project elevations, dredging grades, machine control, and related mapping applications. Various initiatives are underway by NOAA, FEMA, and other agencies to refine the models of some of the various vertical datums listed above—resulting in a consistent National Spatial Reference System that models and/or provides transformations between the orthometric, tidal, and ellipsoidal datums. Paramount in these efforts is the NOAA "National VDatum" project that is designed to provide accurately modeled transformations between ellipsoid-based reference systems, orthometric datums, and tidal datums.

a. NAD83/GRS80 ellipsoid. In the near future, most engineering and construction site control (including GPS/GNSS-based machine control systems) will be referenced using various DGPS/RTN techniques; therefore, it is essential that all USACE project elevations be referenced to the NAD83/GRS80 ellipsoid so that the relationship between this ellipsoidal datum and local hydraulic/tidal datums is firmly established. Most federal agencies, including USACE in EM 1110-1-1003 (*NAVSTAR GPS Surveying*), have specifications for measuring and defining vertical elevations derived from satellite-based measurements.

b Ellipsoid and orthometric heights. In recent years much emphasis has been put on the determination of orthometric heights from GPS ellipsoid height measurements, as shown in

Figure 2-2. In this figure the "plumb line" is the curved line between a point on the earth's surface and a point on the geoid, everywhere tangent to the direction of gravity (or, in other words, everywhere perpendicular to all equipotential surfaces through which the line passes). The "orthometric height" is exactly the distance along this curved plumb line between the geoid and point on the earth's surface. Close approximations can be made, but for absolute accuracy, gravity needs to be measured along this line, requiring a bored hole, which is impractical. For general geodetic control survey applications in USACE, GPS height difference observational accuracy can equal or exceed traditional (leveling) observations. As illustrated in Figure 2-2, with a highly accurate model of the geoid (geoid height—"*N*"), the purely geometric ellipsoidal height "*h*" determined by GPS can be transformed into an orthometric height "*H*"—e.g., NAVD88.

$$h \approx H + N \quad or \quad H \approx h - N \qquad \text{Eq. 2-1}$$

where,

h = Ellipsoid Height (NAD83/NSRSxx)
H = Orthometric Height (NAVD88)
N = Geoid Height (GEOIDxx)

c. Ellipsoid and geoid heights. The ellipsoid surface has nothing to do with the level surfaces and it cuts through all level surfaces because it is not a function of the Earth's gravity field. Therefore GPS-derived ellipsoid heights are not related to the geoid or the gravity field; thus requiring a model to obtain differences between the geoid and ellipsoid to determine orthometric height. Geoid height (also termed geoid separation or geoid undulation) is the difference between the geoid and ellipsoid at any given point on the earth's surface. The geoid height is always negative in CONUS (as shown in Figure 2-2). The term "equipotential surface" is defined as an irregular surface, whose gravity potential energy is constant at every point. By extension, therefore the force of gravity is perpendicular to an equipotential surface at every location on that surface. Because the value of gravity potential energy can be any number (corresponding to one equipotential surface), there are therefore an infinite number of equipotential surfaces surrounding the Earth with each equipotential surface lying either completely within or completely without another surface; they do not intersect one another. Due to the non-homogenous distribution of Earth's masses, each of these surfaces has its own distinct shape.

d. Geoid and local mean sea level. The "geoid" is the one equipotential surface which most closely fits global mean sea level in a least squares sense. However variations between local mean sea level and the geoid at one location may be radically different from such variations at another location. As an example the LMSL-geoid difference in New Orleans is not the same as LMSL-geoid difference in Miami, Florida since the geoid is fit to global mean sea level and its definition is therefore not strongly influenced by the local hydrodynamic phenomena which affect local mean sea level. In the absence of all forces besides gravity, the ocean surface would lie on the geoid. However tides, currents, river runoff, wind, circulation, and other forces all impact sea level. The effects of these forces do not average to zero over time, and since these forces vary from site to site, any given tide gage may determine local mean seal level but not directly determine the geoid. Due to this difference in variations between the geoid and local mean sea level, and the fact that 26 tide stations were held fixed in establishing NGVD29, the NGVD29 reference surface was warped to allow the local mean sea level at tide stations to define the "zero elevation" of heights in the NGVD29 datum; hence, NGVD29's reference surface is not equipotential.

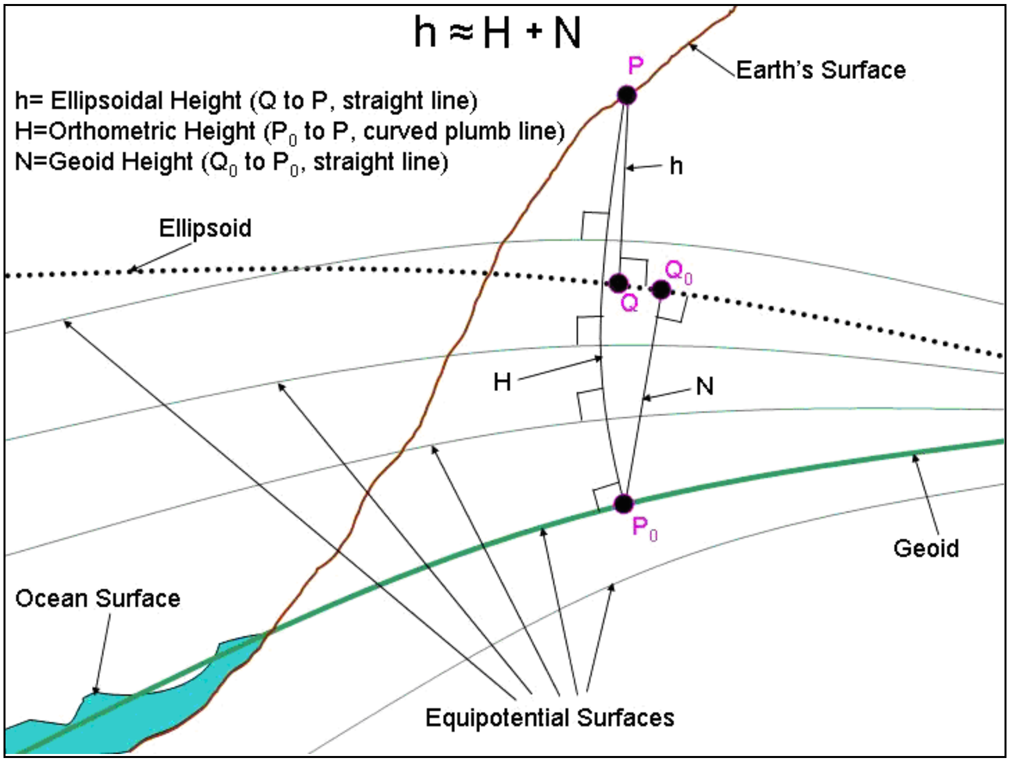


Figure 2-2. Relationship between the ellipsoid, geoid and orthometric heights. (IPET 2007)

e. Satellite height and leveled height differences. Over short distances, satellite derived elevation differences are not as accurate as those that are determined from traditional spirit or digital leveling observations. Therefore, traditional spirit leveling will normally be used for local construction stakeout. Satellite (RTN) observations are acceptable for detailed site plan design.

f. Orthometric height and dynamic height differences. Leveled elevation differences between bench marks do not yield either orthometric height differences or dynamic height differences. Spirit or digital leveling differences in elevation must be corrected to obtain orthometric heights or dynamic heights—i.e., orthometric corrections or dynamic corrections—see Hofmann 2006. Orthometric corrections, being a function of a level line length and direction, are usually negligible for engineering purposes. Dynamic height corrections are usually negligible except in high elevation (energy head) differences, such as those occurring between a hydroelectric power plant's reservoir intake structure and the lower spillway. Due to inaccuracies in NAVD88 leveling adjustments, a "hydraulic corrector" must be applied at subordinate points on the Great Lakes in order to obtain a reference engineering, construction, or navigation datum. These hydraulic correctors are published by the IJC Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data (IJC 1995).

g. GPS/GNSS reference frames. Navigation satellite orbits are computed from data collected by a global network of receivers coordinated by the International GNSS Service for Geodynamics (IGS). The accuracy of the GPS orbits depends on many factors, including the accuracy of the coordinates of the data collection sites. The earth's surface is not fixed and rigid like an eggshell. It consists of many sections, or plates, which move slowly over time in various directions and rates in a process called crustal motion. Scientists have been studying this movement for several reasons. This includes trying to determine where land masses are with respect to one another and where they will be in the future. IGS monitoring sites are located on these crustal plates. The International Earth Rotation Service (IERS) periodically computes the positions of the sites for a given date. The sites define the IERS, the International Terrestrial Reference Frame (ITRF), and the date defines the epoch. The IERS also computes the movement velocities of the sites to estimate where the sites will be in the "near" future with some degree of accuracy. The ITRF is an internationally accepted standard, and is the most accurate geocentric reference system currently available. The longer the sites operate, the better the positions and velocities can be determined and the more accurate the satellite orbits will be.

2-6. Tidal Datums Used to Reference Coastal HSPP and Navigation Projects. Tidal datums are used to establish local tidal phase averages as reference levels from which to reckon flood/hurricane protection structure heights or depth measurements in a navigation project. One of these tidal averages is the mean level of the water surrounding the gage. Observations are taken from a tide gage that has been collecting data for a period covering a 19-year National Tidal Datum Epoch period—NTDE. This time period allows inclusion of all practical variations in the path of the moon about the earth and the earth about the sun. Tidal datums are locally derived and must not be extended into areas that have differing hydrographic characteristics, without substantiating measurements. The most commonly used tidal datums are:

a. Mean Sea Level (MSL) or Local Mean Sea Level (LMSL). The average height of the surface of the sea at a tide station for all stages of the tide, typically covering a 19-year period which is usually determined from hourly height readings measured from a fixed and predetermined reference level. Most USACE coastal hydrodynamic modeling and design is referenced to MSL.

b. Mean Lower Low Water (MLLW). The average height of the lower of the two low waters occurring in a day, at a tide gage over a 19-year period. Most CONUS, and some OCONUS, coastal navigation projects are referred to this datum. This datum superseded Mean Low Water (MLW) that was previously used as the navigation reference datum for the East Coast CONUS. Some HSPP and navigation projects may still be referenced to a legacy MLW datum.

c. Mean High Water (MHW). The average height of all high waters at a tide station, covering a 19-year period. Heights of bridges over navigable waterways and legal coastal shoreline boundaries are referred to this datum. Likewise are legal shoreline boundaries in many jurisdictions (variations of MHW are outlined in Chapter 7). Coastal shorelines shown on navigation charts depict MHW whereas depths on the same chart are referred to Mean Lower Low Water. Exceptions to this are found on USACE inland navigation charts.

d. Mean Tide Level (MTL). Sometimes termed half-tide level, a plane often confused with LMSL that lies close to LMSL. MTL is the midpoint plane exactly between the average of MHW and MLW at a tide station. MTL does not include all the tide levels (i.e., MHHW and MLLW). Hydraulic design manuals sometimes erroneously refer to MTL as being synonymous with Mean Sea Level.

Additional details on these tidal reference planes are covered in Chapters 4 and 7, and in NOAA technical publications listed at Appendix A.

2-7. Geodetic and Hydraulic Datum References on USACE Projects. A variety of vertical reference systems are used on Corps flood protection and navigation projects. Of significance is the local relationship between the terrestrial or orthometric datums (NAVD88) and the hydraulic datums (MSL, LMSL, MLLW, LWRP, Normal Pool, etc.) from which flood protection heights and navigation grades are modeled and designed. During the detailed design or evaluation of flood protection or HSPP projects, outdated or superseded geodetic datums (e.g., NGVD29, MSL 1912) should be referenced to the federal NSRS datum—e.g., NAVD88. Likewise, outdated riverine, pool, reservoir, lake, or tidal reference planes should be referenced to the current NSRS datum.

a. CONUS and OCONUS reference datums. References throughout this guidance to the NSRS (NAVD88) are applicable only to the current vertical adjustment in the CONUS. NOAA has established independent vertical datums (orthometric or tidal) for some OCONUS locations—e.g., Puerto Rico (PRVD02), Guam (GUVD04), US Virgin Islands (St. Thomas--VIVD09). Other OCONUS locations may have local tidal datum references—see Table 2-3. CONUS projects can be referenced to the NAD83/GRS80 ellipsoid and NAVD88. All OCONUS locales can be globally referenced to the WGS84 ellipsoid and local NWLON tidal gage, as applicable. Geodetic reference datums for most CONUS and OCONUS project areas are outlined in Tables 2-2 and 2-3.

Table 2-2. Geodetic and Hydraulic Datum References for USACE Flood Risk Management, Navigation, and HSPP Projects (CONUS).

Project Location	Geodetic Reference Horizontal	Geodetic Reference Vertical	Hydraulic/Tidal Reference
Inland rivers, pools, reservoirs, etc.	NSRS (NAD83)	NSRS (NAVD88)	USACE modeled LWRP LWRP or pool
Coastal (tidal waters) Navigation projects HSPP projects	NSRS (NAD83)	NSRS (NAVD88)	NOAA NWLON (NTDE) hydrodynamically modeled Local Mean Sea Level (LMSL) or MLLW (navigation projects) between NWLON stations
Great Lakes	NSRS (NAD83)	NSRS (IGLD85) [1]	NOAA NWLON hydrodynamically modeled local IGLD85

[1] A separate IGLD85 datum is specified for Lakes Ontario, Erie, Huron, Michigan, and Superior.

Great Lake	IGLD85 Chart Datum	Ordinary High Water Mark (Section 10)
Superior	601.1 ft	603.1 ft
Michigan-Huron	577.5	581.5
St. Clair	572.3	576.3
Erie	569.2	573.4
Ontario	243.3	247.3

Hydraulic slopes are specified for connecting channels (e.g., Detroit River, St. Clair River, and St. Marys River, Niagara River, St Lawrence River). Additional details on IGLD are in Chapter 6.

Table 2-3. Geodetic and Hydraulic Datum References for USACE Flood Risk Management, Navigation, and HSPP Projects (OCONUS).

Project Location	Geodetic Reference Horizontal	Geodetic Reference Vertical	Hydraulic/Tidal Reference
Alaska	NSRS (NAD83)	NSRS (NAD83/GRS80) local datums or NAVD88	NWLON tide gages (LMSL/MLLW)
Puerto Rico	NSRS (NAD83)	PRVD02 [1]	Gage PBM 975 5371A (San Juan)
U.S. Virgin Islands:			
St. Thomas	NSRS (NAD83)	VIVD09 [1]	Gage 975 1639 F (Charlotte Amalie)
St. Croix	NSRS (NAD83)	VIVD09	Gage 975 1401 M (Lime Tree Bay)
St. Johns	NSRS (NAD83)	VIVD09	Gage 975 1381 Tidal 2 (Lameshur Bay)
Hawai'i			
Kaua'i	NSRS [2] (NAD83)	Local Tidal MLLW	Gage 161 1400 (Nawiliwili) Tidal 5
O'ahu	NSRS (NAD83)	Local Tidal MLLW	Gage 161 2340 BM 8
Moloka'i	NSRS (NAD83)	Local Tidal MLLW	Gage 161 3198 Tidal 10
Lana'i	NSRS (NAD83)	Local Tidal MLLW	(Univ of Hawaii gage-Kaumalapau)

Table 2-3 (Continued). Geodetic and Hydraulic Datum References for USACE Risk Management, Navigation, and HSPP Projects (OCONUS).

Project Location	Geodetic Reference Horizontal	Geodetic Reference Vertical	Hydraulic/Tidal Reference
Hawai'i (Contd):			
Maui	NSRS (NAD83)	Local Tidal MLLW	Gage 161 5680 BM A
Hawai'i	NSRS (NAD83)	Local Tidal MLLW	Gage 161 7760 Tidal 4
Kaho'olawe	NSRS (NAD83)	Local Tidal "MLL"	n/a ["Mean Lower Low" datum]
Ni'hau	NSRS (NAD83)	Local Tidal MLLW	n/a [3]
American Samoa:			
Tutuila (Pago Pago)	NSRS (83) HARN 2002	American Samoa Vertical Datum of 02	Gage PBM 177 0000 PBM S
Tau	NSRS (83) HARN 2002	American Samoa Vertical Datum of 02	n/a
Aunuu	NSRS (83) HARN 2002	American Samoa Vertical Datum of 02	n/a
Ofu	NSRS/ HARN 02	USGS 1963	n/a
Rose	NSRS/ HARN 02	USGS 1963	n/a
Olosega	NSRS/ HARN 02	USGS 1963	n/a

Table 2-3 (Continued). Geodetic and Hydraulic Datum References for USACE Risk Management, Navigation, and HSPP Projects (OCONUS).

Project Location	Geodetic Reference Horizontal	Geodetic Reference Vertical	Hydraulic/Tidal Reference
American Somoa (Cont)			
Swains	NSRS/ HARN 02	USGS 1963	n/a
Guam	NSRS (83) HARN 1993	Guam Vertical Datum of 2004 GUVD04	Gage 163 0000 PBM TIDAL 4
Northern Marianas Islands:			
Saipan	NSRS (83) 2002 HARN	No. Marianas Vert Datum of 2003 (NMVD03)	Gage 163 3227 PBM UH-2C
Rota	NSRS (83) 2002 HARN	No. Marianas Vert Datum of 2003 (NMVD03)	n/a [Tidal 3]
Tinian	NSRS (83) 2002 HARN	No. Marianas Vert Datum of 2003 (NMVD03)	n/a [Tidal 1]
Aguijan	NSR (83) 2002 HARN	No. Marianas Vert Datum of 2003 (NMVD03)	n/a

Table 2-3 (Concluded). Geodetic and Hydraulic Datum References for USACE Risk Management, Navigation, and HSPP Projects (OCONUS).

Project Location	Geodetic Reference Horizontal	Geodetic Reference Vertical	Hydraulic/Tidal Reference
Marshall Islands:			
Kwajalien	WGS84	Local Tidal	Gage 182 0000 Tidal 8
Palau/Babeldaup	WGS84	Local Tidal	n/a
Micronesia (Federated States):			
Chuuk	WGS84	Local Tidal	184 0000 (Chuuk)
Kosrae	WGS84	Local Tidal	n/a
Pohnpei	WGS84	Local Tidal	n/a
Yap	WGS84	Local Tidal	n/a
Wake Island	NSRS (83)	Local Tidal	Gage 189 0000
Midway (Sand Island)	NSRS (83)	Local Tidal	Gage 169 9910 Tidal 21
Johnson Atoll	NSRS (83)	Local Tidal	Gage 161 9000 PBM MON JON

NOTE: Data in this table were obtained from Jacksonville District, Honolulu District, NGS, and CO-OPS. It is considered current as of 2009. NGS and CO-OPS are periodically updating horizontal and vertical references in these OCONUS areas; thus, users should contact the local District, NGS, and/or CO-OPS to ensure the latest reference datums are being used.

[1] PRVD02 and VIVD09 are currently (2010) being updated based on new leveling and gravity data
[2] All horizontal datums in Hawaiian Islands based on HARN 1993 adjustment
[3] n/a – indicates reference datum or gage is "not available" or is uncertain

b. Inland waterway reference datums. Various datums are used in controlled and free flow portions of inland river systems. Gages on the main stream Mississippi, Ohio, and Missouri Rivers, and their tributaries, are referenced to various datums. Gage zeros may be referenced to a geodetic datum, a low water reference plane, an arbitrary stage elevation, or a purely arbitrary elevation. Gage records maintained and published by USACE, USGS, or other agencies will should clearly define the gage zero reference datum. Some gages have been updated to

NAVD88; however, in many cases this was accomplished using VERTCON/CORPSCON approximations.

(1) Low water reference planes. On the Mississippi River, between the mouths of the Missouri and the Ohio Rivers (the Middle Mississippi River), depths and improvements are referenced to a LWRP. No specific LWRP year is used for the Middle Mississippi north of Cairo, IL. Below Cairo, IL, depths and improvements along the Mississippi River are referenced to a dated LWRP (e.g., LWRP74, LWRP93, LWRP07). These hydraulic-based reference planes are established from long-term observations of the river's stage, discharge rates, and flow duration periods—often developed about the 97-percent flow duration line. The elevation of the LWRP drops gradually throughout the course of the Mississippi; however, some anomalies in the profile are present in places. The gradient is approximately 0.5 ft per river mile in some reaches. The ever-changing river bottom will influence the LWRP. Changes in the stage-discharge relationship will influence the theoretical flow line for the LWRP. The LWRP is periodically updated by District H&H branches in the Mississippi Valley Division (MVD)—see Figure 2-3.

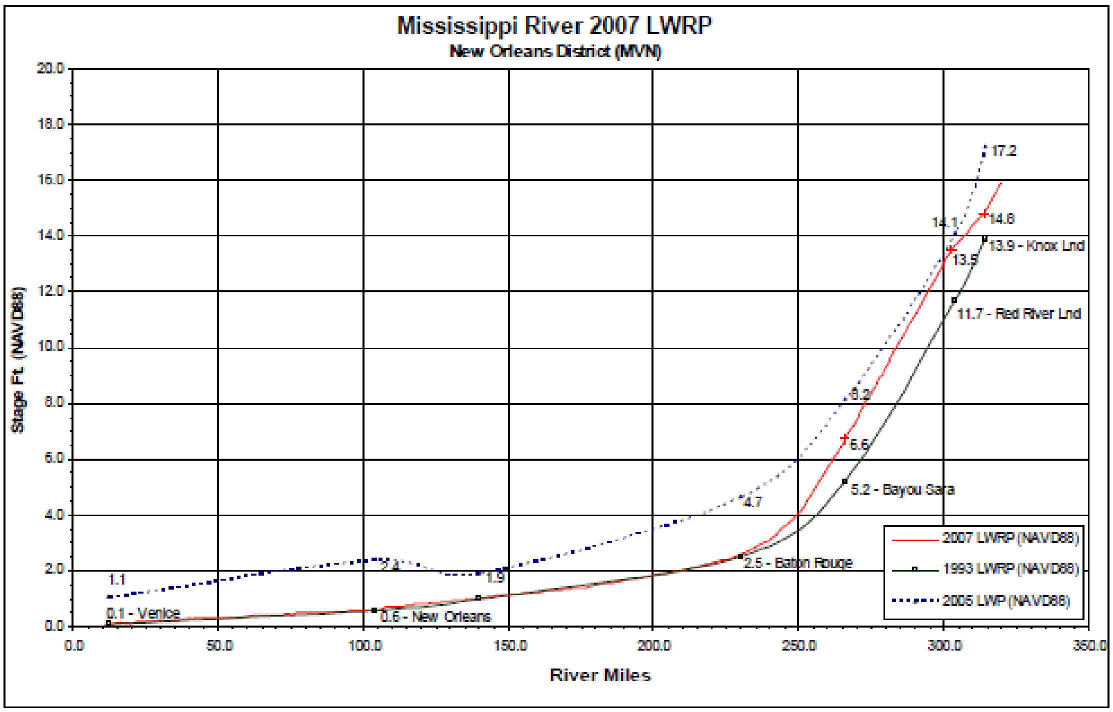


Figure 2-3. Lower Mississippi River LWRP [1993/2005/2007] relationships—New Orleans District. (CEMVN)

(2) Controlled pools. Between river control structures, low water pools are used to reference maintained navigation depths. Since these pools themselves may exhibit some slope, sufficient gages/benchmarks within the pools may need to be established to account for any slope. In the Upper Mississippi River pools, flow profiles between the upper and lower dams establish "project pool" elevations and "ordinary high water profiles." Elevations/stages of

"primary control points" or "hinge/pivot points" between locks and dams are also defined for each pool.

(3) Reservoirs. Most upland reservoir elevations are referenced to an orthometric datum—i.e., a legacy NGVD29 or NAVD88.

c. Sample inland river gage datums and stages. The following examples in Table 2-4 are taken from selected gages covering some 1,700 miles of the Mississippi River from Minneapolis, MN to the Gulf. These sample gages (from over 200) illustrate the varied datums and gage zero/stage references used on inland river systems.

Table 2-4. Reference Elevations on Selected Gages—Upper, Middle, and Lower Mississippi River.

Gage	River Mile	Gage Datum Elevation (ft)	Flat Pool Elevation (ft)	Stage at Flat Pool (ft)
[St. Paul District: most gage datum elevations are set to 700.0 ft or 600.0 ft ... referenced to NGVD29]				
St. Anthony L/D (U)	654.1	700.0	796.5	96.5
L&D 5 (L)	738.1	600.0	651.0	51.0
Winona, MN	725.7	640.0	643.8	5.8
L/D 7 (L)	702.5	600.0	631.0	31.0
L/D 10 (L)	615.1	600.0	603.0	3.0
[Rock Island District: gage datum elevations are referenced to MSL 1912]				
L/D 11 (L)	583.0	588.2	592.0	3.8
Ft Madison Br., IA	383.9	6.8	518.2	518.2
L/D 22 (U)	301.2	446.1	459.5	13.4
[St. Louis District: gage datum elevations are referenced to NGVD29]				
L/D 22 (L)	301.2	446.1	449.1	3.0
L/D 24 (U)	273.5	421.81	445.5	445.5
L/D 27 (L)	185.1	350.0	380.5	380.5

Table 2-4 (Concluded). Reference Elevations on Selected Gages—Upper, Middle, and Lower Mississippi River.

Gage	River Mile	Gage Datum Elevation (ft)	Flat Pool Elevation (ft)	Stage at Flat Pool (ft)
			Low Water Reference Plane	[open flow]
St. Louis, MO	179.6	379.94	376.4	
Cape Girardeau, MO	52.1	304.65	309.9	
Cairo, IL (Ohio Riv.)	2.0	270.47	277.9	
	Above Head of Passes			
New Madrid, MO	889.0	255.48	303.48	
Vicksburg, MS	435.7	46.23	99.47	
Red River Landing	302.4	0.0	60.94	
New Orleans	102.8	0.0	21.27	
Head of Passes	-0.6	0.0	12.03	

d. Datums on other river systems. Gages in portions of the Ohio River are referenced to the "Ohio River Datum (ORD)." Flood stages are relative to the gage zero—e.g., at Cincinnati, the 52.0 ft flood stage is relative to the 429.613 ft gage zero (ORD). The Green River is referenced to the "1929 General Adjustment" which may be equivalent to NGVD29. Gages on the Missouri River and tributaries are referenced to "MSL." Other USACE projects have similar local river datum references that are hydraulically modeled—e.g., Atchafalaya River, Columbia River, Snake River, etc.

e. Variety of inland reference datums. The following excerpt from the University of Illinois "Natural Resources Geospatial Data Clearinghouse" illustrates the need to have reliable conversions between legacy datums and to standardize datums for civil works applications.

> *"Each benchmark is tied to a specific known elevation marker, called a datum. Agencies working in Illinois rely on several vertical datums which are not easily related to each other. Some of these include:*
>
> *NAVD88, the North American Vertical Datum of 1988.*
> *NGVD29, the National Geodetic Vertical Datum of 1929.*
> *The City of Chicago Datum, one of numerous legacy municipal datums.*
> *NGVD12 [i.e., 1912 Adjustment], used by the USACE Rock Island District for management of the Mississippi River.*
> *The Ohio River Datum, used by the USACE Louisville District for management of the Ohio River.*

Approximate conversions between these legacy data exist, but their use degrades the precision of the measurements. During emergencies, these inconsistencies can cause confusion and waste valuable time. Multiple datums also make it virtually impossible to create useful, accurate, seamless maps and supporting flood data for the state. An example of the problems that multiple vertical datums can cause can be found in the Mississippi Valley in Northern Illinois, where levee top surveys are in NAVD88 and water level measurements are in NGVD12.

Because of the large number of different datums, numerical conversions between networks are approximate. This fact lowers the accuracy of existing elevation data sets."

CHAPTER 3

Survey Accuracy Standards and Procedures for Connecting Projects to the National Spatial Reference System

3-1. General. As outlined in Chapter 1, the designed, constructed, and maintained elevation of a project must be referenced to a consistent framework, or vertical datum. References to two primary and distinct reference datums are required:

a. Hydraulic or Water Level Datums. Water surface elevation relative to a locally defined hydraulic reference plane on a river, pool, lake, or tidal body, from which flood protection design elevations or navigation grades are derived.

b. Geodetic or Orthometric Datums. Three-dimensional horizontal and vertical frameworks defined relative to a federally recognized terrestrial and/or extraterrestrial (satellite-based) reference datum.

Throughout the life cycle of a project, these two reference datums must be accurately established, maintained, and defined relative to "Permanent Bench Marks" (PBMs) established at each project site, hereinafter termed “Primary Project Control Points" (PPCP). Supplemental, or "Local Project Control Points" (LPCP), and "Temporary Bench Marks" (TBM) used for construction orientation and grade, are established from these PPCPs. These PPCPs must be firmly connected to nationwide vertical reference frameworks—the NSRS, in coastal regions the NWLON, and defined datums in OCONUS areas. The following paragraphs in this chapter provide guidance on establishing PPCPs and LPCPs at each project site.

3-2. Definitions. The following definitions apply to terms used in this and subsequent chapters.

a. Geodetic Surveying. Survey measurements performed to relate project features to a nationwide reference datum (i.e., the NSRS), typically using static GPS observations over long baselines or precise geodetic differential leveling methods. Geodetic surveys discussed in this guidance are performed for nationwide geospatial reference purposes only; they are not applicable to local project design and construction surveys outlined in the next paragraph.

b. Topographic or Engineering and Construction Surveying. Surveys used to set project control monuments on levees and related flood protection structures, topographic surveys for planning and design, construction stake out, levee cross-sections, levee profiling, etc. Engineering and construction surveys are performed using total stations, differential levels, and/or GPS/RTN methods; following the techniques outlined in EM 1110-1-1005 (*Control and Topographic Surveying*). Procedures and accuracies generally follow "Third-Order" methods described in that manual. These surveys, or fixed control monuments/bench marks established therefore, are usually not included in the NSRS; however, there may be exceptions.

c. Primary Project Control Points (PPCP). Bench marks set on or near a project that are connected with and published in the NSRS, and are used to densify local project control monuments or develop project features. These NSRS bench marks may be established by the NGS, USACE, or other agencies. Each USACE project and water level gage should have at least one PPCP.

d. Local Project Control Points (LPCP). Monuments (PBMs, TBMs, hubs, etc.) used to reference project features, alignment, elevations, or construction. Monuments may be atop levees (e.g., PBMs set at levee sector "points of intersection" or PIs) or offset to the levee alignment. These monuments will usually have local X-Y-Z (SPCS) coordinates along with local project station-offset coordinates. LPCPs are usually not part of the published NSRS; however, they should be directly established from or relative to a PPCP described above. A minimum of three project control bench marks (PPCPs and/or LPCPs) are required for advertised construction plans and references to water level gages.

e. Project NSRS Network Accuracy. This refers to the spatial accuracy of a project's PPCP relative to NSRS points (bench marks) in the nearby geographical region. NSRS regional network accuracy is significant in defining relative orthometric and hydraulic gradient relationships between river gages or tidal gages. It is also significant in defining accuracy relationships between elevations of points established by various federal, state, or local agencies. The NSRS network accuracy is NOT significant or applicable to local project construction stakeout—see "Local Network Accuracy" below. Depending on the type of project and surrounding terrain gradients, required NSRS network elevation accuracies may range from ± 0.1 ft to ± 1 ft. The USACE has adopted a nominal NSRS accuracy standard of ± 0.25 ft.

f. Local Network Accuracy. (Engineering and construction accuracy). Spatial accuracy of a LPCP or project features relative to nearby local reference monuments on the project. Local project accuracy is critical for construction with X-Y-Z tolerances at the ± 0.05 ft level. Local accuracy tolerances are always much smaller than NSRS network accuracy tolerances.

g. Survey Accuracy Standards. Specified target positional accuracy tolerances for a project control monument/bench mark or other project feature (e.g., levee profile, intake structure, inverts, top of floodwall, ground shots).

h. Survey Specifications. Survey procedures and equipment requirements.

i. Uncertainty. The propagated network, instrumentation, and observation errors on a surveyed PBM or feature elevation. Roughly synonymous with "project" and "local" accuracies described above. Refer to Chapter 9 for a more detailed discussion of propagated elevation uncertainties on levee grades or navigation projects.

j. Global Positioning System (GPS) Surveys. "GPS surveys" referenced in this manual imply differential carrier phase GPS baseline measurements—also termed "DGPS" surveys. Code phase GPS or autonomous GPS positioning accuracies are not suitable for project control. A number of DGPS survey methods may be employed in establishing project control, variously termed "Static DGPS Baselines," "Real Time Network" (RTN), "Real Time Kinematic" (RTK), "Post-Processed Kinematic" (PPK), and "Virtual Reference Network" (VRN). Refer to EM 1110-1-1003 (*NAVSTAR GPS Surveying*) for details on performing these carrier phase DGPS surveys.

k. Continuously Operating Reference Stations (CORS). The NGS coordinates a GPS network of over 1,400 Continuously Operating Reference Stations (CORS) throughout North America and over 1,800 worldwide, as of 2010. Each CORS site provides GPS carrier phase and code range measurements in support of three-dimensional positioning activities throughout the United States and its territories. The CORS system enables relative positioning accuracies to better than 0.25 ft relative to the NSRS, both horizontally and vertically.

l. Online Positioning User Service (OPUS). OPUS is an interactive/Internet-based NGS software system that processes static GPS baselines relative to the CORS. It provides near real-time X-Y-Z coordinates relative to the NSRS. OPUS processes GPS data files with the same models and tools which help manage the CORS network, resulting in "CORS/OPUS" coordinates which are both highly accurate and highly consistent with other users. A computed "CORS/OPUS" NSRS position on a bench mark can also be shared publicly via the NGS/NSRS database. Planned upgrades to OPUS may allow merging CORS baselines with conventional GPS network, topographic, and/or differential leveling observations.

3-3. Distinction between NSRS Control and Local Project Control.

a. Project control. A critical distinction must be made between:

(1) Geodetic Control. The regional “geodetic survey" process of referencing USACE project elevations to NAVD88 or NAD83 relative to nearby points on the NSRS (PPCP), and

(2) Local Engineering & Construction Control. Engineering and construction surveying requirements necessary to design, align, stake out, and construct a local flood or water control structure, a HSPP, or a navigation project relative to local project control (LPCP).

b. Control accuracy. Figure 3-1 illustrates the distinction between NSRS network and local project control accuracies. The PPCP has been connected to other adjacent points in the NSRS to an accuracy of ± 0.22 ft. This “NSRS Network Accuracy” is based on the adjustment statistics from the point’s connection, such as GPS baseline reductions, differential leveling loop closures, etc. The adjusted NSRS elevation of 298.72 ft is assumed absolute and is used to establish elevations on the two levee LPCP monuments shown in Figure 3-1. The elevations of these levee LPCPs may be determined by various topographic survey methods—levels, DGPS, RTN, RTK, or total station. Figure 3-1 depicts dashed lines from the PPCP to each local control PBM at Station 0+00 and Station 15+72.4; indicating GPS surveys were used to obtain elevation differences over each baseline. Given observed differential elevations over each baseline from the PPCP to Stations 0+00 and 15+72.4, NAVD88 elevations are transferred to these local monuments. Due to error propagation, these local LPCP elevations have a slightly larger NSRS “network” accuracy than the PPCP. However, their “Local Network Accuracy” of ± 0.1 ft is based on the observed GPS baseline closure accuracies. Had differential levels been run along the levee between stations 0+00 and 15+72.4, then the level misclosure would give an indication of the relative accuracy. The LPCPs thus have both a local (relative) accuracy needed for construction and a NSRS network accuracy needed for regional engineering and mapping purposes. These resultant local and network accuracies may also be termed "uncertainties."

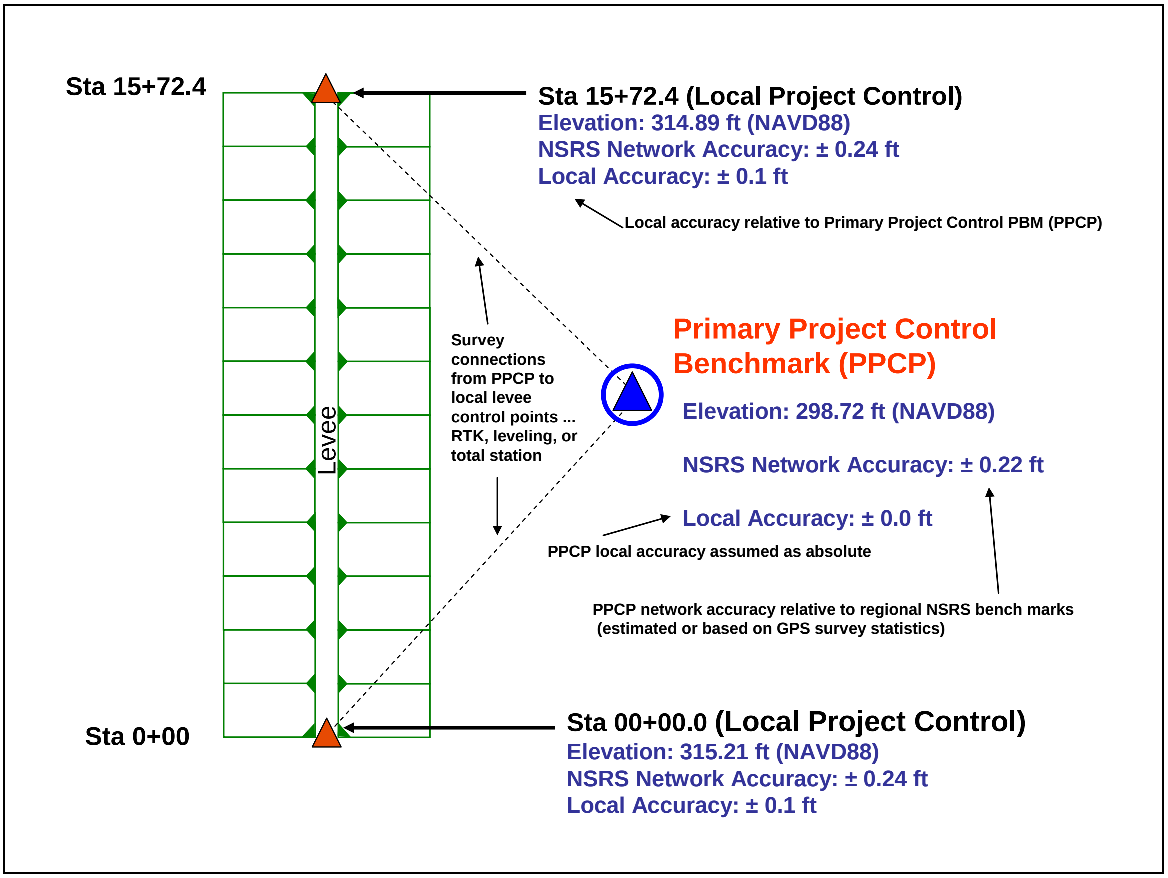


Figure 3-1. Distinction between Primary Project Control and Local Project Control points on a simple levee segment-- Network and Local Accuracies.
(Hydraulic stage or tidal datum relationships not shown)

c. Control survey method. The survey method used to connect local project control PBMs and TBMs is dependent on the accuracy requirements of the project. DGPS, RTK, or RTN methods (generally accurate to ± 0.1 ft) will normally suffice for design and construction of most USACE civil works projects. More accurate differential leveling may be required for water control structures, locks, dams, floodwalls, etc. If the above distinction between local and network project accuracies is not clearly understood, then unnecessary USACE resources may be expended performing higher accuracy “geodetic” surveys to achieve elevation accuracies that have no hydrologic or hydraulic engineering requirement; either within USACE or in conjunction with other agencies.

d. Control database. The "USACE Survey Monumentation Archival and Retrieval Tool" (U-SMART) database options allow linking PPCPs with LPCPs on a specific USACE flood risk management or navigation project. Details on U-SMART are discussed in Section 3-13.

3-4. Recommended Accuracy Standards for USACE Project Control. PPCP connections to the NSRS are made by field survey techniques—typically by traditional differential leveling or by differential GPS height observations between published NSRS PBMs and PPCPs.

a. PPCPs must be geospatially referenced such that designed protection elevations are:

(1) Consistent with federally mandated vertical datums (e.g., NAVD88, IGLD85).

(2) Consistent with federally mandated horizontal datums (e.g., NAD83).

b. The minimal accuracy standards in Table 3-1 apply to USACE PPCPs that are established relative to the regional NSRS network; that is these PPCPs are directly connected by differential leveling and/or GPS baselines to nearby NSRS points. These NSRS connection observations to PPCPs shall be submitted to NGS for inclusion in the NSRS. These are minimal accuracy standards that are believed adequate for most inland flood risk management and coastal projects. The accuracy standards in Table 3-1 do NOT apply to supplemental LPCPs, topographic, or construction surveys conducted from these primary points—see paragraph 3-3 on the critical distinctions between "primary" and "local" project control.

Table 3-1. Recommended Minimal or Target Accuracy Standards for Connecting Primary Project Control Points on USACE Projects to the US Department of Commerce NSRS Network.

	NSRS Accuracy (95%) [1]	Reference Datum (CONUS)
Vertical	± 0.25 ft (± 8 cm)	NAVD88
Horizontal	± 2 ft (± 60 cm) [2]	NAD83

[1] Accuracies are at the 95% confidence level relative to regional points published by NOAA on the NSRS.
[2] Horizontal accuracy is for global reference purposes—achievable DGPS derived accuracies are currently ± 0.2 ft typical.

c. These NSRS network accuracy standards at the ± 0.25 ft level are believed to be representative of the nominal accuracy requirements for the vast majority of USACE levee systems and related water control projects. These accuracies should support flood forecasting models, stage-discharge relationships, flood inundation modeling, channel design, levee freeboard design, risk assessment, and related river hydraulics work. Additional details on evaluating project accuracy requirements are covered in later sections of this chapter.

d. There may be levee or river segments where these standards are either too rigid or perhaps require tightening, as might be the case in high subsidence regions. This decision on the required project accuracy should be left to those performing hydrology and hydraulics studies over a watershed or flood risk management region. If such technical guidance is not available, then the criteria in Table 3-1 may be used by default. If more rigid accuracy standards are required, then refer to the guidance in Chapter 8.

e. It is also essential that the required survey accuracy be derived from realistic engineering applications associated with a flood risk management system or project. This is best summarized in Appendix A of FEMA's "*Guidelines and Specifications for Flood Hazard Mapping Partners*" (FEMA 2003) which emphasizes the need for establishing reasonable map accuracy and resolution specifications for flood insurance studies:

> *"The specified accuracy of FIRM work maps produced by Mapping Partners must be sufficient to ensure that the final FIRMs produced by FEMA can be reliably used for the purpose intended. However, the accuracy and resolution requirements of a mapping product must not surpass that required for its intended functional use. Specifying map accuracies in excess of those required results in increased costs, delays in project completion, and reduction in the total numbers of new or revised products that the Mapping Partner may generate. Mapping accuracy requirements must originate from functional and realistic accuracy requirements."*

The above statement makes it imperative that the project's functional and realistic accuracy requirements be defined based on the requirements of a flood system profile model or navigation project model. Once the functional accuracy requirement is defined, USACE surveyors can then define the appropriate survey specifications needed to meet that accuracy.

f. The required NSRS network accuracy of a primary or local project control point (and indirectly to any topographic feature on the project, such as a levee crest, floodwall cap, pump station invert, etc.) is also determined by the engineering requirement for regional consistency between these points. These regional network accuracy requirements relative to the NSRS may be contingent on compliance with one or all of the following:

(1) USACE, USGS, FEMA, NOAA or other agency hydrologic or hydraulic analyses, models, or water surface profiles between and within large river reaches/basins and river stage gages.

(2) USACE/FEMA/other flood inundation mapping study accuracies.

(3) Consistency with FEMA flood insurance study accuracies performed under FEMA's National Flood Insurance Program (NFIP)—Flood Hazard Maps, (Digital) Flood Insurance Rate maps (FIRM/DFIRM), etc.

(4) Consistency with Federal mapping accuracy standards in the project area—e.g., USGS.

g. NSRS network horizontal accuracy standards (± 2 ft) in Table 3-1 are obviously not critical for hydraulic engineering purposes. This nominal horizontal standard can be easily exceeded with minimal observation times using various DGPS methods. This would be done in cases where recovered bench marks do not have a horizontal position. When static DGPS observations are conducted at a point for elevation determination, horizontal accuracies relative to the NSRS will usually be around the ± 0.2 ft level.

3-5. FEMA Accuracy Standards for Flood Insurance Rate Maps. Since regional conformance with FEMA NFIP studies is an essential goal of any USACE flood risk management project and/or study, both USACE and FEMA must be on the same vertical datum—i.e., NSRS NAVD88—or, at minimum, have a firmly established relationship between different vertical datums. FEMA standards and specifications clearly detail this intent. Tables 3-2 and 3-3, taken from Appendix A (*Guidance for Aerial Mapping and Surveying*) of FEMA's "*Guidelines and Specifications for Flood Hazard Mapping Partners*" (FEMA 2003), illustrate the required FIRM/DFIRM accuracy requirements relative to the NSRS. In summary, FEMA NSRS regional elevation accuracy standards are (1) standard 2-foot equivalent contour interval accuracy ($Accuracy_z$ = 1.2 foot) appropriate for flat terrain, and (2) standard 4-foot equivalent contour interval accuracy ($Accuracy_z$ = 2.4 foot) appropriate for rolling to hilly terrain. In effect, USACE flood protection structure elevations should have relative NSRS regional network accuracies at or better than the above tolerances in order to be consistent with FEMA flood insurance studies, FIRMs, DFIRMs, etc. The USACE control survey standards and specifications in this guidance document will yield NSRS network accuracies well within these FEMA NSRS accuracy standards. These more precise USACE accuracy standards result from more rigorous hydraulic engineering and levee design requirements than those needed for NFIP studies.

Table 3-2. FEMA Vertical Accuracy Standards. (FEMA 2003)

NMAS Contour Interval	NMAS VMAS 90%	NSSDA $Accuracy_z$ 95%	NSSDA $RMSE_z$	ASPRS 1990 Class 1/2/3 Limiting $RMSE_z$
2 Foot	1 ft	1.2 ft	0.6 ft (18.5 cm)	0.7 ft (Class 1) 1.3 ft (Class 2) 2.0 ft (Class 3)
4 Foot	2 ft	2.4 ft	1.2 ft (37.0 cm)	1.3 ft (Class 1) 2.7 ft (Class 2) 4.0 ft (Class 3)

Table 3-3. FEMA Horizontal Accuracy Standards. (FEMA 2003)

NMAS Map Scale	NMAS CMAS 90%	NSSDA Accuracy $_r$ 95%	NSSDA RMSE $_r$	ASPRS 1990 Class 1/2/3 Limiting RMSE $_r$
1 " = 500 ft	16.7 ft	19.0 ft	11.0 ft	7.1 ft (Class 1) 14.1 ft (Class 2) 21.2 ft (Class 3)
1 " = 1,000 ft	33.3 ft	38.0 ft	22.0 ft	14.1 ft (Class 1) 28.3 ft (Class 2) 42.4 ft (Class 3)
1 " = 2,000 ft	40.0 ft	45.6 ft	26.3 ft	28.3 ft (Class 1) 56.5 ft (Class 2) 84.9 ft (Class 3)

3-6. USGS National Map Accuracy Standards. USGS topographic maps at 1:24,000 (1" = 2,000 ft) are generally designed to be accurate to one-half the contour interval on the map. Thus, for a standard 2 ft contour map, the estimated vertical accuracy is ± 1 ft (at a 90% confidence). The horizontal accuracy is specified at 1/30th of the scale, or ± 67 ft for a 1 in. = 2,000 ft (7.5 minute) quadrangle. The targeted NSRS network accuracy standards performed under this guidance will significantly exceed these USGS mapping accuracy standards.

3-7. Local Topographic, Engineering, and Construction Survey Accuracy Standards. Local levee alignment LPCP bench marks (e.g., PIs, PTs, PCs, gage references, etc.) and topographic features (levee profiles, cross-sections, etc.) should be positioned relative to the nearest PPCP that has been referenced to the NSRS. This PPCP(s) may be a published NGS bench mark or a USACE monument that has been connected to (and input into) the NSRS. These local project control surveys will typically be performed over short distances—for example, within range of an RTK base station, within the coverage of a GPS Real Time Network (RTN), or within a reasonable distance for differential leveling or total station observations. Field survey procedures will follow engineering and construction survey guidelines in EM 1110-1-1005 (*Control and Topographic Surveying*). Recommended survey accuracies of feature points are listed in Table 3-4.

Table 3-4. Recommended Local Project Elevation Accuracies for Flood Risk Management Project Features.

	Relative Accuracy (95%)	Reference Datum
Levee or floodwall control bench marks:	± 0.15 ft	NAVD88/NAD83
Hard topographic features:	± 0.3 ft	NAVD88/NAD83
Ground shots:	± 0.5 ft	NAVD88/NAD83
Construction stake out	± 0.01 to 0.05 ft	Local site
General floodplain mapping (GIS)	± 0.5 to 2 ft	NAVD88/NAD83

NOTES:

Local project control will typically have two horizontal references: (1) a local SPCS system, and (2) the construction station/chainage-offset system.

The above accuracies are not relative to the regional NSRS but are for local topographic and construction purposes. Elevations are reported relative to NSRS vertical datum.

The latest geoid model published by NGS will be used to estimate and correct local geoid undulations for all topographic densification using RTK/RTN methods. At longer distances greater than 3 miles from the RTK base, frequent calibration check points are recommended if a standard RTK/RTN site calibration/localization process is not feasible—see EM 1110-1-1005.

a. Local horizontal accuracies should generally be within the tolerances for vertical accuracies shown in Table 3-4. When using RTK/RTN methods, the horizontal accuracies will be slighter better—and over current RTK/RTN application distances, a ± 0.1 ft (± 3 cm) local relative accuracy should be achieved at any type of point located (assuming appropriate site calibration procedures are followed). For example, the horizontal distance between two levee PIs 2,000 ft apart will be accurate horizontally to the ± 0.1 to 0.2 ft level when these points are connected using either RTK/RTN or total station EDM observations, and usually better than ± 0.05 ft vertically when differential levels are run. These local (relative) accuracy levels are sufficient for any levee stationing stake out needed for construction or maintenance grading. Thus a PI monument will have a local project stationing-offset and elevation coordinate for maintenance and construction, and will also be referenced to the NSRS (NAD83 and NAVD88) for regional mapping orientation purposes.

b. As illustrated on Figure 3-1, NSRS network accuracies of any local bench mark (LPCP) or feature point will be slightly larger than the accuracy of the controlling (primary) NSRS bench mark—due to error propagation in the survey process. For example, if an RTK base is set over a NGS NSRS network point with an established (estimated or published) NSRS “network” accuracy of ± 0.22 ft, and a local project bench mark atop the levee on a PI is shot in with an estimated RTK “precision” of ± 0.1 ft, then the estimated (propagated) accuracy of the PI bench mark is roughly ± 0.24 ft— as computed from $[0.22^2 + 0.1^2]^{1/2}$. If this PI point is later occupied with an RTK base to cut in hard levee features or levee crest ground profiles, then the estimated (propagated) accuracy of these elevations would be roughly ± 0.26 ft relative to the regional NSRS—i.e., $[0.24^2 + 0.1^2]^{1/2} = \pm 0.26$ ft.

3-8. <u>Hierarchy of Preferred Survey Methods for Establishing New Primary Bench Marks Relative to the NSRS</u>. Published NSRS bench marks of Second-Order or higher order should be used as PPCPs when they are on or near a project. When no existing (or published) NSRS vertical control is available near the project, PPCPs must be set to an established density, accuracy, and observing specification. Newly established project control must also be published in the NSRS by forwarding geodetic observations and descriptive data to the NGS. The essential purpose for establishing this primary control is to provide assurance that navigation grades and flood protection structure elevations measured from these PPCPs will be adequately referenced to the NSRS (currently NAVD88). A variety of survey procedures may be used to establish new PPCPs. Table 3-5 details a hierarchy of survey methods by which project elevations and grades can be connected to the NSRS (and the NWLON if applicable). The order of preference in Table 3-5 is somewhat dependent on the mechanism for inputting data to the NSRS—item [II] being the simplest, and [III] and [IV] currently being the most difficult.

a. Preferred survey method. The survey method chosen from the Table 3-5 will have a major impact on the amount of field effort and cost. Preference [I] obviously requires minimal field work other than verifying the current adequacy and stability of the existing NSRS bench mark. The "CORS/OPUS method"—Preference [II]—at a new PPCP can be performed for economically using a one-man survey crew and OPUS-based software to input the data to the NSRS. Positioning this same point by NSRS networked baseline connections—Preference [III]—would require a 3- to 4-man survey crew. If Blue Book techniques are used to input this data into the NSRS, the total cost to establish this point could be 5 to 10 times the cost of Preference [II]. This cost will be significantly reduced when NGS develops software to replace the Blue Book. Differential leveling ties —Preference [IV]—will be cost-effective only over short lines where Third Order closure tolerances can be maintained. They will also require connections with at least two or more published NSRS bench marks. Higher-order instrumentation and procedures will be required over longer lines, significantly increasing field effort. Inputting level line data into the NSRS via Blue Book methods also requires significant administrative effort—the cost of which may exceed the cost of the field work for short lines. NGS is developing software that allows simplified input of leveling data to the NSRS. From the above, it is obvious that effort should be made to locate and utilize existing NGS NSRS vertical control as PPCPs—and establish as few as possible new points. When new primary points must be set, CORS/OPUS methods [II] should be used to the maximum extent possible.

Table 3-5. Preferred Hierarchy of Survey Methods for Establishing New PPCPs Relative to the NSRS.

Order of Preference	Survey Method	NSRS Input Method	Notes
[I]	Use existing NSRS control	not applicable	NSRS check surveys only
[II]	GPS: CORS/OPUS	OPUS input [1]	Restricted to CORS within 200 miles
[III]	GPS: Networked baselines to nearby NSRS marks if CORS/OPUS solutions cannot be performed	Blue Book or OPUS input [1]	Include any CORS baselines in adjustment
[IV]	Differential Leveling from NSRS points	Blue Book or OPUS input [1]	Setting primary points at levees or gages

[1] Various NSRS input methods via OPUS-based solutions are being developed by the NGS. Monitor NGS websites for the current versions of OPUS data input techniques to the NSRS.

b. PPCP coverage density. The density, or spacing, of PPCPs that are directly connected to the NSRS will vary with the geographic extent and type of project. Ideally, a PPCP PBM should be located as close as possible to the project—preferably on one of the project's reference PBMs. In general, each project should have at least one PPCP relatively close to the project and a published PPCP reference bench mark a short distance (< ¼ mile to minimize number of level setups) from a river or tide gage.

(1) Levee projects. Any suitable existing levee control monument may be used as a new PPCP. For extensive levee segments, PPCPs spaced every 15 to 20 miles will generally provide adequate coverage from which to perform any non-NSRS supplemental control observations to LPCPs needed to survey levee grades and features relative to NAVD88, such as by observing RTN/RTK or static DGPS baselines between the PPCP and the LPCPs. For projects that require

multiple PPCPs, the relative accuracy between PPCPs should adhere to the local survey accuracy requirements. Additional details on PPCP density requirements for large levee systems are covered in Chapter 6.

(2) Navigation and HSPP projects. Navigation projects should have a PPCP located as close as possible to the project since that PBM will likely be used for controlling surveys, grading, and dredging operations with RTN/RTK machine control techniques. Ideally a NOAA tidal PBM near the project site is designated as the PPCP and is used as both an RTK base and tidal calibration point. See Chapters 4 and 5 for details on establishing PPCPs on coastal projects.

c. Real Time Networks (RTN). Expanding use of RTN coverage throughout CONUS significantly minimizes the need for a dense network of PPCPs and LPCPs on project sites. Given most RTNs are directly referenced to the NSRS CORs stations, they are, in practice, a "PPCP," requiring only a sparse network of local PPCPs and/or LPCPs for site calibration of the RTN. RTNs, and successor GNSS technologies, are expected to eventually replace the need for monumented NSRS PPCPs; however local LPCP networks will likely still be required for construction site calibration and boundary referencing.

3-9. Preliminary Evaluation of Existing Project Control. For each project, a preliminary evaluation of the acceptability and reliability of existing project control and their reference datums must be made.

a. For example, the main issues to be evaluated for a project would include:

(1) That protection grade elevations are referenced to NAVD88 based on PPCPs published in the NSRS.

(2) That river gages owned and/or operated by the Corps (or other agency gages used by USACE) are referenced to NAVD88 based on control bench marks published in the NSRS, and that the relationship between the geodetic and hydraulic datums at the gage are firmly established and documented.

(3) That project drawings, CADD files, and related documents, contain full and complete metadata on PPCPs and LPCPs, and the relationship between the geodetic and hydraulic datums and any associated legacy datums.

b. Upon completing a preliminary evaluation for each project, it may be determined that no additional field survey work is required for connection to the NSRS. This would include:

(1) Projects that have been recently connected to the NSRS, such as those that were included in a NGS Height Modernization project.

(2) Projects with control firmly surveyed on NGVD29 and directly leveled to NSRS points that were subsequently readjusted to NAVD88.

(3) Projects that were recently connected to the NSRS by local sponsors, levee boards/districts, State DOT, or other local agency, but connections were not published in the NSRS.

c. If the initial assessment determines that the project datum is not referenced to the current NSRS, and a required accuracy tolerance is established, then the amount of field survey effort involved will be largely governed by the following factors:

(1) Availability, acceptability, and accessibility of existing (published or unpublished) vertical control in the region, including RTN networks.

(2) If GPS survey observations are required, the ability to use a CORS/OPUS elevation determination in lieu of observing extensive DGPS static baseline networks.

(3) Availability of expedited procedures for submitting bench mark descriptions and elevation data into the published NSRS, such as OPUS-based input methods.

d. The following paragraphs provide guidance on estimating the field survey scope required that will be needed to update a project datum to NAVD88 and, where applicable, publish the PPCP(s) for a project on the NSRS. These sections relate to the preference options listed in Table 3-5.

3-10. Utilizing Existing NSRS Control for USACE Primary Project Control PBMs. If published NSRS vertical control (Second Order or better) is available on or near a project, and at a density (spacing) adequate for supplemental topographic or geodetic surveying purposes (ideally well less than 15 miles distant from the project site, depending on available methods for surveying supplemental LPCPs), then there is effectively no need to establish a new NSRS primary project control reference point. These existing NSRS bench marks can be used to survey NAVD88 elevations on local control points (LPCPs) at the project site or to perform topographic or hydrographic survey operations—using standard topographic or geodetic survey methods, such as short-term static DGPS baseline observations, RTN/RTK techniques, differential leveling, total station traverse, etc. Optionally these LPCPs can be classified as PPCPs if they are positioned using one of the techniques listed in Table 3-5. The published NGS data for a PBM will be accepted as reliably connected to the NSRS after checks into one or more surrounding NSRS points. In effect, bench marks published by NGS on the NSRS will be accepted at "face value" after verification. If the NSRS bench mark does not have a horizontal position, this can be quickly obtained by a short-term CORS/OPUS observation. General criteria are shown in Table 3-6.

a. A recovered NGS NSRS bench mark will have some elevation uncertainty relative to the nationwide NSRS. Given limited USACE resources, it is not the intent of this guidance to investigate and minimize these published NSRS bench mark inaccuracies. It should be noted that existing NSRS bench mark elevations may have a greater relative uncertainty than elevations determined by height reductions based on recent GPS/CORS observations. In time, it is anticipated that all primary bench mark elevations will be observed and monitored relative to the nationwide CORS network.

Table 3-6. Recommended Criteria when Utilizing Published NSRS Control as the PPCP.

Check validity of published elevation	Yes
Nearby NSRS bench mark elevation check points	At least one – two recommended if feasible
Check survey tolerance between NSRS bench mark elevations	± 0.1 ft (± 3 cm) [1]
Survey elevation check methods	RTN/RTK, CORS/OPUS, differential levels, total station
NSRS input of check surveys	No
Recovery note on NSRS bench mark	Recommended—submit on-line to NGS or U-SMART
Horizontal position on vertical bench mark	Short term (< 2 hours) OPUS observation

[1] The acceptable tolerance between NSRS bench marks is project and site dependent, age of the marks, etc. Higher tolerances may be justified in some instances—use engineering judgment in determining acceptable tolerances.

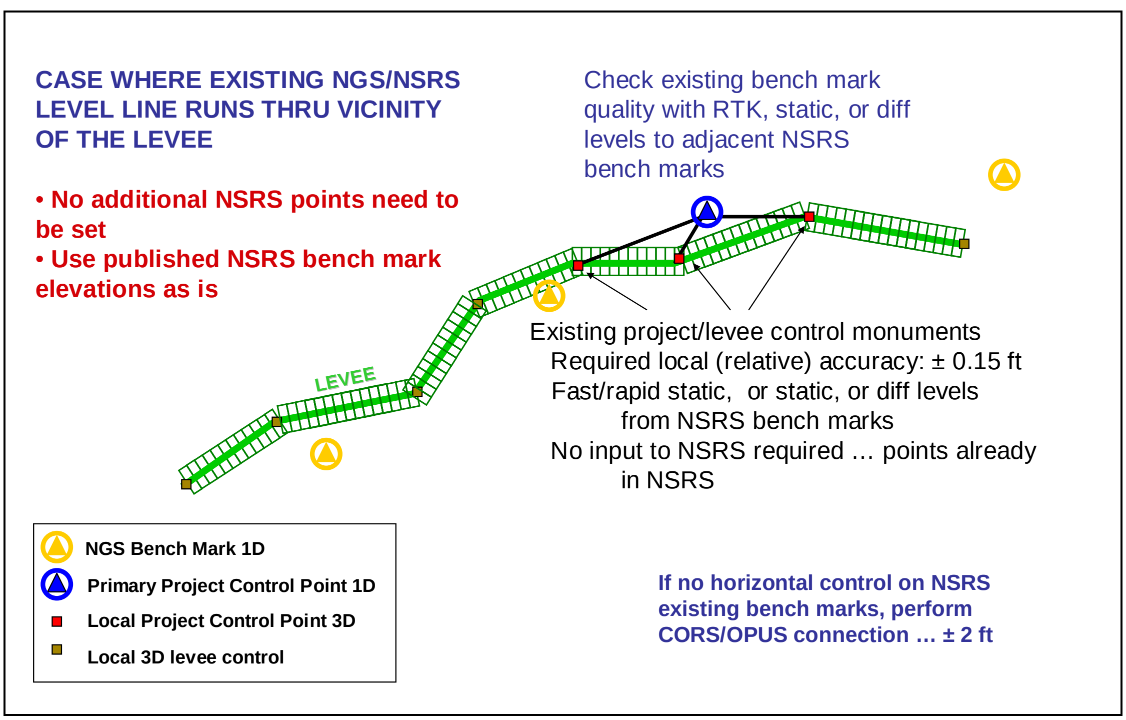


Figure 3-2. Published NSRS control within a levee project.

b. To illustrate a case where existing NSRS control can be used, Figure 3-2 shows a published NSRS line of levels running through a levee segment. In this case, the published NGS bench mark elevations will be accepted as the PPCP, and will be directly used for referencing NAVD88 elevations to supplemental LPCPs on the levee. No long-term static DGPS or CORS/OPUS observations will be required to adjacent points on the NSRS or CORS, other than a vertical tolerance check as indicated in Table 3-6. If the existing NSRS bench mark does not have published horizontal coordinates, a CORS/OPUS observation will provide a general horizontal reference for the PPCP.

c. The first step in evaluating NSRS coverage in a USACE project area is to access the NGS database and search for existing bench marks. This can be done graphically as shown in the screen capture in Figure 3-3. Alternatively, U-SMART can be used to view local NSRS points in a project area—see Figure 3-4. If a USACE levee system is located along a river system parallel with an NGS level line, then any of these bench marks can be directly used to provide NSRS (NAVD88) control on levee points—and only short-term RTN/RTK checks would be performed to confirm NSRS control accuracy and validity of the marks used as control. Per Table 3-6, a tolerance check between the NSRS bench marks of ± 0.1 ft would be considered reasonable.

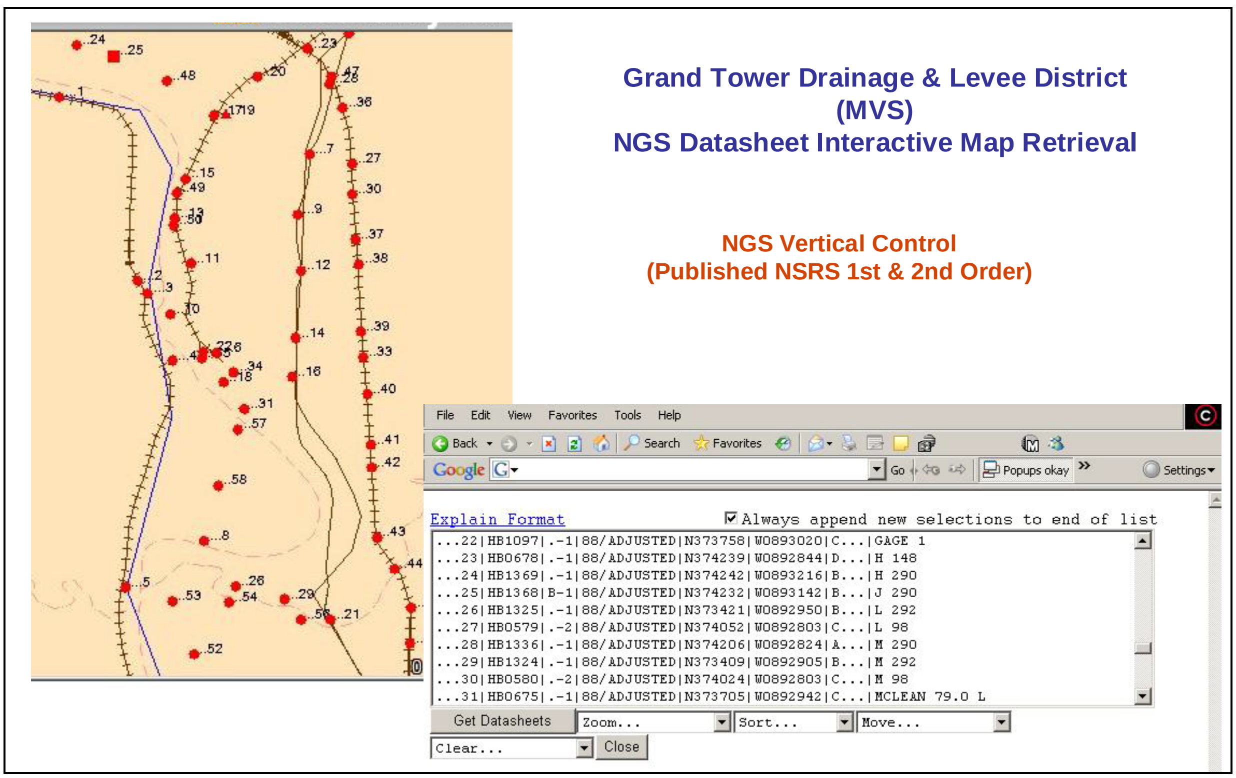


Figure 3-3. Survey control map from NGS web site.

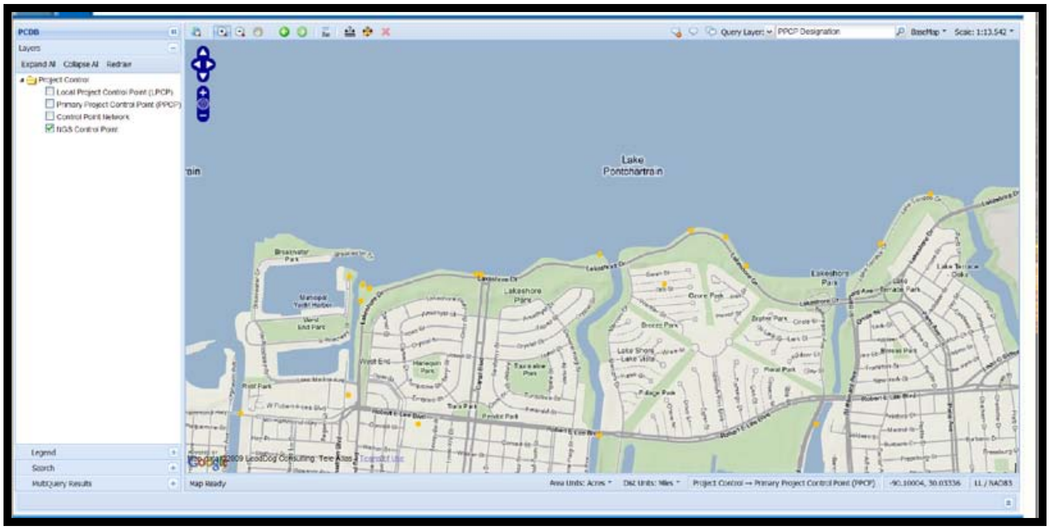


Figure 3-4. U-SMART map showing NSRS database points along Lake Pontchartrain shoreline.

3-11. <u>CORS/OPUS Solutions for Primary Project Control Point Elevations</u>. CORS/OPUS observations (Table 3-5 Preference [II]) will generally be the preferred survey method for relating USACE project control (PPCPs) to the NSRS. Static GPS observations are observed at a PBM relative to a network of CORS. The GPS observables are processed through OPUS and the PBM becomes part of the NSRS if descriptive data are forwarded to NGS—see Figure 3-5. CORS/OPUS solutions are a practical and efficient method of establishing primary project control to a vertical accuracy of ± 0.25 ft.

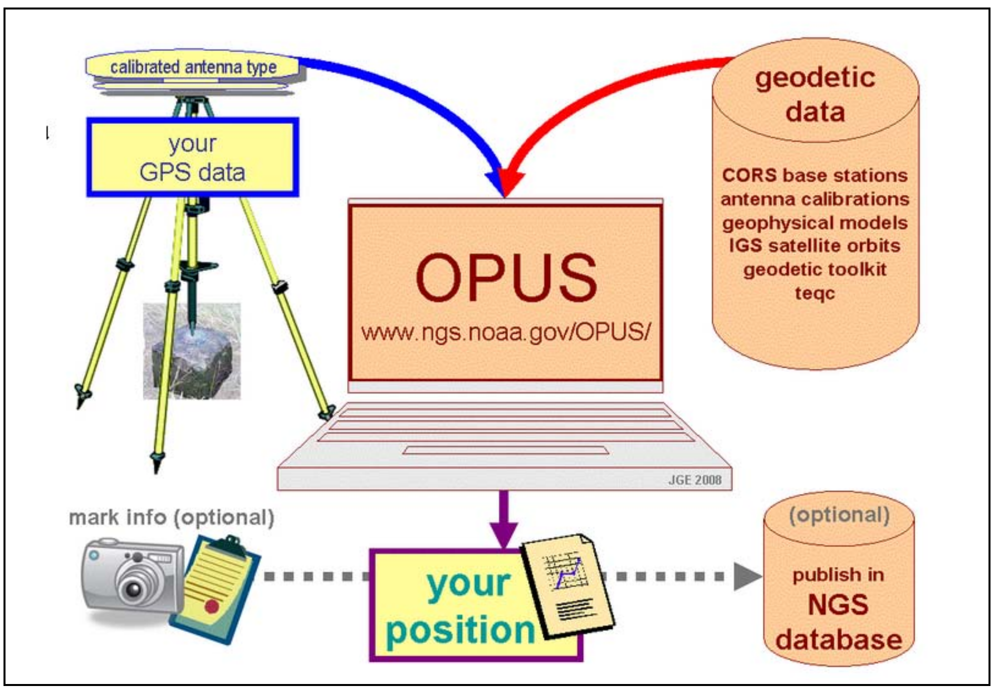


Figure 3-5. CORS/OPUS baseline processing and input to the NSRS database.

a. CORS/OPUS guidelines. When CORS/OPUS solutions are made to establish NAVD88 orthometric elevations on a PPCP, the NGS guidelines in Table 3-7 are recommended. These guidelines are current as of 2010—monitor the AGC and NGS web sites for future changes in these specifications. In the most populated regions in CONUS, CORS coverage is adequate for establishing NAVD88 orthometric elevations on PPCPs. These elevations usually can be obtained in less than one day with a one-man survey crew, and the resulting data can be efficiently input into the NSRS database using OPUS-based input procedures.

(1) CORS/OPUS observations for targeted ± 0.25 ft accuracies to the NSRS do not need to be pre-approved by the NGS; however, one should verify with NGS that the local geoid model is adequate to use to convert GPS ellipsoidal heights to orthometric heights. In most populated regions of CONUS where the NSRS vertical network is fairly dense, the geoid model should be adequate. In these areas, the geoid model accuracy is normally less than ± 3 cm and often closer to ± 1 cm. Thus, errors in the ellipsoidal-orthometric conversion will not be as significant. In mountainous areas or in high-subsidence regions, this may not be the case and NGS should be consulted in advance.

(2) In arriving at the estimated accuracy of a CORS/OPUS solution for an orthometric elevation, the error budget consists of (1) estimated accuracy of the geoid model, (2) the ellipsoid height measurement accuracy, and (3) base CORS station elevation accuracy. In many USACE Districts, < ± 5 cm estimated orthometric accuracies are currently being achieved. The OPUS Solution Report contains an estimate of the orthometric accuracy. Estimated orthometric accuracies exceeding ± 0.25 ft should not necessarily be rejected if "peak-to-peak" tolerances are acceptable. Especially note that valid CORS solutions exceeding some of the tolerances in Table 3-7 may be rejected for input to the NSRS.

Table 3-7. NGS Guidelines for CORS Ellipsoidal and Orthometric Elevation Measurements.
(Primary Project Control Points -- ± 0.25 ft Orthometric Accuracy) [1]

Minimum number of CORS Stations within 200 miles	3
Minimum session time:	One ≥ 4-hour session required plus independent check session recommended. Total of two 4-hour sessions recommended [2]
Number of sessions	2 (see above)
Minimum observations	7,900
Observations used	> 70%
Ambiguities fixed	> 70%
Overall solution RMS	< 3 cm
HI measurements	Fixed height pole recommended; otherwise 3 measurements required in different units
Ephemeris	IGS precise or rapid (available next day)
Maximum Peak-to-Peak tolerances (ellipsoidal):	
Horizontal	< 4 cm
Vertical	< 8 cm
Geoid model	OPUS determined
Geoid model--estimated accuracy at site	NTE 3 cm (check w/NGS)
Data processing and NSRS database input	NGS OPUS-based

Table 3-7 (Concluded). NGS Guidelines for CORS Ellipsoidal and Orthometric Elevation Measurements. (Primary Project Control Points -- ± 0.25 ft Orthometric Accuracy) [1]

Notes:

[1] Some exceptions to the tolerances in these guidelines may be warranted in certain cases; however, a USACE acceptable CORS solution may be rejected for NSRS input. These guidelines are periodically being updated by NGS—they are current as of 2010. Contact AGC or the NGS OPUS web site for subsequent changes and updates.

[2] Since the purpose of the second 4-hour session is used as a check, other methods such as the use of RTN or RTK methods may be used to verify and check the first 4-hour observation. In remote areas it is still recommended that a second 4-hour observation session be done to eliminate the need to travel back to the site if the first 4-hour observation does not meet the requirements.

c. Sample OPUS Solution Report. Figure 3-6 below is an example of a 19-hour OPUS observation at a NOAA tidal bench mark in Georgia. The report statistics indicated the criteria in Table 3-7 were met. (Applicable assessment criteria that should be reviewed in the report are shown as bolded). Note that the 2.0 cm orthometric height accuracy is "peak-to-peak"—not the NSRS relative accuracy estimate.

Figure 3-6. Sample OPUS Solution Report.

```
9511 TIDAL F  [NOAA Tide Gage 867 9511 – Kings Bay, GA — Jacksonville District]
FILE: 71650290.09o 000105225

                              NGS OPUS SOLUTION REPORT
                              ========================

All computed coordinate accuracies are listed as peak-to-peak values.
For additional information: http://www.ngs.noaa.gov/OPUS/about.html#accuracy

      USER: damon.a.wolfe@usace.army.mil            DATE: May 05, 2010
RINEX FILE: 7165029a.09o                            TIME: 18:53:30 UTC


  SOFTWARE: page5  0909.08 master2.pl 0810233      START: 2009/01/29 00:01:00
 EPHEMERIS: igs15164.eph [precise]                  STOP: 2009/01/29 19:15:30
  NAV FILE: brdc0290.09n                        OBS USED: 48669 / 51797   :94%
  ANT NAME: TRM_R8_GNSS     NONE             # FIXED AMB:   259 /   293   :88%
ARP HEIGHT: 1.75                             OVERALL RMS: 0.017(m)
```

Figure 3-6 (Concluded). Sample OPUS Solution Report.

```
REF FRAME: NAD_83(CORS96)(EPOCH:2002.0000)          ITRF00(EPOCH:2009.0778)

         X:       808781.729(m)   0.013(m)          808781.030(m)   0.013(m)
         Y:     -5423271.976(m)   0.018(m)        -5423270.449(m)   0.018(m)
         Z:      3247037.568(m)   0.014(m)         3247037.378(m)   0.014(m)

       LAT:   30 48  6.91976      0.016(m)       30 48  6.94129    0.016(m)
     E LON:  278 28 55.57376      0.014(m)      278 28 55.55623    0.014(m)
     W LON:   81 31  4.42624      0.014(m)       81 31  4.44377    0.014(m)
    EL HGT:          -24.719(m)   0.014(m)            -26.202(m)   0.014(m)
 ORTHO HGT:            3.788(m)   0.020(m) [NAVD88 (Computed using GEOID09)]

                        UTM COORDINATES    STATE PLANE COORDINATES
                         UTM (Zone 17)         SPC (1001 GA E)
Northing (Y) [meters]     3407764.931           89071.578
Easting (X)  [meters]      450456.269          262082.476
Convergence  [degrees]    -0.26520537          0.33222763
Point Scale                0.99963028          0.99994753
Combined Factor            0.99963416          0.99995141

US NATIONAL GRID DESIGNATOR: 17RMQ5045607764(NAD 83)

                              BASE STATIONS USED
PID       DESIGNATION                         LATITUDE    LONGITUDE
DISTANCE(m)
DE6005 GNVL GAINESVILLE CORS ARP          N294111.557 W0821636.736  143609.7
DJ6111 SAV5 SAVANNAH 5 CORS ARP           N320818.937 W0814146.790  149170.2
DK4049 GASK SKIDAWAY ISLAND CORS ARP      N315915.255 W0810122.431  139632.1

                 NEAREST NGS PUBLISHED CONTROL POINT
BC2560      H 62 06                       N304808.555 W0813108.247     113.4

Horizontal network accuracy =    0.00212 meters.
Vertical network accuracy   =    0.00228 meters.

  STATE PLANE COORDINATES - U.S. Survey Foot
         SPC (1001    GA E)
Northing (Y) [feet]         292229.003
Easting  (X) [feet]         859848.923
Convergence  [degrees]      0.33222763
Point Scale                 0.99994753
Combined Factor             0.99995141
```

b. CORS/OPUS data submittal and NSRS publication. The guidelines in Table 3-7 must be followed in order to meet NGS QC and QA criteria for inputting CORS-derived bench mark elevations into the NSRS. The NGS publishes detailed procedures on their web site for submitting CORS/OPUS GPS observations and publishing station data in the NSRS database. A published PPCP datasheet processed through the OPUS database input system to the NSRS is shown in Figure 3-7.

SURVEY DATASHEET (Version 1.0)

PID: BBBS37
Designation: USACE V-LOWPAP-4
Stamping: V-LOWPAP-4 2K9
Stability: Monument will probably hold position well
Setting: Object surrounded by mass of concrete
Description: This monument is used to comply with the Comprehensive Evaluation of Project Datum (CEPD) initiative. This monument is a standard USACE Brass Cap set atop a 5/8" rebar in concrete. This point is +/- 10 feet southerly from the southerly edge of a pedestrian walkway attached to the Cass Street bridge crossing the Big Papillion Creek. The point is located on the left bank of said creek and is marked with a White Carsonite Post.
Observed: 2009-08-06T18:01:00Z
Source: OPUS - page5 0909.08

Close-up View

REF_FRAME: NAD_83 (CORS96)	EPOCH: 2002.0000	SOURCE: NAVD88 (Computed using GEOID03)	UNITS: m	SET PROFILE	DETAILS

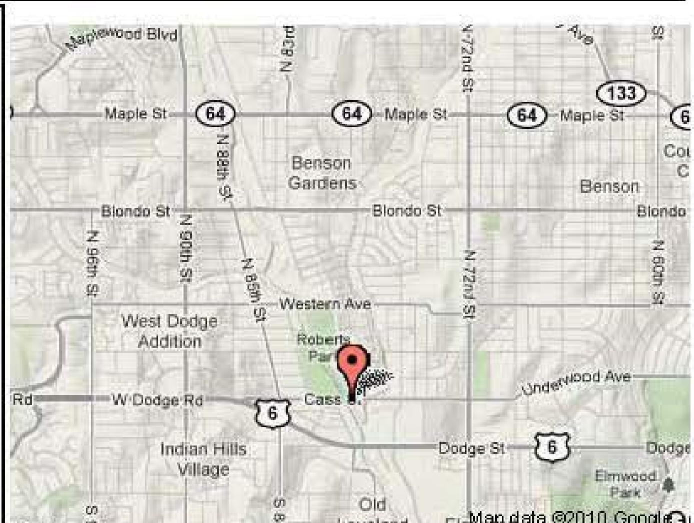
LAT:	41° 15' 47.39187"	± 0.035 m
LON:	-96° 2' 9.04502"	± 0.020 m
ELL HT:	290.587	± 0.034 m
X:	-504889.732	± 0.024 m
Y:	-4774966.285	± 0.034 m
Z:	4184627.958	± 0.029 m
ORTHO HT:	318.019	± 0.049 m

	UTM 14	**SPC** 2600(NE)
NORTHING:	4572209.641m	166359.449m
EASTING:	748310.334m	831967.948m
CONVERGENCE:	1.95591174°	2.62703272°
POINT SCALE:	1.00035890	0.99966752
COMBINED FACTOR:	1.00031330	0.99962196

CONTRIBUTED BY

david.d.salter
US Army Corps of Engineers

Horizon View



The numerical values for this position solution have satisfied the quality control criteria of the National Geodetic Survey. The contributor has verified that the information submitted is accurate and complete.

Figure 3-7. OPUS processed data sheet of a USACE PPCP. (Omaha District)

3-12. GPS Static Baseline Specifications for Networking Primary Project Control Point Connections to the NSRS. This section describes specifications to be used when CORS/OPUS solutions cannot be made (Preference [III] in Table 3-5) and networked static baseline observations must be observed and adjusted. Table 3-8 outlines the recommended GPS observing specifications needed to determine NAVD88 elevations relative to the NSRS based on a target accuracy of ± 0.25 ft. The DGPS static baseline observing specifications for network connections in Table 3-8 are largely tailored around current USACE EM 1110-1-1003 (*NAVSTAR GPS Surveying*) and NGS orthometric height guidelines for 2 cm to 5 cm accuracy orthometric network densification — *Guidelines for Establishing GPS Derived Orthometric Heights* (NOAA 2005). These GPS orthometric height guidelines in Table 3-8 have been modified to fit the nominal ± 0.25 ft accuracy requirements in Table 3-1.

a. GPS survey specifications. The following network connection specifications in Table 3-8 are intended to achieve the nominal target accuracy requirements for USACE primary project control. This is not to say that they will work in all cases, or in all locations, due to a variety of factors too numerous to list here. The bottom line is that on-site baseline reduction and processing software should readily (i.e., same or next day) identify the quality of the results from a constrained network adjustment statistical summary.

b. Data submittal to NSRS via Blue Book procedures. When OPUS-based NSRS submittal methods cannot be utilized, GPS observations and leveling observations to newly established PPCPs must be adjusted and submitted to the NSRS using NGS Blue Book procedures—"*Input Formats and Specifications of the National Geodetic Survey (NGS) Data Base*" (NOAA 1994). The Blue Book is a guide for preparing and submitting geodetic survey data for incorporation into the NSRS database. NOAA 1994 provides overall instructions and a checklist for submitting raw data, vector solutions, project and station data, station descriptions, applicable horizontal and vertical connections, least squares adjustments, a project sketch, and a project report. Additional guidance, tutorials, and required software are referenced therein with web addresses for downloading. It is recommended that the A-E performing the field surveys work directly with other firms that have an established record for producing accepted Blue Book submittals to ensure proper procedures and documentation are followed throughout the project.

c. Resultant accuracy estimates. Since the specifications in Table 3-8 have been modified from the NOAA 2005 specifications to meet USACE project orthometric accuracy requirements, it is important that published NSRS data sheets contain a statement to that effect. Including the estimated orthometric accuracy from the constrained network adjustment in that statement would be warranted.

Table 3-8. USACE Guidelines for Establishing GPS-Derived ± 0.25 ft Accuracy Orthometric Elevations on PPCPs using GPS Network Connections to NSRS Bench Marks.

Occupation time based on baseline distance to nearest two NSRS bench mark(s):

Distance	Time
< 20 km	30 min
20-40 km	60 min
40-60 km	180 min
60-80 km	240 min
80-100 km	300 min
> 100 km	> 5 hours

NGS pre-approval required	Yes (local NGS advisor, HQNGS, or NGS web site)
Number of days station occupied:	1 day (perform interim break-down and reset)
Dual-frequency receiver required:	Yes
NGS modeled geodetic quality antenna :	Yes (ground plane recommended)
Minimum number of observations per baseline:	2
Fixed-height tripods/poles:	Required
Satellite altitude mask angle (minimum):	10 degrees (collect) 15 degrees (process)
Maximum allowable VDOP:	5
Precise ephemeris:	Recommended, but not required
Geoid model:	Most recent
Add CORS baselines to adjustment:	Yes
Maximum distance to CORS points:	No restriction—weight accordingly with local NSRS baselines
NSRS input:	Blue Book or OPUS-based input

Table 3-8 (Concluded). USACE Guidelines for Establishing GPS-Derived ± 0.25 ft Accuracy Orthometric Elevations on PPCPs using GPS Network Connections to NSRS Bench Marks.

Notes:

Static (networked) DGPS baseline connections may be required in cases where the current geoid model has unacceptable accuracies in a particular region, such as in sparsely NSRS controlled mountainous areas, or in places where CORS stations are too distant—greater than 200 miles. Regardless, CORS baselines will be used in the adjustment if available.

DGPS network connection procedures will require considerably more field effort and must follow the guidelines in Table 3-8. Inputting networked DGPS observation data into the NSRS will currently require “Blue Booking.” However, it is expected that an alternate “Blue Booking” methods (i.e., OPUS) eventually will be available from NGS for adjusting traditional networked data and inputting results into the NSRS.

At least two baselines tied to or “networked” with nearby NSRS points should be observed. These local baselines will be combined with CORS baselines, and adjusted using NGS software routines.

Proposed observation schemes for networked baseline observations to nearby NSRS points shall be pre-approved by NGS. Pre-approval may be obtained from the local NGS geodetic advisor or from designated NGS HQ staff. The format for submitting proposed schemes should follow the “Project Proposal Form” available from the NGS.

3-13. The USACE Survey Monumentation Archival and Retrieval Tool (U-SMART). The U-SMART system, a fast and user-friendly web-based system (Figure 3-8), facilitates compliancy with the requirement to link the proper geodetic, hydraulic, tidal, and legacy control to their respective projects. Each project is required to have a minimum of 3 control points, one of which must be directly connected to and included in the NSRS. These control points must also be connected to the local water surface datum/model used for engineering designs and studies. U-SMART provides the tools to link local control networks together and link the project to the appropriate gage and the local legacy control points used for deformation studies and historical surveys.

a. U-SMART running as part of the web-based CorpsMap system serves as the liaison between USACE project control and the NGS/NSRS database. The U-SMART system continually monitors the NGS database looking for spatial changes, and alerts the user of required updates to local control. Benefits of U-SMART include:

(1) Common source for all project control.

(2) No desk drawer control files or duplicates.

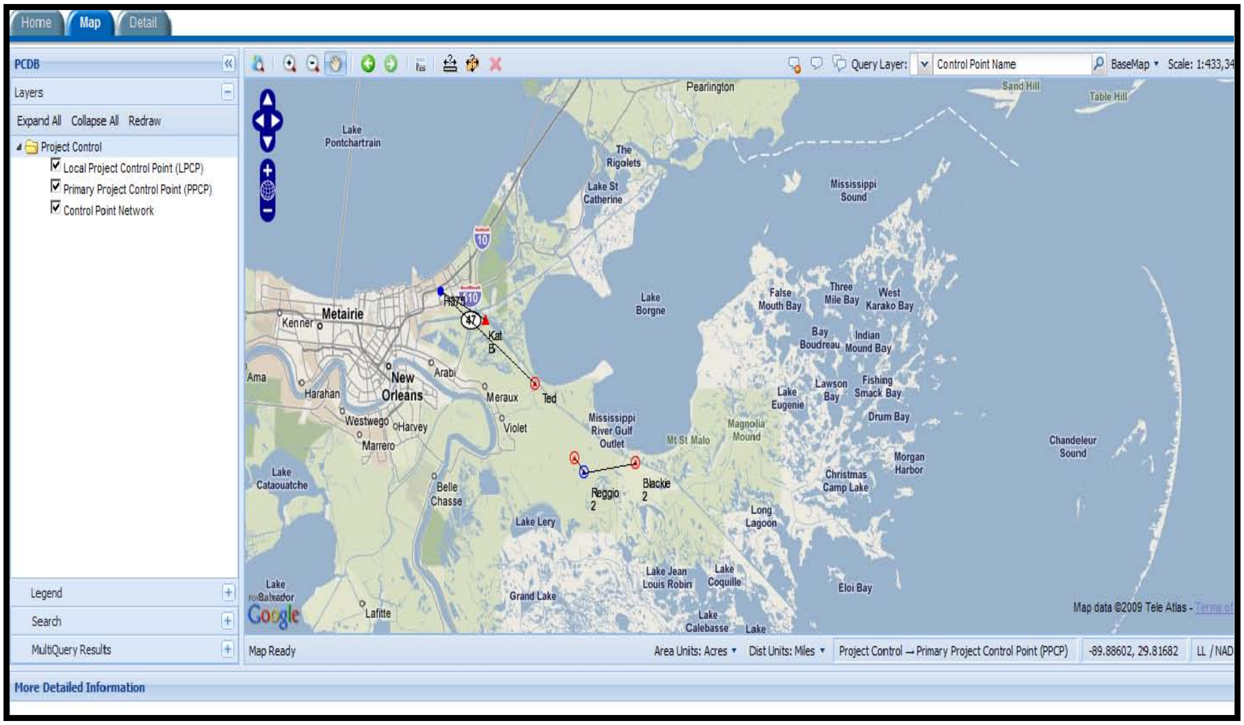


Figure 3-8. U-SMART web-based map interface indicating PPCPs and LPCPs.

(3) Entire District on same page.

(4) Compares coordinates against NGS database.

(5) Links project control to authorized projects.

(6) Links primary control to local project control networks.

(7) Minimized maintenance at the local level.

(8) Easy archival and retrieval.

b. Many USACE districts already have in place a system for archiving geodetic control which requires development, maintenance, and storage space. Many of these systems are not publicized, which limits their effectiveness in providing the common source of control information to USACE customers. Some districts with large resources have developed comprehensive systems to capture their survey control data while other districts with limited resources are still using hard copies in the old file cabinets. The 1990s vintage "SEMMS" control database system attempted to provide a national platform for USACE survey control; however, only a handful of districts are currently using it. It is critical that the data contained in the district's archives be maintained and available for designers, planners, and engineers to access. With a U-SMART central repository for USACE survey control, it is less likely that the wrong survey control will be used.

c. The Control Point Description Form (Figure 3-9) is used to input horizontal and vertical coordinates, images, and other information about the PPCPs and LPCPs in the U-SMART database. The form is a simple PDF document that allows the user to populate the appropriate fields depending on the purpose of the submittal. The form has several flavors and can be used for documenting newly established points, recovery notes, entering historical information on the mark such as local/legacy coordinates, and can also be used to edit existing information in the database. Easy to populate pulldowns, checkboxes, and radio buttons simplify the process of data entry. Maps, pictures, and images are also captured and archived with the form. Once the form is completed, the data is imported into U-SMART by the District Datum Coordinator or a representative with the delegated responsibility to insure the quality of the information.

d. For additional information on U-SMART contact the Army Geospatial Center. See the web link in Chapter 1.

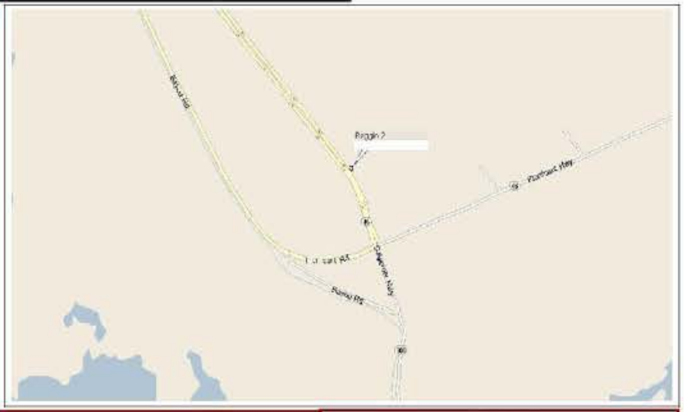
3-14. <u>Methods for Determining the Relationship between Legacy Project Datums and NAVD88</u>. Many of the legacy reference datums on USACE flood risk management projects, hurricane protection projects, river gages, reference pools, flow lines, flood stage, etc, are not referenced to the federal NSRS, and as such cannot be easily incorporated into hydrologic, hydraulic, flood inundation, and risk assessment models, or related to regional reference datums being used by other local, state, and federal agencies In CONUS the methodology used to shift historical or legacy survey data (e.g., NGVD29) to NAVD88 will vary depending upon many factors such as time, funding, accuracy requirements, etc. The most accurate and costly method is to re-observe each bench mark used for an old survey of interest. Even with the establishment of new elevations we can only estimate the changes that have taken place between then and now. The relationship between the surveyed features and the control marks may have also changed due to subsidence, settlement, or NSRS readjustments.

a. General. Transforming between legacy NGVD29 and NAVD88 is not straightforward, given NGVD29 has not been supported or updated by NGS since it was superseded in the early 1990s; thus, elevations still referenced to NGVD29 can have unacceptable vertical errors. Models have been developed for performing general "mapping grade" transformations from NGVD29 to NAVD88. These models (e.g., VERTCON and CORPSCON) were not intended to provide survey or construction quality accuracy, and must be used with caution given they are only coarse estimates. Floodwall or levee flood protection elevations should not be designed, constructed, or certified based on uncertain transformations from NGVD29 using CORPSCON.

b. Datum transform methods. Generally there are four methods to determine the datum/epoch shift. It is important to maintain the historical project files and source documents documenting what datum was used when, and how the project datums were derived. This is usually detailed down to the individual bench mark level of detail, documenting what bench marks were used and what elevations were used. Whichever method is used, transformations all have various elevation uncertainties, and it is important to have knowledge of the uncertainties of the final elevations used.

USACE Survey Marker Archive & Retrieval Tool Datasheet Type: New

Designation: Reggio 2
Project: MRGO
Stamping: Reggio 2
PID NGS: AT0804 COE:
State: Louisiana
County: St. Benard
District New Orleans
Nearest Town: Reggio
USGS Quad:
T.R.S.:
Nearest Hwy/Mi:
B/L Sta/Off:
Date Recovered: May 2007
By: Chustz
Condition/Stability: Good D
Setting/Monument Type: SS Rod
Owner:
GPS Suitable: (•) Yes ( ) No
Obstructions: [ ] N [ ] E [ ] S [ ] W



- Horizontal -

Datum: NAD83 (2002.00)
Lat: 29 50 40.71876 N
Lon: 89 45 32.43138 W
Local Accuracy: 1-cm
NSRS Accuracy: 2-cm
Survey/Computation Method:
Static GPS Network
Date Observed: Sep 1, 2009

- Vertical -

Datum: NAVD88 (2004.65)
Elevation Ht: 4.9
Ellip Ht: -79.163 Ft
Local Accuracy: 2-cm
NSRS Accuracy: 5-cm
Survey/Computation Method:
Static GPS Network
Date Observed: Sep 16, 2007 Geoid03

Description/Comments:
The station is 25.0 ft. northeast of centerline of north bound lanes of hwy, 3.6 ft. North from north end of a bridge concrete rail, 2.1 ft. northeast of a concrete curb and 1.5 ft. northwest of a concrete abutment wing. Station is a stainless steel rod accessed through a logo cap stamped- reggio2 1987, flush with top of logo sleeve cover missing otherwise in good condition.

Access: open

- Tidal/Hydraulic Gauge Relationships -

Owner/Code: USACE
Gauge ID: 56123 Epoch: 83-01

- Datum -	- Elevation -
LMSL	0.51
MLLW	-0.02
Select	
Select	

Zone: Northing: Easting: Convergence: CSF:

- Horizontal View -

- Close-Up View -

Required Fields In Red [Reset Form] [Submit] U-SMART ver 1.1 9/24/2009

Figure 3-9. Sample U-SMART datasheet at a PPCP. (New Orleans District)

(1) Field Measurements with Known Historical Elevation(s). This method will yield the most accurate values based on the historical reference bench marks. The reference bench marks will need to be recovered and occupied/surveyed using CORS/OPUS, RTN, RTK, or other methods, depending upon required accuracy. The difference between the legacy elevation used for the original survey and the NAVD88 elevation established from the new network will directly tie in the old work to the latest control. This will not account for relative differences between the project control and the project features, to include any differential subsidence or settlement that may have occurred after the legacy reference datum was established.

(2) Field Measurements without Known Historical Elevation(s). When the reference bench mark legacy datum has not been documented and unknown, some assumptions will be required, such as what bench mark was used and what its elevation was. Again, CORS/OPUS, RTN, RTK, or other methods depending upon required accuracy may be used to establish a new elevation on the reference mark. The historical elevation will have to be assumed based on what was available at the time of design. The difference between the assumed historical elevation and the newly established elevation will be used to shift the survey to the new datum/epoch.

(3) Common Published Marks in Survey Area. When time and money are constraints, the closest marks with published elevations in both datum/epochs can be used to determine an average shift for the area. This method contains many assumptions and therefore is the less accurate and contains more uncertainty but may be of use on some projects.

(4) CORPSCON or VERTCON conversions between NGVD29 and NAVD88. These conversions are approximate and do not account for subsidence or the changes in elevation from epoch to epoch. The conversion models were constrained to the published elevations at the time the conversion model was created (ca 1990). These models contained errors associated with the already deteriorating NGVD29 elevation accuracies in 1990. These methods should not be used for anything other than a simple datum shift, keeping in mind that subsidence is not accounted for.

c. FEMA guidance for converting to NAVD88. Appendix B in FEMA's "Guidelines and Specifications for Flood Hazard Mapping Partners" (FEMA 2003) contains standards and criteria for converting between NGVD29 and NAVD88 datums, including guidance for the conversion of unrevised flood elevations. These conversion standards are largely based on CORPSCON and VERTCON procedures. Methods for computing average datum conversions over a FIRM study region are defined. Maximum conversion tolerances of ± 0.25 ft are prescribed in this FEMA guidance.

3-15. <u>Summary of USACE Survey Standards for Connecting Projects to Nationwide Reference Datums</u>. Table 3-9 summarizes the recommended procedures and standards covered in this manual for connecting projects to the NSRS.

Table 3-9. Summary of Recommended Survey Standards for Referencing Grade Elevations on Navigation Projects, Multipurpose Projects, Levees, Floodwalls, and Related Retaining Structures.

	Inland Flood Protection & Navigation Projects [Rivers, lakes, reservoirs, pools, Great Lakes]	Coastal Hurricane & Shore Protection Projects Coastal Navigation Projects
<u>Reference Datums</u>:		
Hydraulic Reference Datum [1]	LWRP, pool, etc. (stage, flood profile) Hydraulically modeled from gages	Local MSL or Local MLLW (stillwater, surge, etc) Hydrodynamically modeled from NOAA tide gages
Geodetic (Orthometric) Reference Datum [1]	NOAA NSRS (NAVD88)	NOAA NSRS (NAVD88)
High subsidence or crustal uplift areas	NOAA Time Dependent Reference Network	NOAA Time Dependent Reference Network
Ellipsoid Reference Datum [Optional/Recommended Reference]	GRS80 (NAD83 NSRS)	GRS80 (NAD83 NSRS)
Legacy Geodetic or Tidal Datums	Sea Level, NGVD29, Cairo, MSL 1912, etc. Define relationship to NAVD88	MLW, Mean Low Gulf, Cairo, etc. Define relationship to NOAA Local MSL or Local MLLW

[1] The relationship between these two datums must be physically determined at PBM reference points and mathematically modeled throughout the project area.

Table 3-9 (Continued). Summary of Recommended Survey Standards for Referencing Grade Elevations on Navigation Projects, Multipurpose Projects, Levees, Floodwalls, and Related Retaining Structures.

	Inland Flood Protection & Navigation Projects [Rivers, lakes, reservoirs, pools, Great Lakes]	Coastal Hurricane & Shore Protection Projects Coastal Navigation Projects
Primary Project Control Points (PPCPs):		
Primary PBM published in NSRS	Yes	Yes (NOAA CO-OPS database if applicable)
Minimum PBMs required per project/segment	1	1
Spacing of primary PBMs NTE	15 – 20 miles (see Chapter 6)	15 - 20 miles (see Chapters 4 and 5)
Recommended min vertical accuracy	± 0.25 ft relative to NSRS	± 0.25 ft relative to NSRS
High subsidence or uplift areas	± 0.15 ft relative to NSRS	± 0.15 ft relative to NSRS
Recommended min horizontal accuracy (for mapping/GIS applications only)	± 2 ft relative to NSRS—NAD83	± 2 ft relative to NSRS—NAD83
GPS ellipsoid height observation at PPCP	Recommended	Recommended

Table 3-9 (Continued). Summary of Recommended Survey Standards for Referencing Grade Elevations on Navigation Projects, Multipurpose Projects, Levees, Floodwalls, and Related Retaining Structures.

	Inland Flood Protection & Navigation Projects [Rivers, lakes, reservoirs, pools, Great Lakes]	Coastal Hurricane & Shore Protection Projects Coastal Navigation Projects
Water Level Gages: [2]		
Primary gage reference PBM connected to	NSRS (NAVD88)	NSRS and NOAA CO-OPS tidal network
Minimum number of reference PBMs at gage	3	5 (NOAA tide gage)
Primary reference PBM metadata repository	NSRS and/or U-SMART	NSRS & NOAA CO-OPS database; U-SMART
Periodic gage inspection metadata repository	NSRS and/or U-SMART	NSRS & NOAA CO-OPS database; U-SMART
Update low water or tidal datums	per Division/District H&H requirements	per NOAA published revisions (typically 19 years, or 5 years in high subsidence areas)

[2] See Chapter 4 for additional details

Table 3-9 (Continued). Summary of Recommended Survey Standards for Referencing Grade Elevations on Navigation Projects, Multipurpose Projects, Levees, Floodwalls, and Related Retaining Structures.

	Inland Flood Protection & Navigation Projects [Rivers, lakes, reservoirs, pools, Great Lakes]	Coastal Hurricane & Shore Protection Projects Coastal Navigation Projects
RECOMMENDED STANDARDS:		
Local Project Control Points (LPCPs) for Design & Construction:		
PBMs spaced	as required	as required
Reference datum and/or NOAA tidal	Primary project control (NSRS)	Primary project control (NSRS)
Minimum number of PBMs for construction contract plans & specifications	3	3

Table 3-9 (Concluded). Summary of Recommended Survey Standards for Referencing Grade Elevations on Navigation Projects, Multipurpose Projects, Levees, Floodwalls, and Related Retaining Structures.

	Inland Flood Protection & Navigation Projects [Rivers, lakes, reservoirs, pools, Great Lakes]	Coastal Hurricane & Shore Protection Projects Coastal Navigation Projects
Site Plan Topography Accuracies:		
Reference datum	Local project PBM control	Local project PBM control
Hard topographic features: Floodwall cap elevations, culverts, inverts, first floors, boring references, etc.	± 0.3 ft	± 0.3 ft
Ground shots on levee (profiles or cross-sections)	± 0.5 ft	± 0.5 ft
Floodplain topography (general mapping)	± 0.5 ft to ± 2 ft relative to NSRS	± 0.5 ft to ±2 ft relative to NSRS
Construction stake out—set hubs to	± 0.01 to 0.05 ft	± 0.01 to 0.05 ft

CHAPTER 4

Procedures for Referencing Datums and Dredging Grades on Coastal Navigation Projects

4-1. General. This chapter provides guidance on evaluating and establishing vertical reference grades on coastal navigation projects in tidal waters. It covers the procedures needed to ensure these projects are adequately referenced and modeled relative to the National Water Level Observation Network (NWLON) tidal datum and the National Spatial Reference System (NSRS) orthometric datum established by the Department of Commerce as outlined in Chapter 1 and ER 1110-2-8160. It also covers the tidal gaging and modeling methods used to define the varying MLLW datum plane at a project site, including NOAA's recently developed "VDatum" software tool that transforms vertical navigation datums throughout CONUS coastal regions. This chapter also discusses real-time GPS/RTN survey methods that are employed to measure the local water surface elevation relative to the MLLW datum. Much of the guidance in this chapter is also applicable to hurricane and shore protection projects covered in Chapter 5.

a. Scope. In coastal areas, and in coastal inlets, accurately modeling and measuring the varying tidal datum plane (e.g., LMSL or MLLW) relative to NAVD88 and the NAD83/GRS80 ellipsoid is the challenge. Measurement of the elevation of the actual water surface relative to the tidal reference datum must be done accurately in order to determine the acoustically surveyed depth/elevation of a point relative to the tidal datum. This water surface elevation varies temporally due to tidal phase latencies, tidal currents, and meteorological effects such as wind. This chapter provides procedural information and guidance to ensure survey and dredge positioning systems are effectively compensating for these tidal variations and other effects in coastal regions and inland rivers subject to fresh water flow and tidal influence. Non-tidal inland river, pool, reservoir, and lake datums are covered in Chapter 6.

b. Requirements to reference coastal navigation projects to NOAA MLLW datum. In accordance with the intent of Section 224 of WRDA 1992 and "The National Tidal Datum Convention of 1980" (NTDC 1980), navigation projects in coastal tidal areas must be defined relative to the datum shown on official NOAA navigation charts and NOAA tidal predictions for the project area. The WRDA 1992 amendment to Section 5 of the Rivers and Harbors Appropriation Act of 1915, which is excerpted below, supersedes previously authorized reference datums (e.g., Mean Low Water on Atlantic and Gulf coasts), and specifically directs that the datum defined by the U.S. Department of Commerce be used.

> *Section 5 of the Act of March 4, 1915 (38 Stat. 1053; 33 U.S.C. 562), is amended -- (as indicated). "That in the preparation of projects under this and subsequent river and harbor Acts and after the project becomes operational, unless otherwise expressed, the channel depths referred to shall be understood to signify the depth at mean lower low water as defined by the Department of Commerce for nautical charts and tidal predictions in tidal waters tributary to the Atlantic and Gulf coasts and at mean lower low water as defined by the Department of Commerce for nautical charts and tidal predictions in tidal*

waters tributary to the Pacific coast and the mean depth for a continuous period of fifteen days of the lowest water in the navigation season of any year in rivers and nontidal channels, and after the project becomes operational the channel dimensions specified shall be understood to admit of such increase at the entrances, bends, sidings, and turning places as may be necessary to allow of the free movement of boats.

USACE projects that are still defined relative to non-standard or undefined legacy datums (e.g., Mean Low Gulf (MLG), Gulf Mean Tide, MSL, NGVD, MLW, COEMLW, etc.) should have technically valid transforms to the NOAA MLLW chart/tidal datum for the area. In isolated cases, the legacy datum may be retained as the reference grade provided its relationship to NOAA MLLW datum is accurately defined based on current gage data at the project site. In such projects, depth data furnished to NOAA and other project users must indicate the primary reference gage, along with the tidal datum epoch period and the relationship between the legacy datum, NOAA MLLW, and NAVD88. Legacy "Low Water" datums must be periodically updated for sea level change and regional subsidence using similar computational techniques established by NOAA for coastal waters. Refer to Appendix C for additional details on referencing coastal projects to the federal MLLW datum.

c. References. This chapter does not cover the detailed theory, principles, and computational procedures for establishing tidal datums from observed gage data, or for performing hydrodynamic tidal modeling of navigation projects. For more technical information on these topics consult the USACE and NOAA technical publications listed in Appendix A.

4-2. <u>Overview of Procedures Needed to Reference Grades on Navigation Projects</u>. Figure 4-1 illustrates the various datum relationships that will need to be established to ensure a navigation project complies with the requirements in ER 1110-2-8160. Actions to develop these relationships are summarized below. Subsequent sections in this chapter detail specific procedures for each of these actions.

a. Primary Project Control Point (PPCP) reference. Designate a NSRS published PPCP(s) needed to position survey and dredging operations over the project reach using RTK techniques. The PPCP shown in Figure 4-1 provides RTK coverage over the entire project reach. Alternatively, a RTN may be utilized, provided that it is "site-calibrated" to NSRS tidal bench marks. The PPCP should have horizontal coordinates (NAD83) of sufficient accuracy (< ± 2 ft relative to the NSRS) to position survey and dredging operations. As shown in the figure, the PPCP provides the relationship between NAVD88, NAD83/GRS80 ellipsoid, MSL, and the Geoid (geoid height), and perhaps a legacy datum such as NGVD29. Its vertical accuracy (< ± 0.25 ft relative to NAVD88) is usually adequate for RTK initialization; site calibration being performed relative to the NOAA tidal gage reference bench marks shown in Figure 4-1. Further details are covered in Section 4-3.

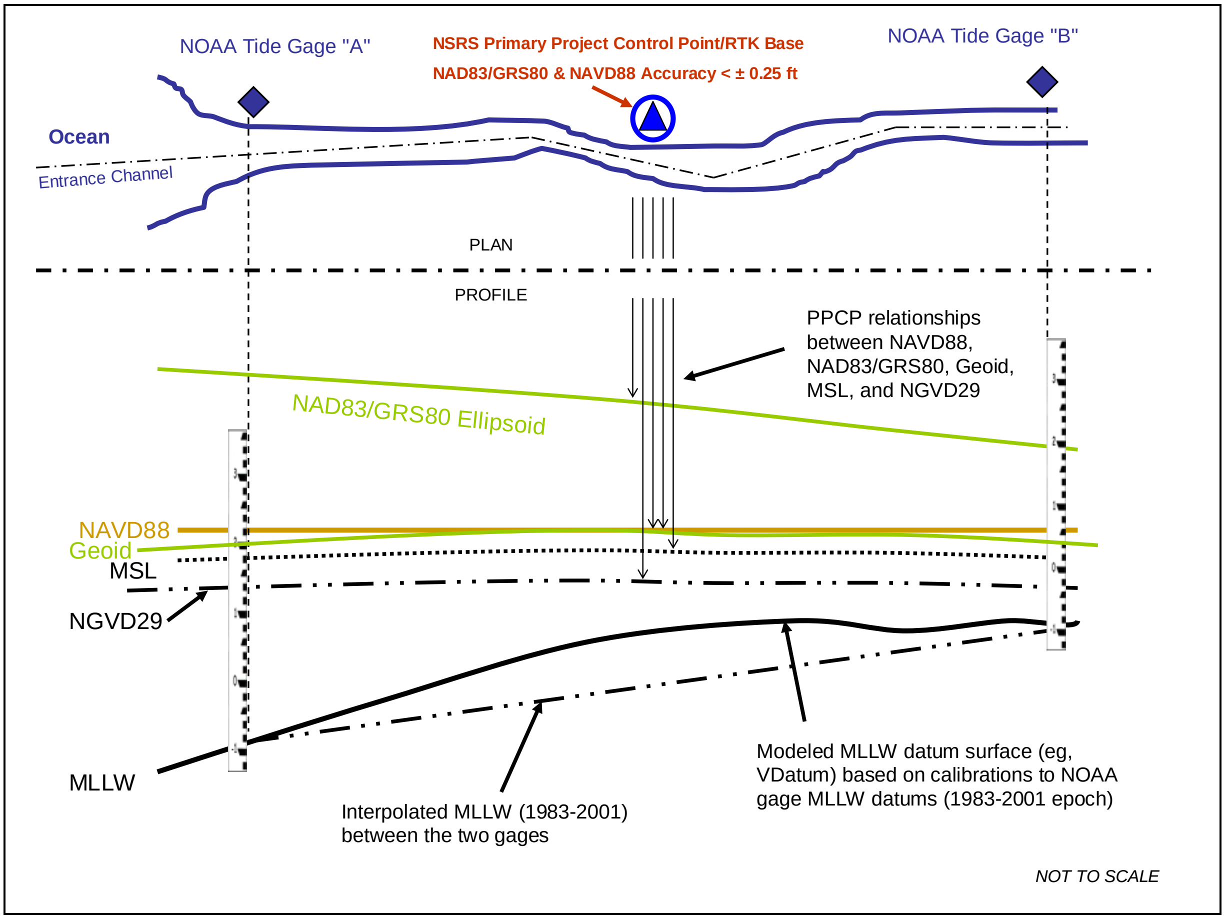


Figure 4-1. Geodetic and tidal datum relationships at a typical coastal entrance navigation project.

b. Tide gage and tidal bench mark references. Designate tide gages and tidal bench marks that reference the dredging datum for the project. These gage sites shall be used for calibrating RTN/RTK positioning systems to MLLW. Published MLLW elevation data for reference PBMs at these gages becomes the reference dredging datum for the project. Gages from any agency (e.g., NOAA, USACE, USGS, States, etc.) may be used; however, the MLLW reference datum must be current and referenced to the latest official NOAA National Tidal Datum Epoch NTDE). Depending on the size of the project and tidal characteristics, more than one gage may be required. The two gages shown in Figure 4-1 provide redundant RTK calibration points, at the entrance and upstream from the entrance. If only one gage at the entrance existed on this project (i.e., NOAA Tide gage "A") then the upstream gage "B" would have to be established in order to adequately model the reference MLLW surface. Further details on gage references are covered in Section 4-4.

c. Geoid model. Designate a geoid model for reducing observed RTK ellipsoid heights of the local water surface to the reference orthometric datum—NAVD88. As shown in Figure 4-1, the reference ellipsoid, geoid, and NAVD88 are not parallel and differ spatially over the project. The geoid (i.e., NGS "hybrid" Geoid XX) will match NAVD88 at some NSRS benchmarks but may deviate slightly away from those fixed points. GPS receivers and hydrographic survey systems provide software to model and compute in real time the relationship between the NAD83/GRS80 ellipsoid and the geoid, as necessary to compute NAVD88 elevations at any point on the project. Further details are covered in Section 4-7.

d. MLLW tidal model. Designate a MLLW tidal model that provides the relationship between NAVD88 and the MLLW datum at any point on the project. As shown on Figure 4-1, the MLLW tidal model may be based on a simple interpolation between the gages, or by a hydrodynamic tidal model, such as "National VDatum," that refines the actual MLLW variations due to topographic and bathymetric effects. As shown in the figure, the tidal model must also be related to the current NTDE. Further details on tidal models and VDatum are covered in subsequent sections in this chapter.

e. Tidal phase and water surface elevation corrections. Designate procedures used to correct for tidal phase and hydrodynamic/meteorological effects on the water surface elevation throughout the survey area relative to the location of the reference gage. This correction is not shown in Figure 4-1; however, the magnitude of this correction can far outweigh errors in tidal modeling. Details on the use of RTN/RTK methods to correct surface elevation measurements are covered in Section 4-7.

4-3. Establishing Primary Project Control Point (PPCP) References. This section provides guidance on establishing PPCPs needed to reference excavation grades on a navigation project.

a. Orthometric and tidal datum relationships. As outlined in Chapter 1, it is essential that the relationship between geodetic, tidal, and ellipsoidal datums be firmly established at a navigation project. This relationship is essential for determining the water surface elevation using RTK survey methods.

(1) Figure 4-2 illustrates the relationship between these datums at a PPCP and a tide gage. The tidal bench mark PBM A “000 9999 A” is used to reference the gage and contains only the elevation relationship between the gage zero and the various computed tidal datums. It does not have any geodetic datum elevation, which is common at many historical NOAA tide gage sites.

(2) In Figure 4-2, a nearby, published NSRS geodetic bench mark (PBM B “USED 123”) has established orthometric (NAVD88) and ellipsoidal heights, based on precise geodetic leveling and long-term static GPS observations to surrounding NSRS points. In cases where the NSRS mark has not been connected by precise geodetic leveling, the NAVD88 orthometric height may have been computed based on a GPS ellipsoid height observation coupled with the estimated geoid height, as was illustrated in Chapter 2.

(3) PPCP "USED 123" would likely be used as a reference base for RTK surveys of the project, and the tidal bench mark "000 9999 A" would be used to "site-calibrate" the RTK system to the reference dredging datum (e.g., MLLW and/or LWRP). The relationships between the datums at each PBM can be obtained by field leveling or GPS surveys connecting the two bench marks. This field survey effectively establishes the datum relationships at both PBMs.

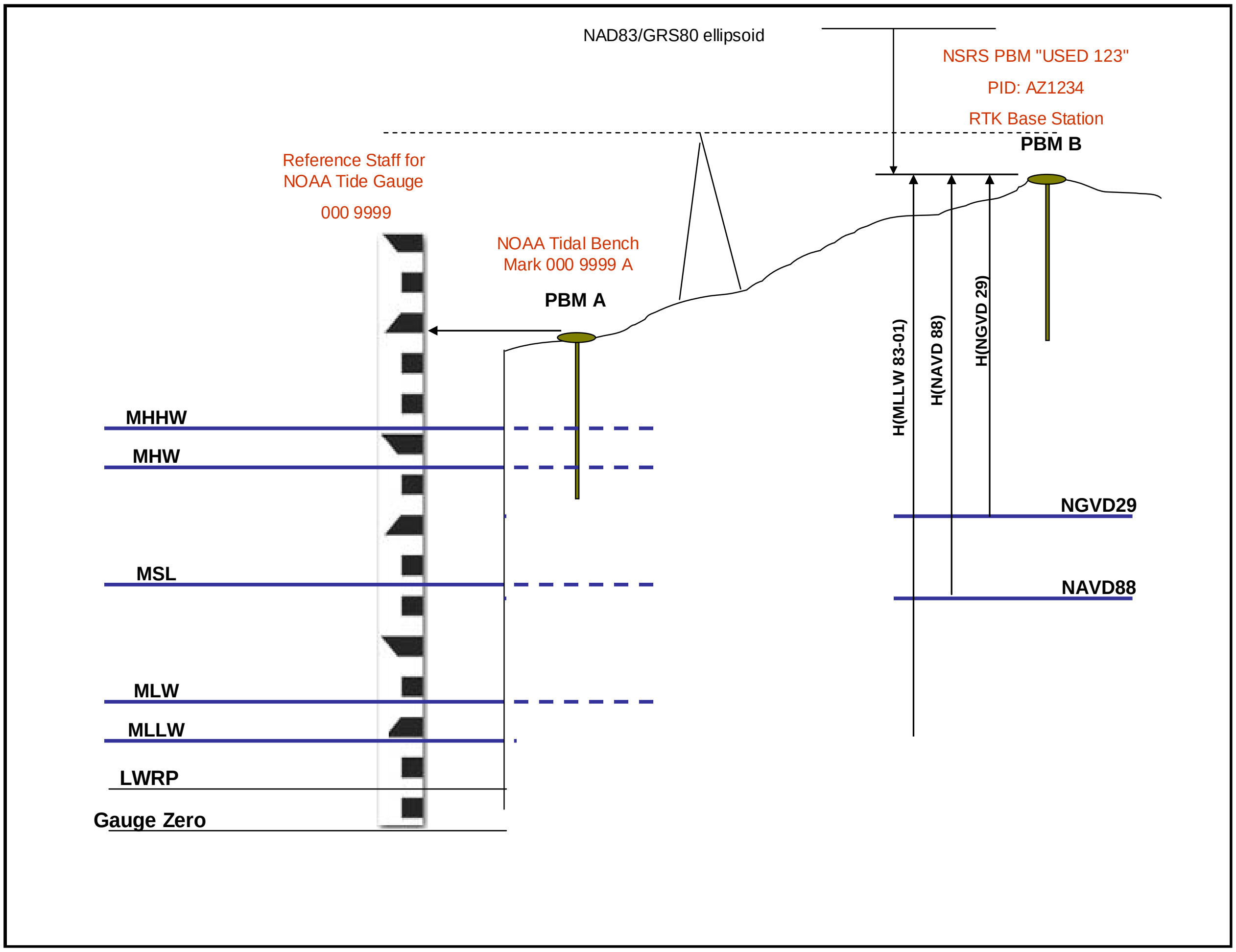


Figure 4.2. Establishing the relationships between orthometric datums and tidal datums at a gage site.

(4) If this gage in Figure 4-2 were located in a river transition area, a hydraulic Low Water Reference Plane (LWRP) stage, or a specified river datum such as the Columbia River Datum, is also defined at the gage site. Both tidal and non-tidal low water (or low flow) datums overlap at this site. See Chapter 6 for additional details on non-tidal river (low water) reference planes.

b. Establishing RTK base stations. RTK base stations on a PPCP can utilize either an existing NSRS PBM near the project site or a USACE PBM that is connected to the NSRS. In most CONUS coastal locations, CORS/OPUS observations are suitable and recommended for establishing a new PPCP or establishing an NAVD88 elevation on a published NSRS point with only horizontal control. Rarely would a long-term static GPS observation network be required to establish a new PPCP. The nominal PPCP accuracy standard of ± 0.25 ft (X-Y-Z) should be adequate for an RTK base, noting that the "Z" (MLLW) calibration is performed relative to a tidal PBM, not the PPCP elevation determined from a CORS/OPUS observation.

(1) RTK or RTN coverage in the project area must be assessed to determine the number of PPCPs needed to reach the project limits. In areas beyond reliable real-time data links, post-processed kinematic (PPK) procedures may be an option. In large open bays, and beyond reliable single-base RTK positioning limits, a PPK solution may be a necessity.

(2) In areas covered by government or commercial RTNs, an NSRS PBM is required near the project site to perform RTN site calibration.

(3) If static GPS network observations are needed to establish NSRS control on a PPCP or tidal PBM, then the procedures outlined in Chapter 3 should be followed. This may be necessary in isolated project areas or in OCONUS.

c. Connecting NOAA tide gage reference bench marks to the NSRS (NAVD88). It is desirable to reference MLLW datums at tidal bench marks to NAVD88. In order to support NOAA's program to update tidal bench mark elevations to NAVD88, tidal bench marks may be positioned using the CORS/OPUS methods described in Chapter 3. These GPS elevation observations will be input into the NSRS using the procedures described in Chapter 3. Recovery notes and updated descriptions on CO-OPS tidal bench marks not yet published in the NSRS (but published in the NWLON database without a NGS "PID" link) should be transmitted directly to CO-OPS.

(1) In some cases, NOAA tidal bench marks may be used as a PPCP for an RTK base if they are more suitable than the NSRS PPCP. In Figure 4-2, survey connections by differential leveling from the NSRS PBM "USED 123" to the tidal bench mark "000 9999 A" would provide adequate X-Y-Z control on the tidal bench mark to be used as the RTK base.

(2) If the NOAA tidal bench mark is distant from the nearest NSRS PPCP, then CORS/OPUS observations at the tidal bench mark are recommended; establishing $< \pm 0.25$ ft horizontal and vertical accuracy on this point which is adequate for initializing RTK observations.

d. Summary. Figure 4-3 outlines the decision flow process involved in establishing and designating a PPCP used for RTK control on a navigation project.

4-4. Designating a Primary Tidal Reference Gage for a Navigation Project. All navigation projects must have one or more primary tidal bench marks that are directly referenced to an established tide gage. The gages must adequately model the project area and be suitable for RTK calibration purposes. A gage's computed reference datum (e.g., MLLW or LMSL) shall be based on relatively current observations and shall be referenced to the latest NTDE established by NOAA. The gage shall also have a sufficient number of tidal reference bench marks. The procedures for computing the reference MLLW datum at the gage shall be consistent with NOAA standards and specifications. This section describes the process for evaluating the adequacy of existing gage data at a project site, and if deemed inadequate, the steps needed to establish a new reference datum.

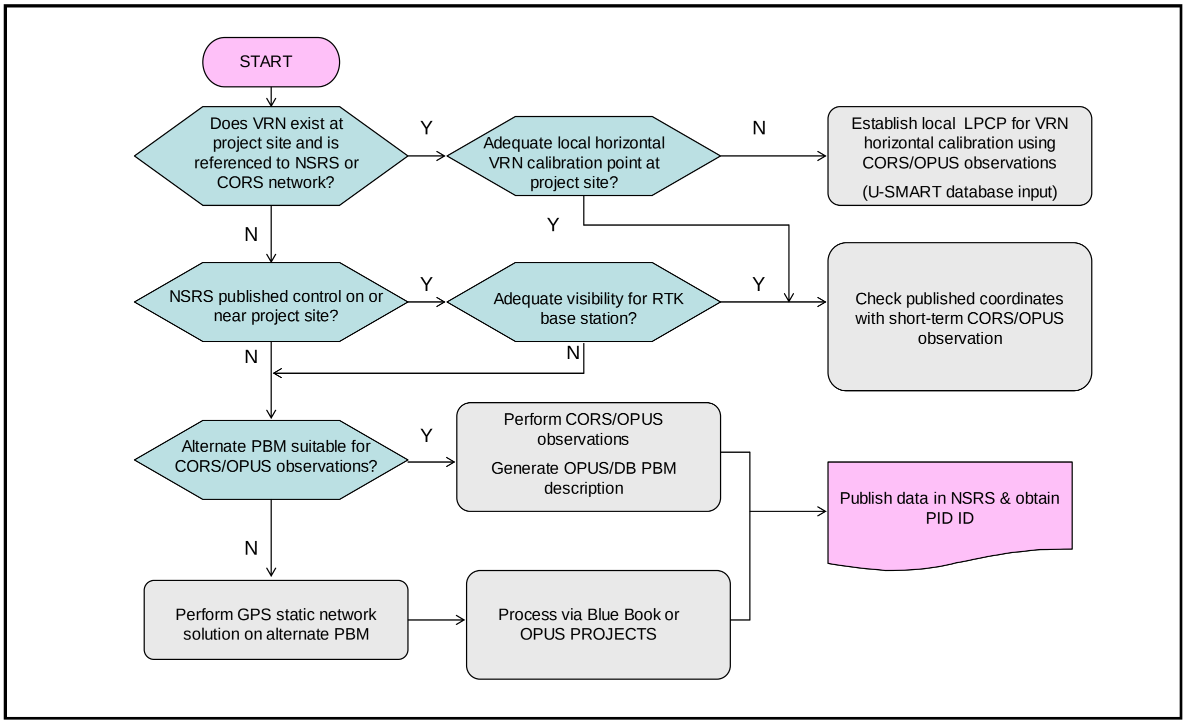


Figure 4-3. General decision process for establishing a navigation project PPCP.

a. Assessing the quality of existing tide gages and computed water level datums. The NWLON is the NOAA nation-wide network of permanently operating tide gages with accepted tidal datums and published bench mark elevations. It also includes the network of historical NOAA tide gage locations that have published tidal datum elevations relative to the latest NTDE. The navigation reference datum on a project site must be adequately connected with this NOAA NWLON network. This implies using either a NOAA gage site that is on, or is connected with, the NWLON, or a locally operated gage that conforms to NOAA/CO-OPS specifications. Isolated bench marks (those of USACE or any other agency) that purport MLLW or MSL reference elevations should be considered highly suspect unless their connection with a NWLON gage site can be firmly established. These connections are usually performed by simultaneous comparison methods, direct differential levels, or static GPS connections to a NOAA tidal bench mark. Any such marks must also contain an NTDE designation attached to their elevation that signifies it has been adjusted to the current NOAA tidal epoch.

(1) Use of active or historical NOAA gages. Published NOAA tidal bench marks are found on or near the vast majority of USACE deep-draft and many shallow-draft projects. These tidal PBMs may be referenced to an active NOAA gage or a historical NOAA gage with archived tidal data. Since few of USACE's 900+ navigation projects have actively operating NOAA gages, the adequacy of historic gage data must be evaluated. This would include assessing the period of record, the age of the data, subsequent channel deepening or realignment, inlet changes, jetty or breakwater modifications, etc. These physical changes may have modified the tidal characteristics since the gage data were recorded. For example, datums at a site computed from a 30-day series in 1970 may be suspect, particularly if subsequent construction or other physical changes have modified the tidal characteristics in the area. NOAA has dropped

published bench marks sheets from historical short-term stations that were established prior to the 1970's. In such cases, a new gaging program to update the reference datum may be warranted. Procedures for establishing tidal datums using short-term gage observations are described in "*Computational Techniques for Tidal Datums Handbook*" (NOAA 2003).

(2) Tidal bench mark recovery at historical NOAA gage sites. Tidal bench marks at historical NOAA gage sites are often lost or impossible to recover due to dated descriptions. Ideally, at least two tidal PBMs should be recovered to have confidence in the stability of these marks and their reference MLLW datum. Third-Order leveling procedures are considered adequate for this purpose. Additional tidal PBMs should be set such that a total of three to five reference marks are available at the gage site. One of the tidal bench marks should be designated as the primary "PPCP" tidal datum reference for the project and placed in the NSRS.

(a) If only one tidal PBM is recoverable, then the long-term stability of that PBM must be assessed if it is to be used as a primary reference. If no PBMs are recoverable, then a new gaging program would likely be warranted to reestablish the tidal datum—especially on deep-draft projects.

(b) Exceptions to the above may exist at less critical shallow-draft projects with reliable VDatum coverage. In such cases, the VDatum estimate of the NAVD88-MLLW difference may be used. The NOAA Coast Survey Development Laboratory (CSDL) "VDatum Team" should be consulted before making this determination.

(3) Use of other agency tide gages. Many other local, state, and Federal agencies including USACE operate tide gages that may be used to reference navigation datums. As with NOAA gages, the quality of the gage data (e.g., datum computation) and reference bench marks must be assessed. Often these gages are referenced to only one bench mark. The stability and quality of this single reference bench mark must be evaluated. If this gage is to be used as a project reference, then additional reference PBMs should be set with at least one PBM in the NSRS. In any case, for hydrographic survey tidal control, a current NTDE MLLW elevation must be established.

(a) Figure 4-4 illustrates NAVD88 orthometric connections to tide gages from other agencies that may be in the vicinity of the project area. Static GPS baselines are observed from published NSRS control points in the region to a local LPCP set near the gage site. Gage reference PBMs and staff zeros are leveled in from the LPCP. The gage datum relationship to NAVD88 is documented as shown in the figure. Additional gage reference PBMs are set in the vicinity of the gage.

(b) Figure 4-5 shows a case where the original gage reference point is updated and a new reference LPCP PBM is set. The primary reference PBM should be documented in the NSRS and additional gage reference PBMs established. Gage reference points must be clearly documented as shown in the figure.

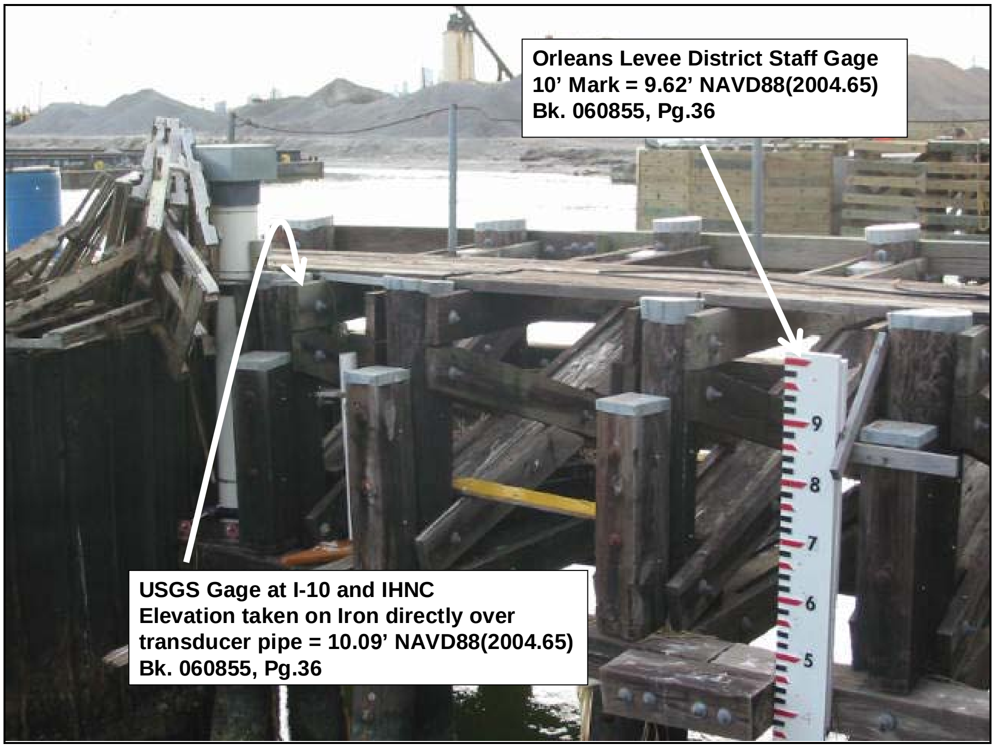


Figure 4-4. Gage reference elevations (USGS and Orleans Levee District gages at I-10 and Inner Harbor Navigation Canal (IHNC)—from IPET 2007).

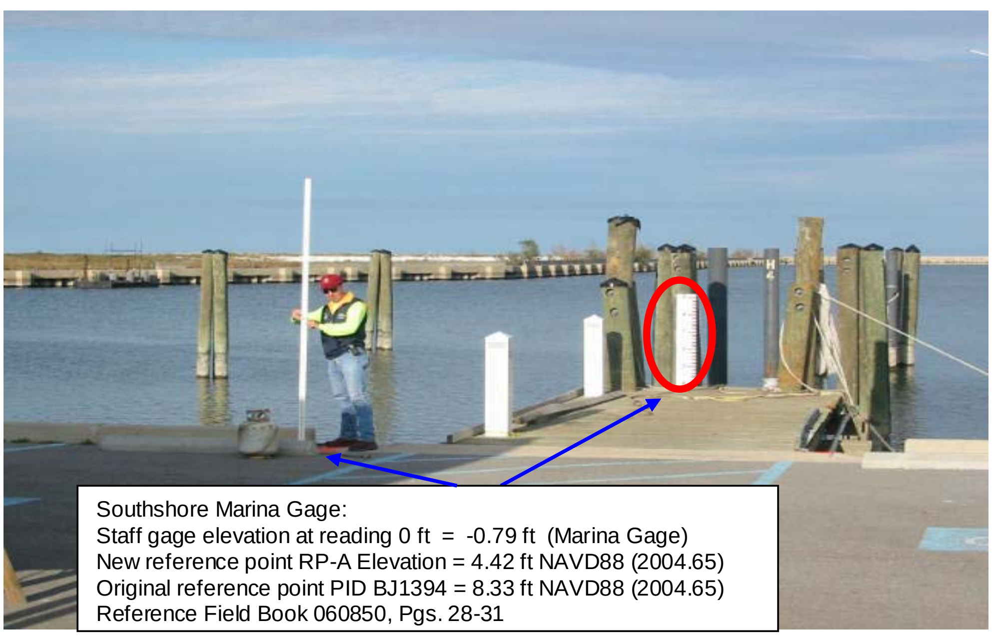


Figure 4-5. Revised gage reference points and elevations. The above photo shows location of PBM "RP-A" and red circle shows staff gage.
(Orleans Levee District gage at Southshore Marina, Lake Pontchartrain--from IPET 2007)

b. Projects with inadequate reference gages or undefined datums. Many USACE navigation projects, particularly isolated shallow-draft projects, do not have tidal datum references that can be reliably related to the current NOAA NWLON gage network. Typically, these projects have been referenced to a local datum or a legacy NGVD29 reference. These legacy reference datums may or may not be referenced to any tide gage. In other cases, the density of gages is inadequate to model the tidal regime over the project, or VDatum coverage may not extend to the head of the maintained navigation channel. This may occur on intracoastal waterways or on projects where the head of maintained navigation is distant from the coastal entrance. A number of options exist to bring these projects into compliance with ER 1110-2-8160, all of which are dependent on status and use of the project, commercial traffic, and related funding availability. These options may include:

(1) In coordination with NOAA CO-OPS, install a short-term tide gage to develop an updated reference datum using simultaneous comparison techniques relative to NOAA NWLON primary or secondary gages (see NOAA 2003). A 30-day observation period will suffice for most USACE navigation projects; however, 90-days are preferred by NOAA for QC purposes and to minimize datum errors. Longer gage observations (e.g., 3 to 12 months) may be required on more critical deep-draft projects. Less critical shallow draft projects may be effectively referenced to the NWLON with 7-day simultaneous gage observations where datum errors are not deemed critical to the project.

(2) If the project area is covered by a NOAA VDatum model, use this model to estimate the tidal datum relationship relative to NAVD88 on an established PPCP used for referencing RTN surveys. Check with NOAA CO-OPS as to the reliability of the VDatum model in the area. This would represent an interim solution for non-critical shallow draft projects with no maintenance funding. The NOAA VDatum model must be calibrated at existing or historic gage sites—see Appendix D. If no historic tide gage data exists for the project, then a gage may need to be established to calibrate the VDatum model.

(3) Maintain a legacy tidal datum reference noting the uncertainty of this reference on all published data for the project. VDatum may be used to estimate the datum relationships at the project site. This option may be applicable for inactive, soft-bottom, shallow draft projects that have not had any significant funding or maintenance activity in decades (i.e., funding a $50,000 to $100,000 tidal gaging and modeling program could not be economically justified).

c. Navigation project tide gage and modeling options. Figure 4-6 illustrates five of the more common cases of tide gage and tidal model coverage found on USACE navigation projects. The following sections outline possible corrective actions needed to bring the project into compliance with ER 1110-2-8160. Larger projects may require more than two gages to calibrate VDatum models for referencing hydrographic surveys and dredging operations—e.g., Tampa Harbor, Florida as illustrated in Appendix D.

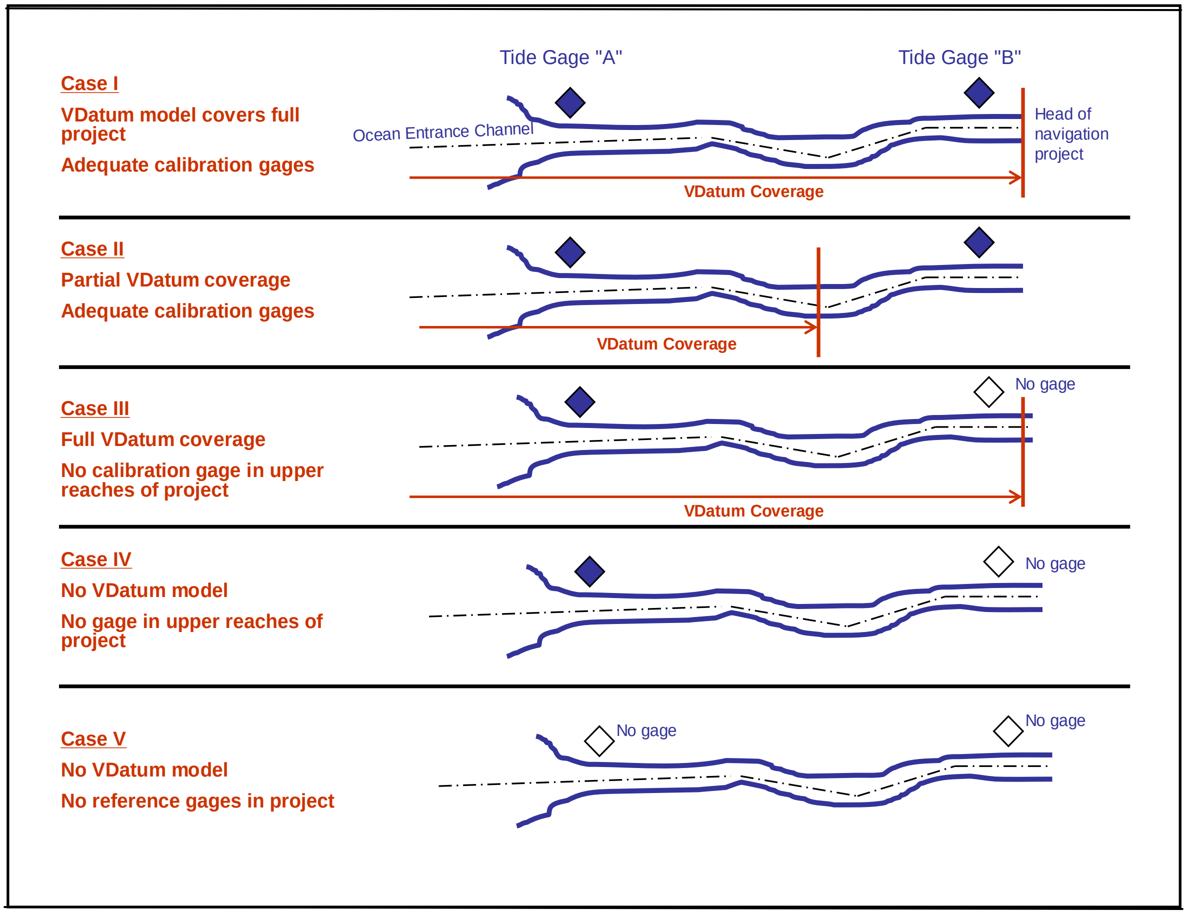


Figure 4-6. Tide gage and VDatum coverage cases that may exist at a navigation project. RTK/RTN surface elevation measurement is assumed. Tide gages are referenced to NOAA NWLON network.

(1) Case I. This case has adequate gage and MLLW datum model coverage to calibrate and reference RTK/RTN surveys over the entire project.

(2) Case II. Although existing tide gages are sufficient to reference the project, VDatum coverage does not extend over the entire project. In this case, the MLLW datum model would have to be interpolated between the VDatum model limit and the upstream gage.

(3) Case III. This case is an example of inadequate gage coverage to calibrate the VDatum model in the upstream reaches, and to reference surveys in these upper reaches. The VDatum model may adequately depict the MLLW reference surface throughout the project but reliance on one calibration gage at the entrance may be problematic if the distance upstream and tidal range variation is significant. For small, shallow draft projects, a single calibration/reference gage may be adequate. Likewise, small deep-draft projects only a mile or two inland from the entrance can be covered by a single gage. For projects of extended lengths upstream, an additional reference gage needs to be established. To correct this case, a short-term tidal comparison relative to the

entrance gage would be performed. This may entail establishing a temporary gage for a comparison period of 3, 7, or 30 days, depending on the tidal characteristics in the area.

(4) Case IV. This case is similar to Case III except there is no VDatum model covering the project. As in Case III, an upstream tide gage would need to be established to perform a datum comparison with the entrance gage. The MLLW reference plane between the two gages could be modeled by spatial interpolation methods. The offshore entrance channel datum relationship would have to be extrapolated from the entrance gage, unless other tidal data are available. More robust tidal modeling methods described in Section 4-6 would better model the river and offshore entrance channel.

(5) Case V. This is a "worst case" condition—no existing or historic gages at the project. Depending on various funding and maintenance levels, a full gaging program would be needed to model the project. Two short-term gages would be established for periods of 30 days or more, from which datum comparisons are made with nearby NOAA NWLON stations. The MLLW model would be developed using one of the methods described in Section 4-6.

d. NOAA requirements for short-term tide gages needed to update tidal models at a navigation project. When historical NOAA tide gage sites are occupied, or additional gaging data are needed to model the tidal regime at a navigation project, NOAA requires the following minimum standards in order for the site to be included in the CO-OPS database.

(1) Types of recording gage. At a new site, any NOAA approved type of temporary gage that can measure recorded water levels at 6-minute intervals is suitable. The gage must be firmly tied in and referenced to the local tidal bench marks at the site.

(2) Location of temporary gage. As needed to cover the navigation project and survey calibration. To be coordinated with NOAA CO-OPS.

(3) Length of record. Minimum of 30 days. Longer term if required by NOAA CO-OPS. A shorter term—3 to 7 days—may be used for adding gages within projects for use in calibrating hydrodynamic models and referencing RTK/RTN hydrographic surveys.

(4) Tidal bench marks. Five bench marks are required around the gage site. Follow mark construction requirements outlined in EM 1110-1-1002 (*Survey Markers and Monumentation*). No deep driven rods are required. Type C, F, and G marks are acceptable.

(5) Data format and submittal. Follow NOAA CO-OPS submittal requirements.

(6) Datum transfer computations. Follow NOAA CO-OPS simultaneous comparison standards—see NOAA 2003. NOAA CO-OPS will check datum transfer computations if they are performed in-house or by an A-E.

(7) Third-Order leveling between tidal bench marks. Follow standard procedures in EM 1110-1-1005 (*Control and Topographic Surveying*) for both new and existing gage sites.

(8) Primary tidal bench mark elevation. Tidal bench marks at both new and existing sites will be referenced to and input to the NSRS using CORS/OPUS input methods outlined in Chapter 3.

Detailed procedures for establishing tide gages and computing tidal datums at navigation projects can be found in Section 4 (*Tides and Water Level Requirements*) of "*NOS Hydrographic Surveys Specifications and Deliverables*" (NOAA 2009).

e. Referencing projects to the current tidal epoch. USACE projects must be referenced to the current NTDE defined by NOAA. NOAA periodically updates the tidal datums throughout CONUS and OCONUS to account for sea level change (rise or fall), local land settlement of tidal gage PBMs, and other factors. These periodic apparent sea level adjustments can be significant—ranging from 0.2 ft to 0.5 ft over the last 19-year update period (1983-2001) on the Atlantic East Coast. Projects not updated since the 1940s would have significantly larger differences—note the upward "apparent" sea level trend at Annapolis, MD shown in Figure 4-7. These adjustments represent systematic changes to the local reference datum (e.g., MSL or MLLW). They also represent systematic biases in navigation project depths or shore protection structure elevations.

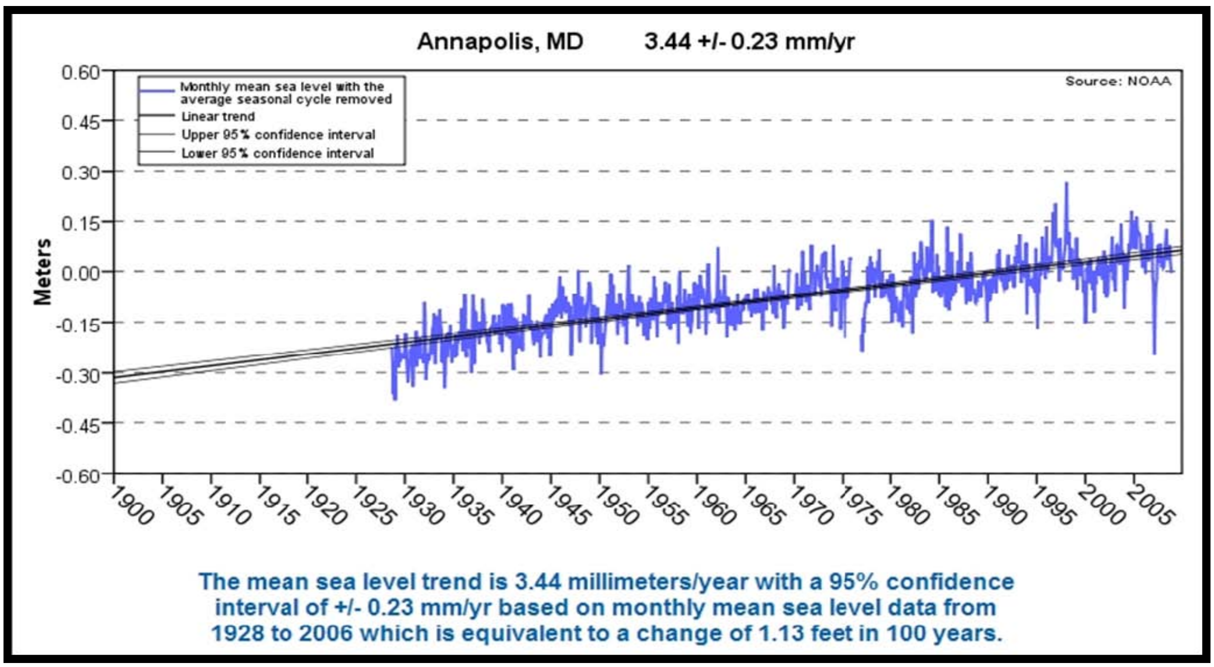


Figure 4-7. Sea level trends at Annapolis, Maryland.

(1) Sea level change impacts on tidal datums. Generally, on most CONUS East and Gulf Coast locations, sea level rise results in maintaining deeper navigation projects than were authorized, and overdredging if the sea level rise is not accounted for. Conversely, on shore protection structures, sea level rise results in less protection than originally designed, assuming this predicted rise was not factored into the design. Numerous USACE, NOAA, and academic technical publications provide guidance on estimating future sea level change for use in the

design of project grades. USACE technical guidance on assessing sea level change recommends that potential relative sea-level change must be considered in every USACE coastal activity as far inland as the extent of estimated tidal influence ... and that fluvial studies (such as flood studies) that include backwater profiling should also include potential relative sea-level change in the starting water surface elevation for such profiles, where appropriate. Sea level change projection uncertainties must be coupled with the uncertainties in the reference datum relating the projected sea level parameters. Refer to Chapter 9 for uncertainties associated with reference datums.

(2) Impact of tidal epoch changes on dredge clearance surveys. Figure 4-8 illustrates the impact of a tidal epoch change on a project that was dredged relative to the superseded 1960-1978 tidal epoch. Adjustment to the latest epoch (1983-2001) significantly reduced the number of strikes above grade that would have required additional dredging had the superseded epoch been held. Of importance is that the required dredging grade of 36.0 ft on the 1960-1978 epoch was 36.22 ft on the 1983-2001 epoch—the project was overdredged by 0.22 ft.

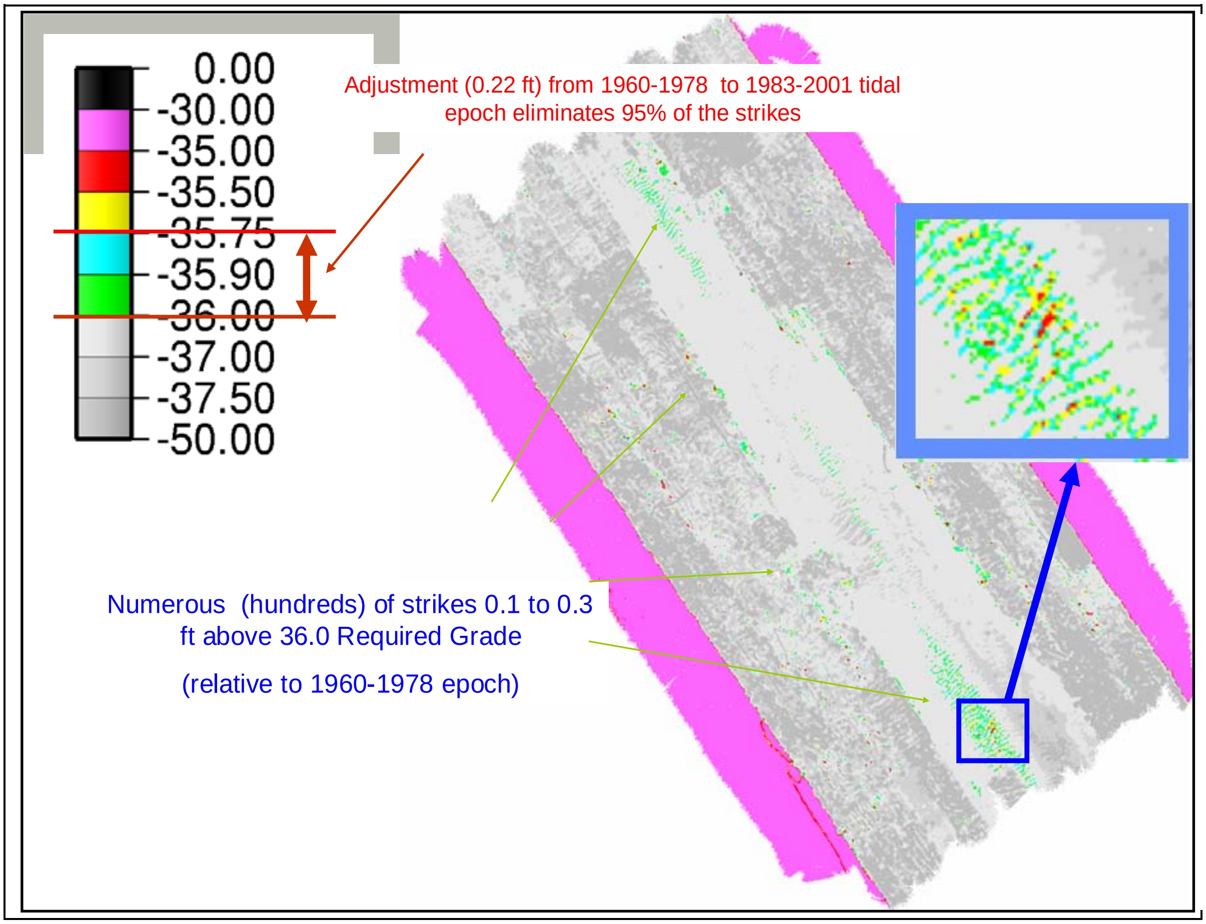


Figure 4-8. Impact of tidal epoch updates dredging strike detection and clearance grades.

(3) Epoch updates are averages from long-term estimates. The adjusted sea level or MLLW datum elevation is based at the midpoint of the epoch. Thus, the current epoch (1983-2001) is averaged about 1993. Tidal epoch adjustments are easily corrected by ensuring projects

are updated when NOAA completes a periodic epoch change. In areas with significant rates of sea level changes or subsidence, NOAA CO-OPS should be consulted to assess the need for shorter 5-year modified tidal datum epochs. NOAA has introduced modified 5-year tidal datum epoch procedures in areas of Louisiana and Texas (due to rapid subsidence) and Alaska (due to rapid land uplift).

4-5. Tidal Datum Uncertainty Estimates. Tidal datums used to reference USACE navigation projects contain two major error sources. These include (1) The regional accuracy of the MLLW datum computed at the project reference gage relative to 19-year NWLON stations, and (2) The local or relative spatial accuracy of the MLLW datum model relative to the project's reference tide gage and the reference orthometric datum (currently NAVD88). If RTN or RTK survey techniques are performed, then the uncertainties in the ellipsoid height measurement and geoid model must be factored into the overall "tidal-geoid" model covering the navigation project. If RTN or RTK surface elevation measurements are not used (i.e., tide readings from a gage are extrapolated to the project site without phase or range correction) then no readily defined accuracy estimate can be made, other than assuming that "worst case" tidal range, tidal phase, and hydrodynamic meteorological conditions occur between the gage and project site.

a. Estimates of NOAA tidal datum regional accuracies. Computed accuracies of tidal datums at a navigation project gage site refer to the uncertainties of the established reference datum at a gage site, such as MLLW or MSL. These estimates have application when a tide gage must be installed at a USACE project to reestablish the reference datum.

(1) Datum errors. The total error of computed tidal datums at a NOAA gage has the following component errors.

(a) Gage measurement error. The measurement error is a combination of the gage/sensor and processing error to refer the measurements to a station datum.

(b) Datum comparison error. The error in computation of equivalent 19-year tidal datums from short-term tide stations. The shorter the time series, the less accurate the datum, i.e., the larger the error. The closer the subordinate station is in geographic distance and in tidal difference to a control station, the more accurate the datum. Estimated maximum errors of an equivalent tidal datums based on one month of data is 0.26 ft for the Atlantic and Pacific coasts and 0.36 ft for the coast in the Gulf of Mexico (at the 95% confidence level).

(2) Tidal datum uncertainties at NOAA gages. Table 4-1 lists the estimated accuracy (i.e., uncertainty) of computed tidal datums for various lengths of gage observation periods. It indicates that, in general, tide stations with at least 3 months record have determined a tidal datum to within ± 0.15 ft. If a NOAA historical gage has some 12 months of record (which is common) then the accuracy of the computed MLLW datum at that point is better than ± 0.1 ft. Refer to NOAA 2001 (*Tidal Datums and Their Applications*) for more details.

Table 4-1. Generalized Estimated Uncertainties of Tidal Datums for East, Gulf and West Coasts when Determined from a Short Series of Record – at 95% Confidence levels.
(from Table 2, NOAA 2001)

Series length (months)	East Coast	Gulf Coast	West Coast
1	± 0.26 ft	± 0.36 ft	± 0.26 ft
3	± 0.20 ft	± 0.30 ft	± 0.22 ft
6	± 0.14 ft	± 0.24 ft	± 0.16 ft
12	± 0.10 ft	± 0.18 ft	± 0.12 ft

(3) Computed tidal datum error. The estimates in Table 4-1 are regional generalized uncertainties and should only be used for planning purposes. Instead of the regionalized approach in the above table, the following relationships may be used to estimate tidal datum uncertainties for each individual subordinate tide station. Specifically, the tidal datum uncertainty is determined from the relationship of the subordinate tide station to the control tide station to which the simultaneous comparison is being made. Assuming most subordinate tide stations for NOS hydrographic surveys are operated for less than one-year durations, the regression equations for mean low water for one-standard deviation ("*s*") estimates are of the form:

$$s_{1\ month} = 0.0068\ ADLWI + 0.0053\ SRGDIST + 0.0302\ MNR + 0.029$$

$$s_{3\ months} = 0.0043\ ADLWI + 0.0036\ SRGDIST + 0.0255\ MNR + 0.029$$

$$s_{6\ months} = 0.0019\ ADLWI + 0.0023\ SRGDIST + 0.207\ MNR + 0.030$$

$$s_{12\ months} = 0.0045\ SRSMN + 0.0128\ MNR + 0.025$$

where:

ADLWI is the absolute difference (in hours) in low water time intervals between subordinate and control stations.

SRGDIST is the square root of the geodetic distance between the control and subordinate stations, measured in nautical miles.

MNR is the mean range ratio that is computed from the absolute value of the difference in mean range of tide between control and subordinate tide stations divided by the mean range of tide at the control station.

SRSMN is the square root of the sum of the mean ranges computed by adding the mean ranges of the control and subordinate stations and then taking the square root of this sum.

For stations with series longer than one-year in length, the datum errors can be time-interpolated between the estimate at that station for a one-year series and the zero value at 19 years. Errors in tidal datums for accepted datums from 19-year control tide stations are zero by definition. Using these formulas, estimates of the datum error can be uniquely computed in the planning process for each subordinate tide station being used for the hydrographic survey using historical and accepted tidal datums on file.

(4) Recommended observation periods for USACE projects. When a gage is installed at a USACE project, the above NOAA accuracy estimates may be used to assess the required observation period. Based on Table 4-1, 30 days of simultaneous gage observations should usually be adequate to develop a reliable reference datum at the ± 0.25 ft level on most East Coast and West Coast projects. Deep-draft projects with critical keel clearance issues may warrant 3 to 12 months of observations. All gage installations, observation periods, and datum computations should be closely coordinated with, and approved by, NOAA CO-OPS.

b. Local or relative tidal datum accuracy. It is important to emphasize that the above uncertainties in the computed datum at a tide gage do not necessarily factor into the relative, or local, accuracy of an established tidal datum on a project. The computed/established reference datum at the gage is considered "fixed" for referencing dredging grades. Thus, for the purposes of defining dredging grades on navigation projects, errors in the "global" or regional determination of the reference datum are not usually an issue, other than providing regional uncertainty estimates of tidal datums for storm surge monitoring or like purposes. If a NOAA tidal PBM is used to reference grades at a project site, then both USACE channel clearance surveys and NOAA charts will be referenced to the same "local" MLLW datum.

(1) For small navigation projects with only one reference tide gage, the reference datum at the gage is the designated reference for the entire project, and RTK calibrations are performed to tidal PBMs at this gage. For larger projects with two or more tide gages, calibration discrepancies between the gages may result due to the absolute (regional) datum uncertainties between the tide gages. These calibration differences may or may not be significant. If significant, then a zoned calibration reference should be designated for each project reach—e.g., specify the tide gage to be used for specific channel reaches. The following Table 4-2 is an example of a zoned gage reference on a large 67-mile length project with VDatum coverage. In this example, VDatum model calibrations were made using a regional RTN network, resulting in 0.1 to 0.2 ft variations at the calibration gages. Construction survey plans in a given channel reach are fixed ("zoned") to the gages in the table—i.e., all users must calibrate RTN systems to MLLW at these specific gages for a given channel reach.

Table 4-2. Zoned Reference Gages for Tampa Harbor Channel Reaches.

Tampa Harbor Channel Reach	NOAA Gage	Station ID
Egmont Cuts Mullet Key Cut	Egmont Key Mullet Key	872 6347 or 872 6364
Cut A, Cut B, and Cut C	Port Manatee	872 6384
Cut D, Cut E, and Cut F	St. Petersburg	872 6520
Gadsden Point Cut to PI Cut A & C (HB) and Cut G (PT)	Gadsden Point	872 6573
Cut C (HB) and Alafia River Channel	Long Shoal-MacDill AFB	872 6604
Davis Island, Seddon Island Port Sutton, & McKay Bay Channels	Ballast Point Hooker Point Davis Island	872 6639 or 872 6668 or 872 6657
Cut J (PT) & Cut K (PT)	Port Tampa	872 6607

(2) Regardless of the absolute accuracy of a tidal datum for a project, the relative accuracy (i.e., "repeatability") is most critical for survey and dredging operations. In general, a local tidal datum relative accuracy of ± 0.1 ft should be achievable at most navigation projects where an established tide gage exists. RTN calibrations are performed at this gage and a VDatum type model is used to correct for local MLLW variations.

c. Dredging measurement & payment survey repeatability. As stated above, for USACE tidal datum modeling purposes, and subsequent maintenance dredging and construction of projects, the accuracy of a NOAA gage datum, (or acceptable datums from another agency's long-term gages) will be assumed as absolute—i.e., they will be assumed to have “zero error” ("zero uncertainty") irrespective of the actual computed datum uncertainties at a particular gage. This assumption is valid in that the final developed MLLW tidal model for the project (e.g., NAVD88-MLLW differences in VDatum) will also be considered fixed for project construction purposes. This fixed local VDatum model, when used with RTK, provides repeatability between users (surveyors, dredges, etc.), limited mainly by the precision of the RTK solution and the site calibration. This repeatability is critical for equitable dredge payment surveys. If RTK is not used, and zoning estimates relative to a water level gage are used, then repeatability will be dependent on tidal range and phase variations.

d. Tidal datum accuracies for navigation projects. Table 4-3 represents the desired accuracy of a navigation project model, considering the total propagated uncertainties (TPU) in both the MLLW datum and the geoid.

Table 4-3. Recommended Accuracies for Tidal Reference Datums on Navigation Projects with VDatum Coverage.

	Accuracy (95%)	Relative to Datum
Absolute accuracy of tidal datum relationship at gage	± 0.25 ft	MLLW Regional NWLON
Relative accuracy of local tidal model	± 0.2 ft	Local MLLW at PPCP Gage

Tidal-geoid model numerical resolution:	nearest 0.01 ft
Model 1D or 2D density in navigation channel:	100 to 500 ft (varies with tidal range)
Geoid model:	use latest available at time of study (currently Geoid 09)
Tidal-geoid model format:	1D or 2D (1D for linear navigation channels)

NOTE: The above standards are believed representative for most CONUS navigation projects. Exceptions may exist in extreme tide ranges or in parts of Alaska. See VDatum uncertainty models on NOAA VDatum web site.

In general, a full tidal-geoid model absolute accuracy of < ± 0.25 ft should be achievable at most deep-draft navigation projects where NOAA calibration gage data exists. Local (relative) model accuracy should be better than ± 0.1 ft on such a project—i.e., that accuracy relative to one or more local NOAA gages where VDatum coverage exists. Regardless of the resultant absolute accuracy of a tidal model for a region, the relative ("repeatable") accuracy is most critical.

4-6. Tidal Modeling Methods to Define Local MLLW Datums on Coastal Projects. Defining the MLLW datum tidal model on a navigation project requires the following basic actions: (1) ensure tidal datum reference planes (MLLW) are defined relative to published NOAA gages and tidal benchmarks, (2) ensure the latest tidal epoch adjusted by NOAA is used, (3) model the MLLW reference plane and geoid throughout the geographic extent of the project, (4) verify/calibrate the MLLW model at gage sites, (5) publish/disseminate the tidal-geoid model for users—e.g., a Kinematic Tidal Datum (KTD) file, (6) determine the NAVD88-MLLW datum relationship at tidal benchmarks, and (7) submit any field GPS or gage data to NOAA for their use in expanding the nationwide VDatum. Actions (1) and (2) are easily achieved as long as an existing or historical gage exists at the navigation project. This will likely be the case for the majority of the Corps' deep-draft navigation projects. If not, then a short-term gaging program will have to be developed in order to establish a tidal datum at a project. Any such effort must be coordinated with NOAA in order to ensure the project becomes included in the NOAA CO-OPS gage inventory. Project modeling—actions (3) through (7)—will require close coordination with District H&H elements, ERDC/CHL, and/or NOAA. In small tide ranges, or in survey areas that are small geographically and hydrodynamically simple, linear interpolation of the MLLW model will often be sufficiently accurate and economically developed. By 2012, VDatum models may already have been developed for most projects.

a. Modeling techniques. A number of techniques can be employed to model the variations in tidal datums on a coastal navigation project. These models reflect the changes in mean or diurnal tide ranges that occur on the project. They are configured to relate the difference between NAVD88 and MLLW spatially over the project since RTK observations of ellipsoid heights are reduced to a NAVD88 elevation of the local water surface. These models may be simple or complex depending on the project use and maintenance activity—ranging from assumed constant NAVD88-MLLW differences throughout the project to a full hydrodynamic tidal model of varying NAVD88-MLLW differences based on multiple gages in the project area. These various modeling options include:

(1) Constant NAVD88-MLLW model. Assumes no significant tidal range or phase differences occur between the reference gage and project site—i.e., a "tide correction" at a reference gage is extrapolated to the project site. This "model" is only applicable when the gage is close to a confined project site with a small tide range—i.e., minimal potential phase and wind effects between the gage and the site. The water level elevation ("tide correction") and the NAVD88-MLLW difference at the gage are assumed the same throughout the project. If RTK positioning is used under such conditions, variations in geoid heights must still be applied.

(2) Spatial interpolation model between tide gages. A simple linear or TIN spatial interpolation of the variations NAVD88-MLLW differences between tide gages. Examples of spatial interpolation modeling methods are shown in Appendix E.

(3) NOAA tidal zoning estimates. Tidal zoning models are estimates of tidal ranges and tidal phases at an offshore project site. Tidal zoning is used extensively by NOAA but only on isolated USACE projects (e.g., Chesapeake Bay). A further discussion on tidal zoning is at Appendix E.

(4) NOAA Tidal Constituent And Residual Interpolation (TCARI) model—a NOAA/CO-OPS spatially interpolated model. Incorporated in, and being replaced by, NOAA VDatum models.

(5) Hydrodynamically generated tidal models. Hydrodynamically generated tidal models that spatially develop the tidal regime and MLLW datum variations over the project.

b. Tidal datum models. Figure 4-9 illustrates a tidal datum model for a coastal inlet navigation project. As shown on the figure, the existing MLLW datum model is based on a straight-line interpolation between the gages. A hydrodynamic tidal model, such as VDatum, would fit (calibrate) the induced astronomical tide to the MLLW datums at each gage, as shown by the curved MLLW profile in the figure.

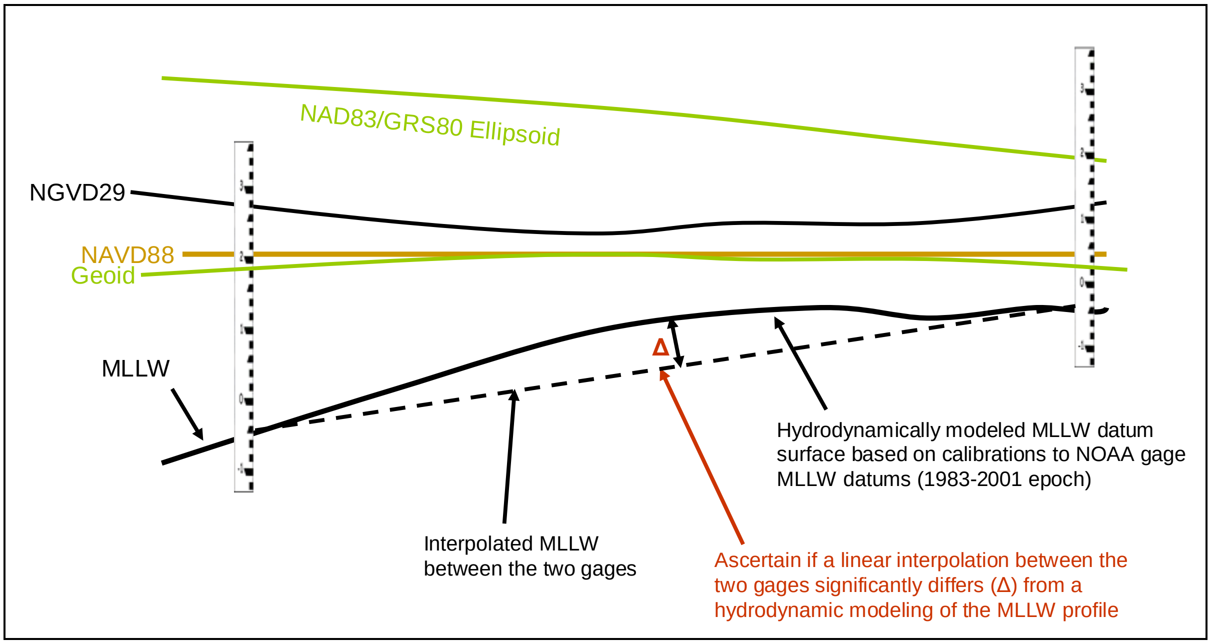


Figure 4-9. Modeled versus interpolated MLLW datums.

(1) Interpolated tidal models. Of significance is whether this project can be just as effectively modeled using a simple straight-line interpolation between the gages as opposed to running a full hydrodynamic model. In lower tide ranges, or with dense gage data, this would be the case. In general, if the estimated variation between a model and straight-line interpolation does not exceed 0.1 ft, then the straight-line interpolation would be acceptable. This variation is indicated by "Δ" in Figure 4-9. The use of validated VDatum models is recommended in lieu of linear interpolation.

(2) Geodetic reference datums. Figure 4-9 also depicts the relationship between other geodetic reference datums. The local geoid model would provide the undulation shown relative to NAVD88, and indirectly relative to MLLW. NOAA's VDatum model includes the transformations between all these datums.

c. Example of a spatially interpolated project. Appendix F contains an example of a Jacksonville District project (Canaveral Harbor) where spatial interpolations of the MLLW reference datum were estimated from NOAA gage data; both in the offshore Entrance Channel and in a semi-controlled pool. These estimates were made prior to receipt of VDatum model data which will supersede these estimates.

4-7. National Vertical Datum Transformation Software (VDatum). VDatum is a vertical datum transformation software tool developed by NOAA for coastal areas that allows users to transform geospatial data among a variety of geoidal, ellipsoidal, and tidal vertical datums. VDatum is important to coastal applications that rely on vertical accuracy in bathymetric, topographic, and coastline data sets, many of which may be produced on different reference datums but need to be merged for hydrodynamic surge models. VDatum has application to most, if not all, USACE coastal navigation projects. It also represents a defined datum reference for USACE projects.

a. Transformation datums. Currently the VDatum software is designed to convert between over 30 geodetic datums, including full continuous models of NAVD88 and MLLW, which are especially applicable to most USACE navigation projects. Various geoid models are also included. Figure 4-10 depicts the variety of datum transforms currently available in VDatum. Only the NAD83/NSRS2007 ellipsoidal datum is utilized on CONUS projects.

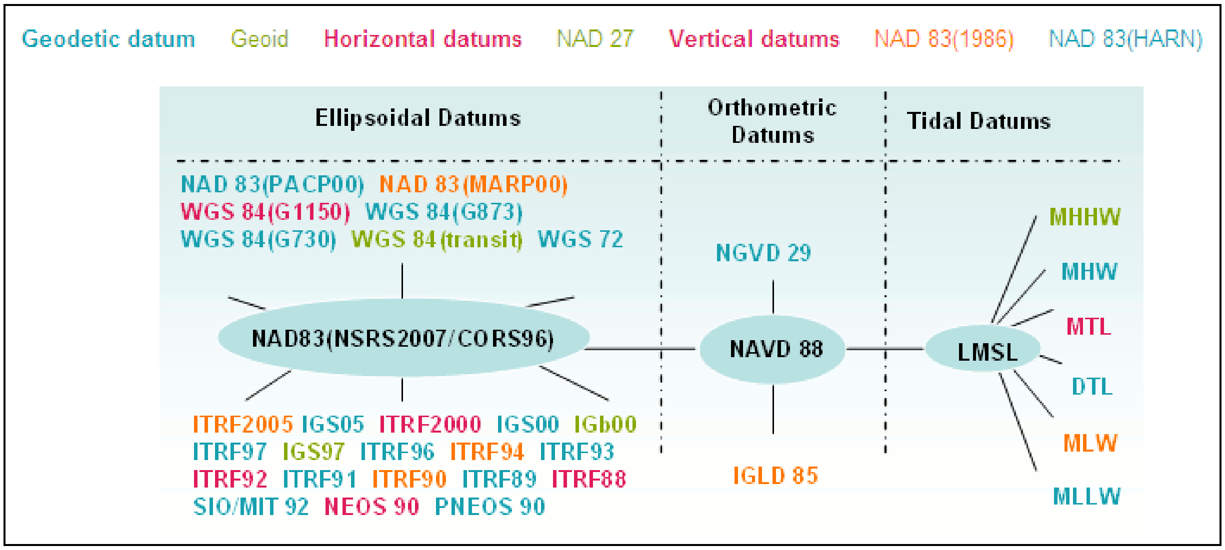


Figure 4-10. Primary VDatum transforms between ellipsoidal, orthometric, and tidal datums.

b. VDatum coverage. Figure 4-11 shows VDatum coverage as of 2010. It is anticipated that complete CONUS coverage will be available in or after 2012. Coverage of some OCONUS areas is in progress. In many cases, VDatum coverage extends up to the head of navigation on deep-draft harbors and ports.

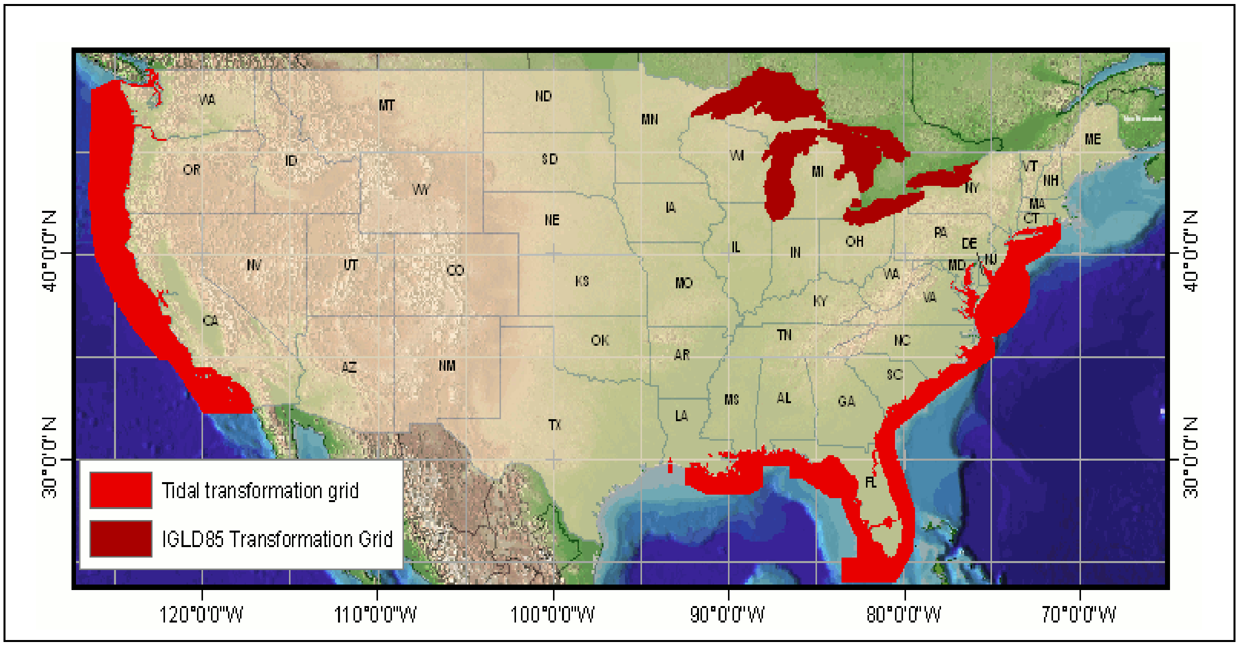


Figure 4-11. VDatum coverage in CONUS as of April 2010 (NOAA).

c. Use of VDatum for surveying and dredging applications. Most USACE applications will involve incorporating VDatum transforms with observed real-time GNSS satellite observations to obtain the elevation of the water surface relative to the local construction datum (e.g., MLLW, LMSL, IGLD85, etc). This water surface elevation, often resolved as a "tide correction" in RTN/RTK hydrographic positioning and orientation systems, is then applied to a measured depth to obtain a "corrected depth." The applicable VDatum ellipsoid-NAVD88-construction datum transforms in a project area are normally output to a 2D TIN model of the project area. This TIN model can then be input into surveying and dredging positioning/orientation software for real-time datum conversions at any point on the project. An example of this is the HYPACK "KTD" file. Future developments in survey and dredge positioning/orientation software will include a seamless input of VDatum transform models.

d. Site calibration. VDatum models of navigation projects need to be "site calibrated" (i.e., verified) prior to use in dredging measurement & payment or clearance surveys. This entails comparing observed water surface elevations at a reference NOAA tide gage/staff to those reduced through VDatum on a RTN/RTK survey positioning/orientation system on the vessel. Given the resolution of tide staff readings and RTK accuracy, tolerances approaching ± 0.2 ft would be expected. Uncertainty estimates for the various VDatum transformations are provided on NOAA's VDatum web site. An example of a VDatum site calibration is shown in Appendix D.

e. Further information on VDatum. Additional technical details on VDatum applications are available from the NOAA Coast Survey Development Laboratory "VDatum Team" web site and from "*Review of Progress on VDatum, a Vertical Datum Transformation Tool*" (Myers 2005).

4-8. Tidal Phase and Water Surface Elevation Variations over a Navigation Project. The major correction in the depth measurement survey of a navigation project is for tidal phase (latency) variations between the reference tide gage and the location of the dredge or survey vessel at the project site. Local hydrodynamic and meteorological effects (e.g., wind set up) changes the water surface elevation profile in the project. These variations due to tidal phase, along with other hydrodynamic or meteorological effects, increase with the distance from the tide gage. These systematic differences can exceed 1 to 2 ft in moderate range projects, and higher on projects with large tide ranges (over 10 ft) or experiencing adverse weather conditions. They are most pronounced during periods of full ebb and flood tide. Many dredging measurement & payment survey disputes and claims arise over lack of adequate compensation/correction for tidal phase and meteorological set up throughout a project site—see EM 1110-2-1003 (*Hydrographic Surveying*) for details..

a. Tidal phase latency variations. EM 1110-2-1003 and EM 1110-2-1100, (*Coastal Engineering Manual*), Part II-6, "*Hydrodynamics of Tidal Inlets*" have numerous examples of the tidal phase and MLLW range variations that typically occur between the ocean and bay at a typical coastal inlet. These tide curves do not include any hydrodynamic or meteorological effects which could, at times, exceed the basic phase variations. Modeling and correcting these tidal phase variations throughout the project is critical.

b. Water surface elevation measurements using RTK techniques. Tidal phase errors and weather/sea surface set up are effectively eliminated by using RTK surface elevation measurement techniques, coupled with inertial measurement and orientation systems. Local water level variations can be measured in real-time using these RTK techniques, either from a local RTK base station set at a PPCP or from a regional RTN system. RTK methods effectively measure the local water surface elevation relative to the ellipsoid; thus, providing direct corrections relative to a MLLW datum at a modeled offshore construction site.

(1) Dredging measurement & payment surveys performed using RTK methods will usually employ a combined tidal-geoid model from which to correct observed ellipsoid heights measured relative to the water surface; to obtain a surface elevation relative to the tidal MLLW model at the project site. Thus, the measured ellipsoidal height of the water surface at any point is corrected for (1) geoid model undulations, and (2) tidal range variations based on hydrodynamic models of the tide in the region. The RTK measurement process is illustrated in Figure 4-12. The actual offshore water surface level above local MLLW (i.e., a "tide correction") is thereby measured at every observation (typically 1 to 10 Hz) made by a survey vessel, dredge, or commercial vessel employing RTK methods; and an average surface level above local MLLW computed using filters and/or an inertial measurement unit (IMU) over a 30 to 120+ second filter period. As long as every user (vessel) employs the same tidal-geoid model for the region, then full repeatability of surface elevation measurements will be achieved. The relative accuracy of the RTK measured surface elevation and tide level will fall around ± 0.05 ft level. The tidal-geoid model developed for the project is considered as absolute.

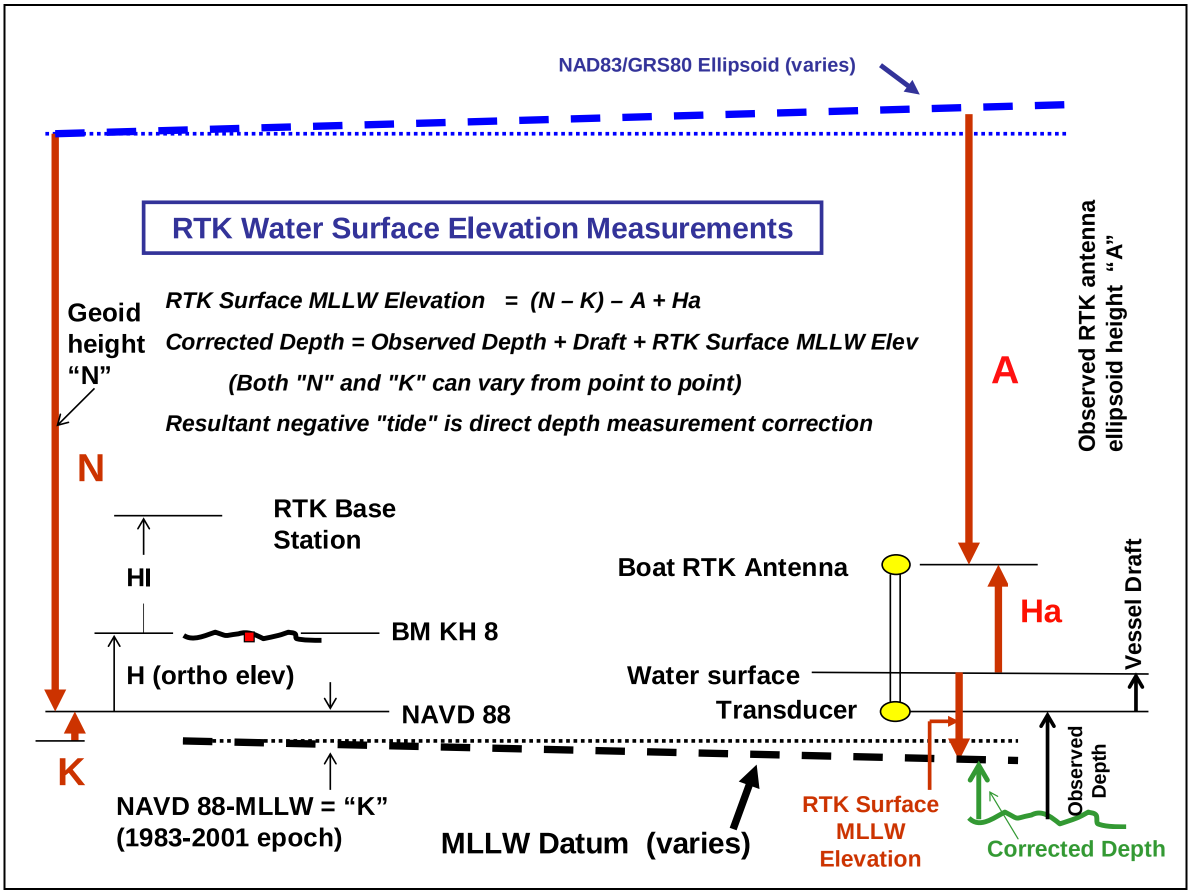


Figure 4-12. RTK Ellipsoid-Tidal-Geoid parameters for water surface elevation measurements. (Note: coordinate system is positive up)

Using the parameters shown in Figure 4-12, the following is an example of RTK observations and vessel depth measurements that derive a "corrected depth" on the MLLW reference datum. Note that observed depths and drafts are shown as positive values in Figure 4-12. Observed depths and corrected depths are shown as positive downward values.

RTK Parameters	Vessel Observations
N = (-) 71.29 ft	Observed Depth: + 40.0 ft
K = + 1.76 ft	Vessel Draft: + 3.0 ft
A = (-) 45.00 ft	
H_a = + 22.05 ft	

RTK Surface MLLW Elevation = (- 71.9 - 1.76) – (- 45.0) + 22.05 = (-) 6.0 ft

Corrected Depth = = + 40.0 + 3.0 + (- 6.0) = 37.0 ft

(2) Geoid model accuracy is a function of the location and density of NSRS vertical control and gravity data in the area. The predicted geoid undulation from the latest geoid model will be

used for offshore entrance channels—areas that obviously have no vertical control but have geoid height estimates using other techniques (airborne gravity). NGS should be contacted to confirm the accuracy of the predicted geoid model does not exceed reasonable tolerances. Likewise, the predicted tidal range in offshore entrance channels 3 to 10 miles seaward may have to be based on established regional models of the ocean tides. In such cases, the estimated accuracy of these regional models may be verified by contacting ERDC/CHL or NOAA. Alternatively, these offshore tidal ranges, and indirectly the geoid model, can be easily confirmed by observing long-term RTK data recorded during the course of a survey in the area.

(3) It is emphasized that the tidal-geoid model developed for each project must be published and disseminated to all users. This may be a simple ASCII file in the form of a gridded difference between NAVD88 and MLLW (NAVD88-MLLW), such as a “KTD” file used by commercial navigation dredging software (HYPACK®). Since most USACE navigation projects are linear, only a 1D model may be required—e.g., a tidal-geoid correction every 100-ft station down the channel centerline. This is adequate to cover the areal extent of a 100 ft to 1,000 ft wide channel. This file may periodically be updated if the MLLW tidal model for the region is significantly modified by NOAA. Thus, the file must clearly identify (metadata) the source of the data. Care must be taken in that in some navigation/dredging processors, the geoid correction may be performed separately by the GPS receiver from the MLLW tidal model correction—i.e., two distinct corrections. Thus the KTD file may contain only the tidal datum correction (NAVD88-MLLW or "K" in Figure 4-11) or may combine both the tidal datum correction "K" and the geoid correction "N." Users must also be advised that RTK, like any measurement system, must be periodically checked (and site calibrated/localized if necessary) against a physical recording tide gage or staff gage.

c. RTK versus gage surface elevation measurement. Figure 4-13 illustrates the application of using GPS/RTK elevation measurement for removing tidal phase and wind-induced errors on a Jacksonville District dredging project at Key West, FL. In this example, a constant 0.3 ft phase bias (and perhaps some wind setup bias) is generated during ebb tide at a point only 3 miles distant from the gage. This phase bias is significant given the tide range at this project is only about 2 ft. Had the NOAA tide gage been used to correct depth measurements during this survey, a 0.3 ft bias would have been translated to the quantity measurements in this Acceptance Section. As shown in Figure 4-13, the RTK-determined elevation of the sea surface at the dredging site was estimated to be accurate to ± 0.05 ft, effectively minimizing the tidal phase and potential meteorologically induced errors at this offshore project site. RTK operations are only successful if the MLLW to ellipsoidal difference are correctly modeled and understood prior to the survey as these two reference planes have slopes relative to each other.

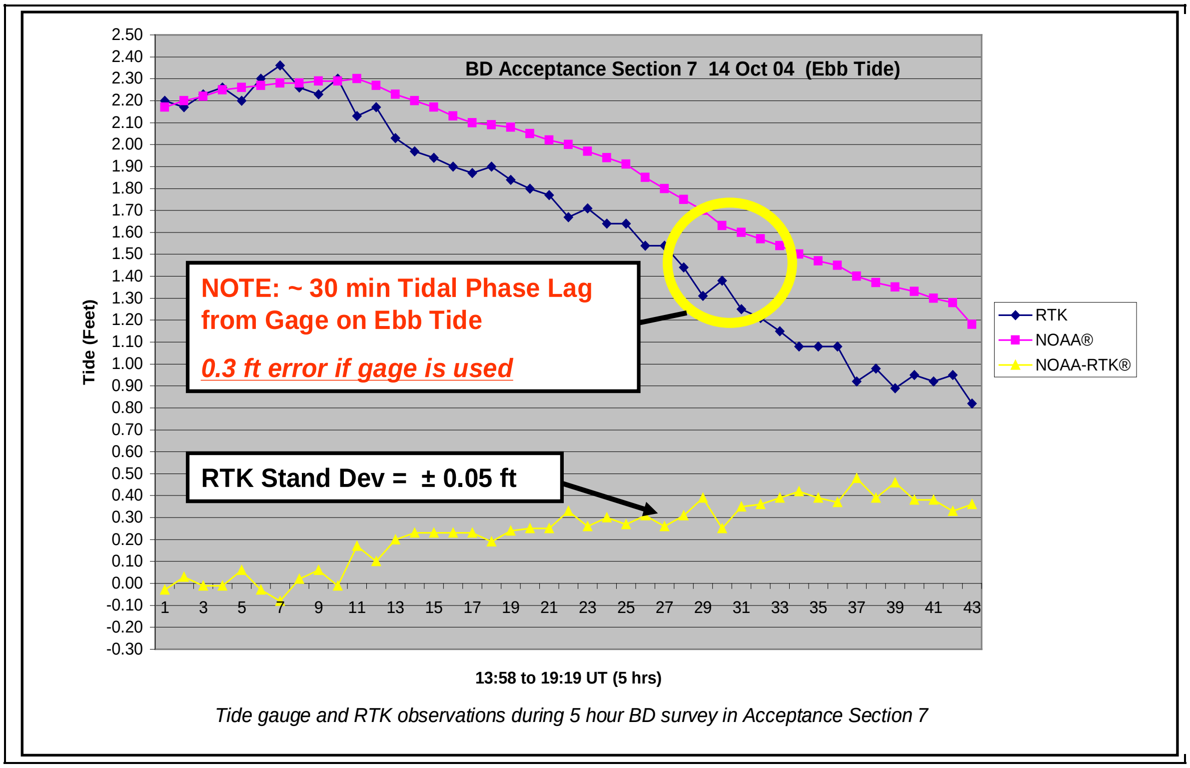


Figure 4-13. Offshore RTK tide comparisons with an onshore NOAA tide gage.
(Key West Harbor, FL Acceptance Section 7—3 miles south of Key West in open water)

d. RTK survey procedure references. See EM 1110-2-1003 for additional details on the impacts of tidal phase errors on dredging measurement & payment surveys. Refer to EM 1110-1-1003 (*NAVSTAR GPS Surveying*), EM 1110-1-1005, and hydrographic survey system user manuals for detailed RTK survey procedures.

4-9. Channel Control Framework Drawing Notes for Navigation Projects. Detailed datum metadata are required on drawing notes for plans and specifications surveys, channel condition surveys, and measurement & payment surveys. It is especially critical that bid documents for dredging projects contain the essential horizontal and vertical datum parameters, gages, and transformation models that will be used during construction. See Appendix D for an example of drawing notes used on a Jacksonville District project with VDatum and partial RTN coverage.

4-10. Summary of Evaluation Factors for Determining a Reference Tidal Datum on a Navigation Project. Table 4-4 outlines a general decision flow process for determining the reference tidal datum for a navigation project, based on the issues discussed above.

Table 4-4. Tidal Reference Evaluation Factors.

I. <u>Existing Tide Gage Data</u>

Does project have an existing tide gage that is adequate to define the tidal datum?
If NO, go to IV.

Is gage data based on relatively recent observations? If NO, go to IV.

Is gage observation series length adequate based on project use? If NO, go to IV.

Is gage datum based on or can be updated to current tidal epoch? If NO, go to IV.

Have physical modifications to project possibly impacted historical gage datum data?
If YES, go to IV.

Does gage location and density adequately model the entire project? If NO, go to III.

II. <u>Reference Tidal Bench Marks</u>

Were two or more tidal bench marks recovered at existing gage station?
Only one tidal bench mark recovered: Evaluate stability & reliability
No tidal bench marks recovered? If YES, go to IV.

Are tidal bench marks stable based on field level checks? If NO, perform checks.

Does existing gage site contain 3 or more reference PBMs? If NO, add PBMs as required.

Do any tidal reference bench marks need to be published in the NSRS?

III. <u>Tidal Modeling</u>

Does NOAA VDatum model adequately cover project to head of authorized navigation?
If NO, contact NOAA CO-OPS for possible extension of VDatum.

Has VDatum model been site calibrated/verified in the field?
If NO, perform field site calibrations.

If no VDatum coverage exists: Evaluate use of spatially interpolated model or NOAA TCARI model.

Has NAVD88-MLLW model file been generated for project?

Table 4-4 (Concluded). Tidal Reference Evaluation Factors.

IV. New Gaging Program Required to Develop Datum

Evaluate (with NOAA CO-OPS) options for using VDatum NAVD88-MLLW relationships
If not an option install a new gage to reestablish datum

Determine length of series requirements in coordination with NOAA CO-OPS	30 days most projects 90 to 360 days on critical clearance deep-draft projects

Were new reference tidal PBMs established and furnished to NOAA CO-OPS?

Do new reference PBMs need to be placed in the NSRS?

V. Hydraulic or Legacy Tidal Datums

Are legacy datums adequately referenced to the current MLLW datum?
If not, is estimated uncertainty documented?

In junctions between river and tidal datums, is the hydraulic low water plane adequately referenced to the NSRS and tidal datums?

Are local flood stages referenced to MLLW and NSRS?

CHAPTER 5

Procedures for Referencing Datums on Coastal Hurricane and Shore Protection Projects

5-1. Purpose. This chapter discusses the procedures to ensure hurricane protection projects and related shoreline protection structures (i.e., HSPP) are adequately connected and modeled relative to the National Water Level Observation Network (NWLON) tidal datum and the National Spatial Reference System (NSRS) orthometric datum established by the Department of Commerce. Coastal hurricane protection and shore protection structures include levees, floodwalls, breakwaters, jetties, groins, and dikes. Beach renourishment/restoration projects are also included in this category. Most of the guidance in Chapter 4 on coastal navigation projects is directly applicable to coastal hurricane and shore protection projects. Thus, references to Chapter 4 will be made where applicable.

5-2. Reference Datums and Tide Gage Connections. Most USACE shore protection projects have been designed relative to a sea level tidal datum—typically MLW or MSL referenced to a superseded legacy NGVD29 datum. Some projects may be defined relative to MHW. The designed protection grade will include allowances for storm surge, wave runup, tidal ranges, and other modeled factors used to develop a protection elevation. Uncertainties in these reference datums must also be considered in the design of protection elevations. Construction of shore protection projects is usually performed relative to a local orthometric datum, such as NAVD88. When RTK methods are used for construction stake out and machine control grading, NAD83/GRS80 ellipsoid references are required. As stated in preceding chapters, the main objective is to establish a firm relationship between the local orthometric and tidal datums at the project site, and that these datums (and any legacy reference datums) are referenced to the current NSRS and NWLON frameworks. For the vast majority of shore protection projects, modeling protection grades to the current tidal datum and NSRS/NAVD88 reference is relatively straightforward and normally requires minimal field survey effort.

a. Reference datums. Figure 5-1 illustrates the reference datums used for various shore protection projects. Design elevations of the beach renourishment berms or crests of the shore protection structures are computed relative to local tidal datums. This LMSL datum shown in Figure 5-1 must be based on nearby tide gages that relate LMSL to the NAVD88 datum. If no gages are near the project site, then the LMSL elevation at the project site relative to NAVD88 must be estimated using established hydrodynamically generated tidal models (e.g., VDatum). On existing projects originally referenced to a legacy reference datum (NGVD29 or MLW), the relationship between these datums and NAVD88 should be established. This assumes the original construction PBMs can be recovered. Stone placement on jetties or breakwaters, or grading of berms or hurricane protection levees, normally would be performed using RTK elevation measurements.

b. Primary Project Control Point (PPCP). All shore protection projects should have at least one primary NSRS bench mark from which all construction is referenced. This PPCP shall be published in the NSRS. On large beach renourishment or hurricane protection projects, more than one primary NSRS reference bench mark may be required. All projects shall have at least three PPCPs or LPCPs for use on construction plans. During Preconstruction Engineering and Design (PED), and prior to construction, the elevations of PPCPs and LPCPs shall have been checked internally—and regionally against other nearby NSRS points. Most coastal areas in CONUS have relatively dense NSRS network coverage (e.g., NGS level lines), from which the project can be directly referenced to this framework with minimal effort.

Figure 5-1. Reference datums on shore protection projects.

c. Tidal datum references. HSPP sites should be evaluated to verify (1) that the design/constructed sea level reference datum is current (i.e., latest tidal epoch and model) and (2) that this sea level datum can be related to the local NSRS orthometric datum (NAVD88) and the NAD83/GRS80 ellipsoid.

(1) If an active or historic NOAA tidal gage is situated in or near the HSPP area, data from that gage can be directly used to reference the project to the current tidal datum epoch. If the gage's tidal bench marks have not been connected to the NSRS orthometric datum, then GPS field surveys will be required to perform this connection. Only a small percentage of NOAA gage sites have tidal PBMs that have been connected to either NGVD29 or NAVD88. HSPP on remote coastlines may need static baseline GPS or CORS/OPUS observations to connect them to the NSRS. In general, CORS/OPUS methods described in Chapter 3 will provide sufficient accuracy to develop NAVD88 and tidal relationships on tidal bench marks at gages without published NSRS data.

(2) If there are no nearby tide gages, then the NAVD88-MSL relationship can be estimated using spatial interpolations between the nearest gages—see Appendix E. The local NAVD88-MSL relationship may also be estimated using NOAA VDatum models of the project region. If estimates are used, project documents must clearly describe the estimating procedure, uncertainties of the estimate, and potential impacts on the design (risk assessment) of the protection elevation of a control structure.

5-3. HSPP Elevation Accuracy Requirements. Table 5-1 lists general elevation accuracy requirements for HSPPs. This table should only be used as general guidance. Site dependent factors may require variations from this guidance.

Table 5-1. Survey Accuracies Common to Shore Protection Projects.

Bench Mark/Activity	Accuracy (95%)	Relative to
Primary Project Control Point (PPCP)	± 0.25 ft	Regional NSRS network
Local Project Control Points (LPCP)	± 0.02 to 0.05 ft	PPCP
Construction TBMs (hubs, nails, etc)	± 0.02 to 0.05 ft	LPCP
Construction grade stakes	± 0.1 ft	LPCP or TBMs
Levee or beach grading	± 0.5 ft	LPCP or TBMs
Offshore borrow area excavation	± 0.5 to 1 ft	PPCP
Offshore stone placement	± 0.5 to 1 ft	PPCP, LPCP, or TBMs

5-4. Shore Protection and Beach Renourishment/Restoration Projects. Shore protection projects are usually designed relative to tidal or orthometric datums, depending on local preferences. On many older projects, the relationship between orthometric and tidal datums is not firmly established. The PPCPs used to control the project should be related to the latest tidal datum and have a firm reference to the NSRS (NAVD88).

a. Control scheme for beach renourishment projects. Figure 5-2 depicts a survey control scheme that is set up for typical beach fill projects. Orthometric elevations of local LPCPs, PBMs, TBMs, grade stakes, and surveys are referenced to a PPCP that is published in the NSRS.

(1) Primary Project Control Point. Depending on the geographical scope of the project, one or more PPCPs may be needed. These PPCPs are used as RTK base stations to control grading operations (machine control), dredge borrow area surveys and excavation, setting grade stakes during beach fill operations, and controlling pre- and post-fill measurement & payment surveys. The NSRS connection will normally be performed following the same accuracy standards and field survey specifications outlined in Chapter 3—e.g., ± 0.25 ft. Checks between the PPCP and other NSRS bench marks are required prior to performing surveys.

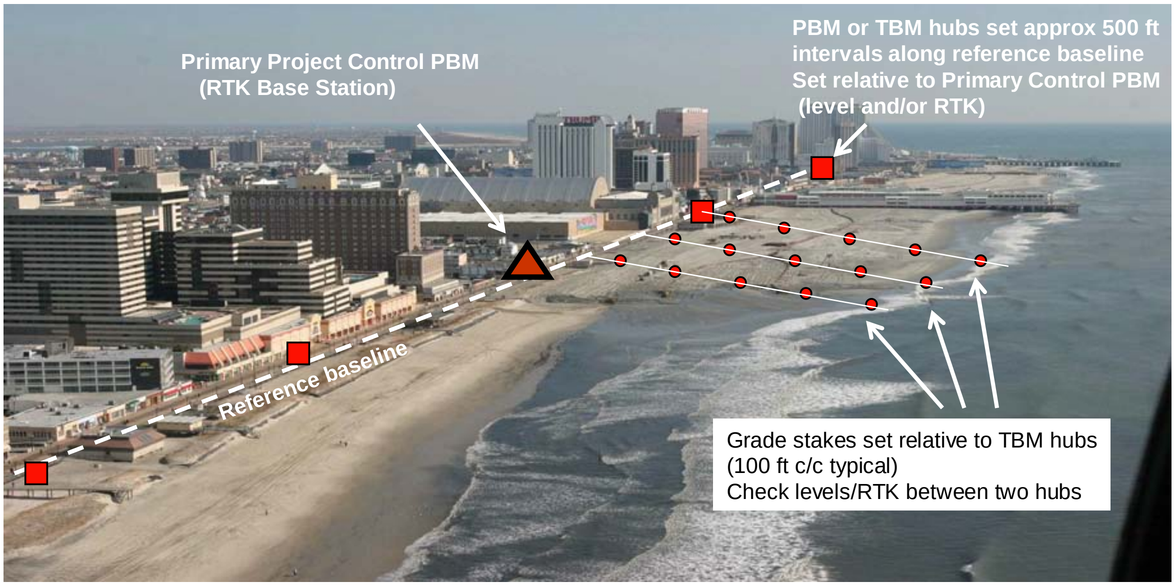


Figure 5-2. Beach renourishment project at Atlantic City, NJ. (Philadelphia District)

(2) Reference baselines. Most shoreline projects are continuously monitored by performing topographic and hydrographic surveys relative to established ranges. These ranges are referenced to a fixed baseline set back from the beach, and beach profile offsets are referenced to this baseline.

(3) Local reference PBMs. The baseline is usually referenced to local PBMs (LPCPs) set at various intervals. These LPCPs will have SPCS coordinates along with local station-offset coordinates. Vertical control on these baseline LPCPs may be referenced to legacy or local datums in order to monitor relative accretion or erosion at a measured profile. The LPCPs should be resurveyed at the beginning of a project. If the project specifications require holding legacy elevations on LPCPs, then any significant differences between these legacy elevations and the resurveyed elevations shall be clearly noted.

(4) Temporary reference or calibration hubs. In cases where no fixed LPCPs are available along the reference baseline, TBM hubs are established at various intervals. These hubs are used for controlling beach profile surveys—checking RTK calibration or for referencing level runs on individual beach profiles. Elevations on these hubs are normally surveyed by differential leveling relative to the PPCPs. Levels are run through local LPCPs and TBM hubs over the project reach. RTK elevation checks should also be made to verify site calibration and check for leveling blunders.

(5) Grade stakes. During beach fill operations, grade stakes are set from the reference LPCPs or TBMs. Either RTK, total station, or differential leveling methods are used. Elevation checks should be made to at least two fixed LPCPs or TBMs. The number of grade stakes on each profile will vary with the number of grade breaks on the project—see Figure 5-3. Grade stake elevation tolerances are normally specified as ± 0.1 ft relative to the reference LPCP or TBM. Grading tolerances are typically ± 0.5 ft.

b. Beach profile surveys. Beach fill measurement & payment surveys are normally performed at 100 ft c/c intervals. Spacing of periodic monitoring study profiles will vary between 200 ft and 1,000 ft. A design profile template is shown in Figure 5-3. Various topographic and hydrographic survey methods are used—see EM 1110-2-1003 (*Hydrographic Surveying*). These surveys are referenced to offsets from the reference baseline, or in some cases, an erosion control line, or construction setback control line. Profiles are run on established range azimuths. In Figure 5-3, the offset profile is referenced to a fixed local PBM for this section.

c. Reference datums. It is likely that older shoreline protection projects were designed and constructed relative to NGVD29 with the assumption that this datum approximated mean sea level (MSL). This NGVD29 is thus a legacy or local datum. These projects have likely been constructed and monitored relative to pre-set range monuments with a "locally "published" NGVD29 reference datum. Resurveys should document the updated relationships between the NAVD88 datum and the legacy datum. This is accomplished by comparing the updated NAVD88 elevation with the legacy NGVD29 elevations on recovered local bench marks.

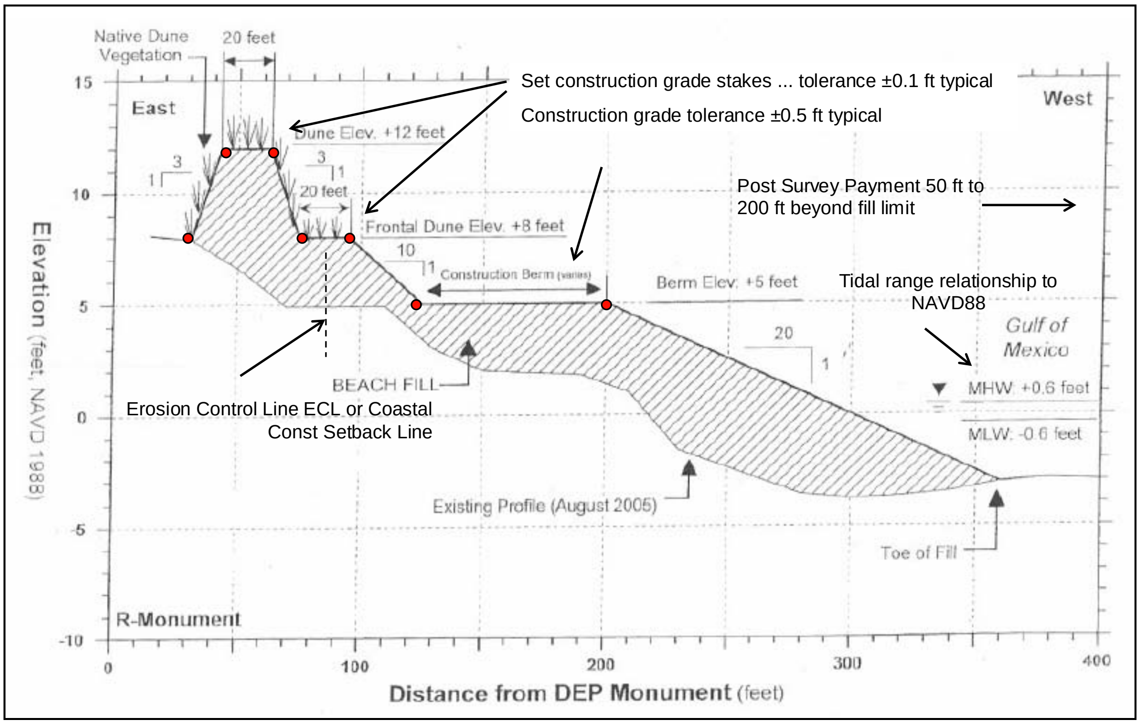


Figure 5-3. Beach profile template used for construction stake out and measurement & payment surveys.

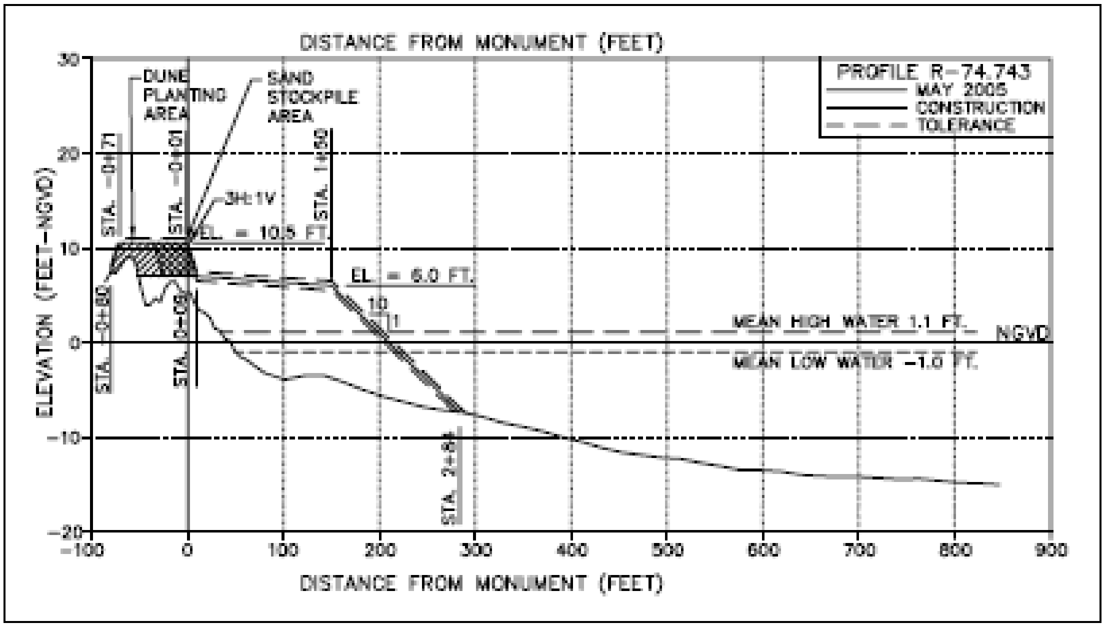


Figure 5-4. Beach profile template referenced to PBM R-74.743 on NGVD.

(1) NGVD29 datum references. In Figure 5-4, taken from construction plans, the "NGVD" elevation of the range monument "PROFILE R-74.743" was likely determined in 1974 when the range monument was set. The original or current relationship with the NSRS is probably unknown. Its "NGVD" relationship to MLW (-1.0 ft) or MHW (+1.1 ft) is likely based on the relationship at the nearest NOAA tide gage, which may be some 10 to 30 miles distant. The tidal epoch is not indicated in the drawing—a 0.25 ft to 0.5 ft tidal epoch difference may have occurred since the early 1970s. If the reference PBM and the NGVD/MSL design grade were not updated to reflect the relationship to the current tidal epoch, any new construction would be constructed 0.25 ft to 0.5 ft below the intended design elevation.

(2) Tidal datum relationships. Beach restoration projects are often distant from an established tide gage. The reference tidal datum may be estimated from nearby gages by spatial interpolation methods described in Appendix E. Such an interpolated tidal datum estimate is normally of sufficient accuracy. An interpolated tidal range between two NOAA gages would be reasonable if the tidal ranges at each gage do not vary significantly—for instance < 0.3 ft. Alternatively, if VDatum coverage exists in the area, this model may be used to estimate the relationship between MSL and the orthometric NAVD88 datum. Project documents must clearly document the source of any estimated relationships between orthometric and tidal datums.

d. Borrow area reference datums. Offshore borrow area elevations or depths should be referenced to the PPCP datum. Dredged excavation grades and borrow area surveys should be controlled by RTK or RTN measurements relative to, or calibrated from, this point. If the borrow area datum is referenced to a tidal datum (e.g., MLLW) then the relationship to the local NAVD88 datum must be documented.

e. Accuracy tolerances. Typical survey tolerances for shoreline protection projects are listed in Table 5-1.

f. Checklist. The following checklist may be used in developing control for a shoreline protection project.

(1) Is project referenced to a published NSRS control point (PPCP)?

(2) Has the PPCP elevation been checked against other published NSRS points?

(3) Are local project control points (LPCPs) firmly connected to the PPCP?

(4) Is the RTK base station situated at a PPCP or LPCP?

(5) Have beach profile reference monuments been set relative to the PPCP and LPCP?

(6) Are profile monuments correctly referenced to an established horizontal baseline?

(7) Have differential levels been run between all LPCP PBMs and range TBMs?

(8) Are leveled PBM and TBM elevations checked with RTK elevations?

(9) Is tidal datum relationship to NAVD88 established over the project site?

(10) Is the tidal datum based on current epoch?

(11) Are any legacy orthometric datums referenced to NAVD88?

g. Example application project—Fort Fisher, NC. Appendix G contains an example of a Wilmington District shore protection project that has been adequately referenced to the current NSRS orthometric datum and to the local tidal datum.

5-5. Breakwater and Jetty Construction Projects. Breakwaters and jetties constructed in tidal areas must be connected to both a local tidal design datum and NAVD88. Designed protection elevations are usually specified above MSL, although other reference tidal datums or legacy orthometric datums may be used (e.g., MLW, MHW, NGVD29). Construction and stone placement elevations will likely be referenced to NAVD88 and RTK methods used to monitor real-time placed elevations. Project control requirements to support construction and in-place surveys are the same as those required for shoreline protection projects described above.

a. Figure 5-5 depicts the control requirements for a jetty construction project. (A similar scheme would be laid out for a detached breakwater). A published PPCP needs to be established near the project site for use as an RTK base or RTN calibration. The elevation of this point is checked against nearby NSRS bench marks. Local control PBMs (LPCP) are then set inland from the jetty base. These points are surveyed relative to the PPCP using GPS or differential leveling. The LPCPs may be included in the NSRS or in the District's U-SMART database. Depending on the type of capping, access, construction requirements, etc, additional TBMs may be set in the stone along the jetty or detached breakwater.

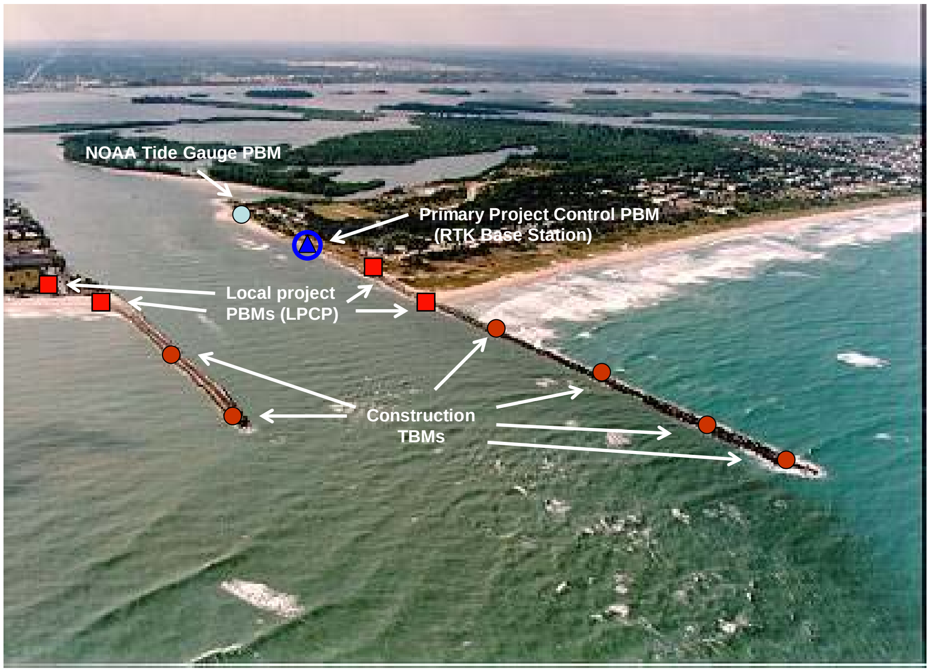


Figure 5-5. Project control requirements for jetty construction or maintenance.

b. The relationship between the orthometric and local tidal datum needs to be developed. In Figure 5-5, a NOAA gage is near the site and the connection between tidal PBMs and the PPCP can be made by GPS or differential levels. If no gage exists near the project site, then spatially interpolated or modeled (VDatum) estimates may be considered.

c. Breakwater monitoring surveys are often surveyed using a combination of subsurface multibeam hydrographic surveys and low altitude LIDAR airborne surveys. It is essential that the PPCP RTK base point for these surveys be the same point and both surveys be referenced to NAVD88 rather than tidal datums. In cases where acoustic returns are scattered by rock, lead line surveys may be required to measure voids.

5-6. Coastal Hurricane Protection Projects. Floodwalls, levees, seawalls, flood gates, pump stations, and related hurricane protection structures in coastal (tidal) areas require defined relationships between the design reference plane (normally MSL and stillwater surface elevations) and the local geodetic orthometric datum. This relationship must be established from a tidal gage at the project site or from hydrodynamically modeled spatial interpolations between tide gages. The latest geodetic, tidal datums, and tidal models established by NOAA must be used—and continually maintained and updated for epoch changes throughout the life of the project.

a. Reference tidal datum. Many hurricane protection project elevations have been designed relative to sea level datums based on interpolated or extrapolated references from gages—far and near. Depending on the type of gage, tidal range, and the distance from the gage, this interpolation or extrapolation may be valid—or sufficiently accurate—within ± 0.25 ft of the reference water level stillwater or surge design surface. Obviously, with sea level rise in many

CONUS coastal areas, the crest elevation of structures may be below that originally designed. In such cases, the original design documents should be checked to verify that allowance for sea level rise was considered in the design elevation.

b. NSRS connection. Connection to an NSRS orthometric datum need only be at the ± 0.25 ft accuracy level. This connection accuracy is usually adequate to relate protection heights to floodplain and first-floor elevations on a federally recognized reference system—e.g., NAVD88. Evaluated shore protection projects that are not on updated tidal and/or NSRS datums will require additional effort. In general, the updated sea level datum can be estimated by linear interpolation given sufficient NOAA or Corps gages exist in the region. The NSRS connection will normally be performed following the same accuracy standards and field survey specifications outlined in Chapter 3—differential leveling, static GPS baseline observations, or CORS/OPUS methods. At least one PPCP on each project shall have both a water level reference elevation and a NAVD88 elevation.

c. Example project: New Orleans to Venice Hurricane Protection Project. Figure 5-6 depicts local levee control PBMs along a portion of the Mississippi River below New Orleans to Venice, LA—West Plaquemines Levee District. Published NSRS PBMs ("N 367" and "J 370") are part of an older NGS level line that have updated time-dependent NAVD88 elevations—"NAVD88 (2004.65)."

(1) The BOOTHVILLE CORS ARP site provides real-time vertical control for GPS/OPUS observations in this region. The tidal bench mark at the Venice (Grand Pass) gage (876 0849 A TIDAL) is likewise connected to the NSRS network. These primary NSRS points provide direct PPCP control for this project area and no field surveys are required to establish additional points. Local levee control points (e.g., LPCP PIs) can be connected directly to these NSRS PPCP points using conventional survey methods, such as RTK. Levee profile elevations or cross-sections can be run directly from these PPCPs or LPCPs; however, given a Louisiana RTN covers this region, that network would be used for supplemental topographic surveys using the PPCPs for site calibration of the RTN.

(2) Since this area contains a LWRP hydraulic datum computation, this relationship between the LWRP to NAVD88 should be documented. The area also contains references to a legacy dredging datum—Mean Low Gulf (MLG). This estimated relationship must also be referenced to NAVD88.

(3) The datum relationships and uncertainties for an LPCP situated atop a levee can be tabulated as shown in Table 5-2. These relationships are based on survey observations from the NSRS network points to the LPCP. LWRP flow profile elevations are determined from interpolations between stage/discharge data at river gages that are referenced to NAVD88. The MLG datum is estimated from the Venice gage.

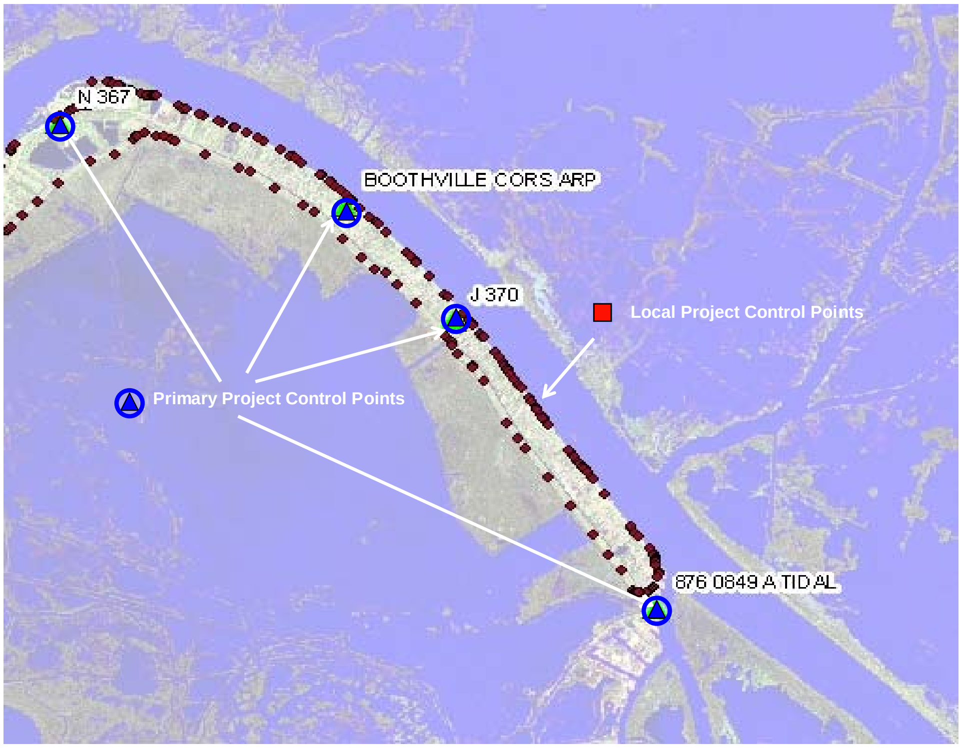


Figure 5-6. Mississippi River levee control connections with the NSRS and NOAA tide gage. Local project control points (LPCP) are connected with the PPCPs. (West Plaquemines Levee District)

Table 5-2. Elevations at a Baseline LPCP Station atop a Hurricane Protection Levee.

Datum	Elevation	Referenced From	Estimated Uncertainty	Relative to
LWRP 05	13.5 ft	River gage (interpolated)	± 0.2 ft	Profile
MSL	14.4 ft	NOAA gage 876 0849	± 0.2 ft	NWLON
LWRP 07	14.8 ft	River gage (interpolated)	± 0.2 ft	Profile
NAVD88 (2004.65)	14.93 ft	NSRS PPCP J370	± 0.05 ft	NSRS

Table 5-2 (Concluded). Elevations at a Baseline LPCP Station atop a Hurricane Protection Levee.

Datum	Elevation	Referenced From	Estimated Uncertainty	Relative to
MLLW	15.0 ft	NOAA gage 876 0849	± 0.2 ft	NWLON
NAVD88 (96)	15.0 ft	NSRS (superseded)	± 0.2 ft	NSRS
NGVD29 (98)	15.8 ft	Published NSRS	± 0.5 ft	NSRS
MLG	17.6 ft	Published NSRS (superseded NGVD29)	± 1 ft	NSRS
Ellipsoid	-64.27 ft	Geoid model	± 0.05 ft	NAD83/GRS80

d. Connection datums on miscellaneous hurricane protection structures. Elevations of floodgates, pump stations, and other structures need to be referenced to current orthometric datums and applicable legacy datums used in the original construction.

(1) Pump Station surveys. Figure 5-7 illustrates a method of determining updated elevations for a pump station. An LPCP is established near the pump station using static GPS techniques from published NSRS bench marks. A NAVD88 elevation is adjusted on the LPCP. Differential levels are run from the LPCP to a TBM or point inside the pump station that can be referenced on the as built drawings for the station—e.g., a floor elevation. An annotated photograph of the reference point is recommended, like that illustrated in Figure 5-7 for the Bayou Ducros #7 Pump Station. From this reference point, the NAVD88 invert elevations can be scaled from the as built drawings. If no as-builts are available, then invert elevations will have to be directly measured by leveling or total station—often a difficult process in confined spaces. If the pump station drawings are referenced to a local datum (e.g., New Cairo Datum (NCD) in the New Orleans area), then the NAVD88-NCD relationship is established. For example, in Figure 5-7 (Elmwood Canal Pump Station), given the first floor elevation is +15.20 ft NCD and the NAVD88 (2004.65) elevation is + 5.87 ft, the NCD-NAVD88 (2004.65) conversion factor is + 9.33 ft. Given the as built drawings show the pump invert elevation at +20.20 ft NCD, then the invert elevation is 10.87 ft NAVD88 (2004.65); computed from [20.20 ft – 9.33 ft]. If applicable, sea level datum references may also be included as an additional datum reference.

St. Bernard Parish Bayou Ducros #7 Pump Station

Jefferson Parish Elmwood Canal Pump Station JP-03
Finished Floor Elev: 5.87 ft NAVD88 (2004.65) Bk.060852, Pg. 43 (2005)

Figure 5-7. Referencing pump station elevations to NAVD88.

(2) Floodwall surveys. Figure 5-8 shows a cross-section of a hurricane protection I-wall on the New Orleans District's Inner Harbor Navigation Canal (IHNC) project. The topographic and hydrographic surveys of this section were performed relative to a local time-dependent NAVD88 datum, using RTK positioning from a NSRS PPCP. The top of wall elevation at this section shows both NAVD88 and Local Mean Sea level (LMSL) references.

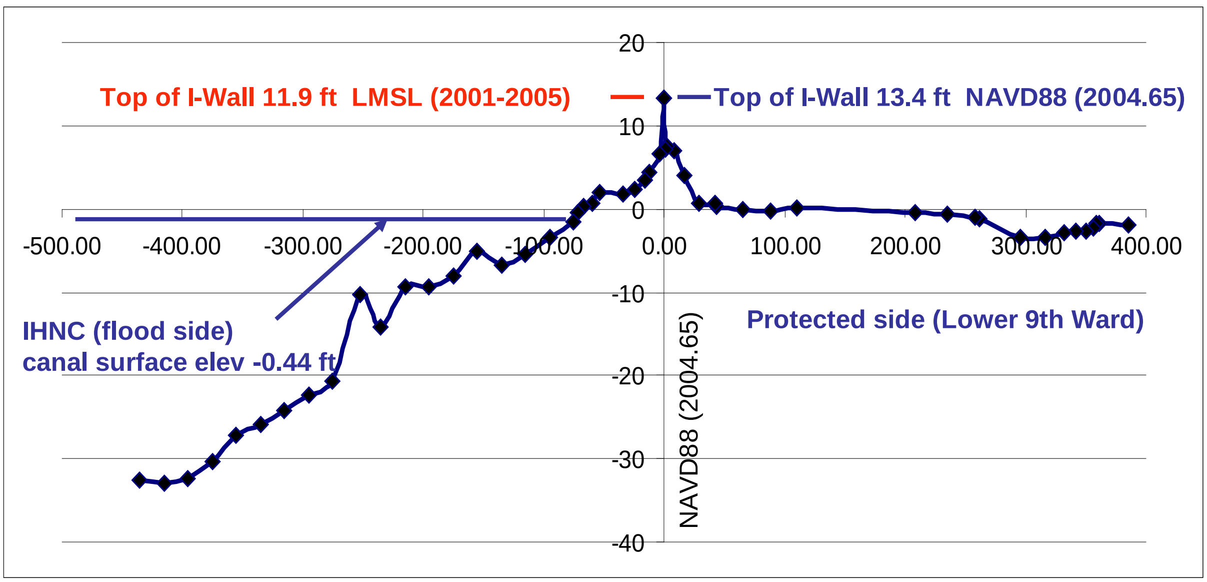
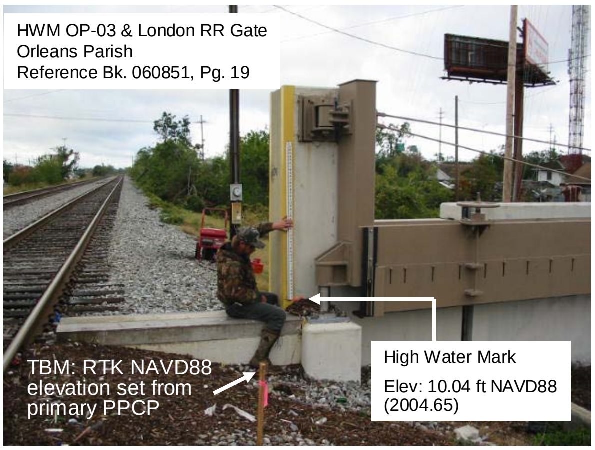


Figure 5-8. Referencing top of floodwall elevation to orthometric and sea level datums.

(3) Post hurricane high water mark (HWM) surveys. High water marks must be on a datum consistent with hydrodynamic surge models or other post-hurricane inundation studies. Therefore it is important that the reference datum used for HWMs be readily convertible to the model or study reference datum, such as LMSL. Using NAVD88 as a consistent reference is recommended, provided that the relationship between NAVD88 and LMSL can be estimated

throughout the region. Required accuracies of HWM elevations are dependent on the precision of the HWM, and the observer who estimated (marked) the HWM. Absolute field marking precisions of HWMs will range between 0.2 and 1 ft; thus, RTK survey methods are usually adequate. As shown in Figure 5-9, a TBM (stake or PK nail) is set in the vicinity of the HWM and a RTK (or fast static) elevation is placed on the TBM. Differential levels or total stations are then employed to survey the HWM on a wall or into the interior of a structure. HWM surveys, resultant elevations, and the reference datum must be clearly documented as shown in the figure.



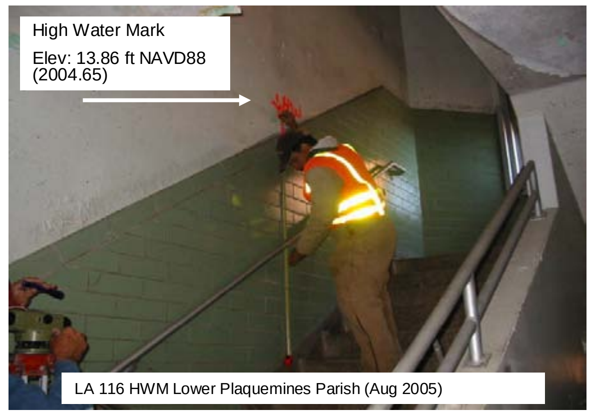


Figure 5-9. High water mark survey procedures and datum documentation.

(4) Monitoring elevations of floodwalls and bridge restrictions. In areas subject to hurricane surge, elevations of designed or constructed structures need to be surveyed. Figure 5-10 depicts topographic surveys of a floodwall intersection with a bridge floodwall. The reference LPCP elevation on the floodwall was positioned using static or fast-static DGPS methods from the surrounding primary control network. From this LPCP point differential levels were run to obtain the elevations of the floodgate, connecting I-wall, bridge floodwall, and the bridge low chord elevation. The LPCP should be described and placed in the District's control database (e.g., U-SMART). As with all HSPP structure surveys, photographic documentation of observed elevation points is recommended. The relationship between local MSL and legacy (constructed) datums may also be noted on the photo. The reference field survey book should always be included as this book will contain additional metadata as to the survey procedures, elevation datums, source PPCPs, etc.

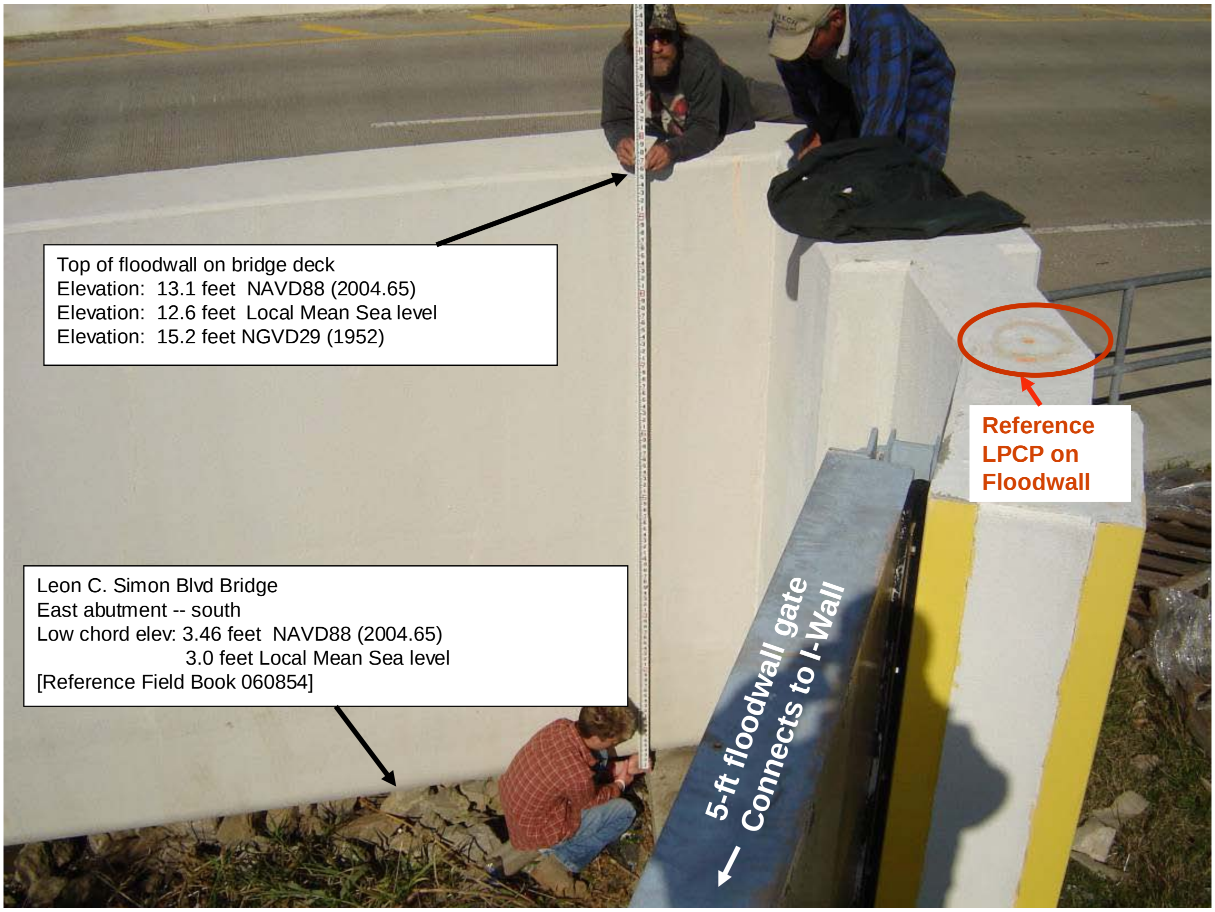


Figure 5-10. Surveying elevations of bridge chords, flood gates, and floodwalls.

CHAPTER 6

Procedures for Referencing Datums on Inland Flood Risk Management, Water Control, and Navigation Projects

6-1. Purpose. This chapter provides guidance for referencing project elevation grades on various inland flood risk management, water control, and navigation projects. Inland projects are defined as those with minimal or no tidal influence—their reference datums are defined relative to (1) modeled or measured hydraulic flows in rivers, (2) pool elevations in controlled reservoirs or between locks and/or dams, or (3) established reservoir and inland lake low water datums. This chapter outlines the field survey procedures needed to ensure these projects and their reference water level gages are adequately connected to the National Spatial Reference System (NSRS) orthometric datum established by the Department of Commerce.

6-2. Reference Grades on Inland Flood Risk Management and Water Control Projects. The relationship between geodetic and hydraulic datums on inland projects was outlined in Chapter 1 (Figures 1-2 and 1-3). In these figures, the relationships between the river stage, local gage, and terrestrial geodetic elevation(s) must be determined for each project. On river systems, these relationships are not constant, and vary spatially with the slope of the low water reference plane, design flood elevation, and geodetic datum readjustments. This relationship must be firmly established—either from direct river gage connections or from modeled hydraulic interpolations between gages.

a. Water surface elevation references. (See Chapter 2). The water surface elevation, or stage, is typically referred to a hydraulically based reference plane: e.g., Low Water Reference Plane (LWRP) in open flow rivers, or, in controlled areas, Project Pool, Low Water Pool, Normal Pool, Ordinary High Water Profile (OHWP), etc. The local river or pool gage may have its own reference datum from which flood stages are defined. The height of a levee or other water control structure is designed relative to predicted flood stage elevations on the river, along with other related design factors.

b. Flood protection or water control structure elevations. The reported elevation at the top of a floodwall, levee, or dam may have elevations based on more than one reference datum. The two minimum required reference elevations must include:

(1) Orthometric elevation directly referenced to current NSRS (NAVD88).

(2) Hydraulic elevation referenced to a defined reference plane (LWRP, pool, gage zero, etc.).

c. Other optional elevation references may include:

(1) Flood stage height reference.

(2) Ellipsoid height—based on local NSRS geoid model—useful for performing RTK topo surveys.

(3) Base Flood Elevation (BFE) reference—from local studies or models.

(4) NGVD29 elevation—usually based on older reference monuments or as-built drawings.

(5) Local USACE legacy datums—e.g., Cairo Datum, Sea Level Datum, MSL 1912.

d. Elevation uncertainties. Figure 1-2 in Chapter 1 illustrated the uncertainties in vertical datums on an inland flood risk management project. Each reference datum listed above has some statistical uncertainty level that must be considered in protection reliability, risk assessment models, and levee/floodwall certification. These datum uncertainties must be estimated for each project. NSRS (NAVD88) regional relative accuracy estimates may be obtained from NGS observation adjustment statistics. These NSRS uncertainties propagate down to local project control point (LPCP) elevations and then to topographic surveys of project grades. Methods for estimating the propagated uncertainties of elevation grades are described in Chapter 9. The accuracy of local hydraulic stage/flowlines may be more difficult to estimate—see Section 6-4. Pool or reservoir stage elevations (and local low water datums) are usually well defined based on long-term gage data. Legacy datum references (e.g., NGVD29) may have highly uncertain accuracies and origins.

e. River gage NSRS (NAVD88) elevation connection requirements. Figure 6-1 depicts a river gage connection to a floodwall cap elevation—shown here to a PBM set atop the wall. In this example, a river gage on LWRP or pool datum is connected with the regional NAVD88 network; thus providing an external (i.e., ellipsoidal and orthometric) reference for the gage, along with the LWRP or pool hydraulic profile reference for the floodwall cap. The relationship between the orthometric height, ellipsoidal height, geoid height, and the hydraulic elevation is shown.

(1) In the top part of Figure 6-1, a level run from the gage reference point (20.0 ft) established an elevation on the top of the I-wall at 40.35 ft (LWRP).

(2) In the lower part of Figure 6-1, differential GPS observations measured an ellipsoid height of 278.02 ft at the PBM atop floodwall. Given a published geoid height of (-) 94.03 ft for this area, the orthometric NAVD88 elevation of 372.05 ft is determined using the relationships outlined in Chapter 2.

(3) The NGVD29 elevation shown in Figure 6-1 (371.76 ft) is based on a modeled (VERTCON or CORPSCON) difference of 0.29 ft from NAVD88. This modeled relationship may be accurate to only ± 0.5 ft, depending on many factors. Legacy or local datums maintained on floodwalls, such as NGVD29, should always be caveated with appropriate metadata, to include estimated age, reliability, and accuracy.

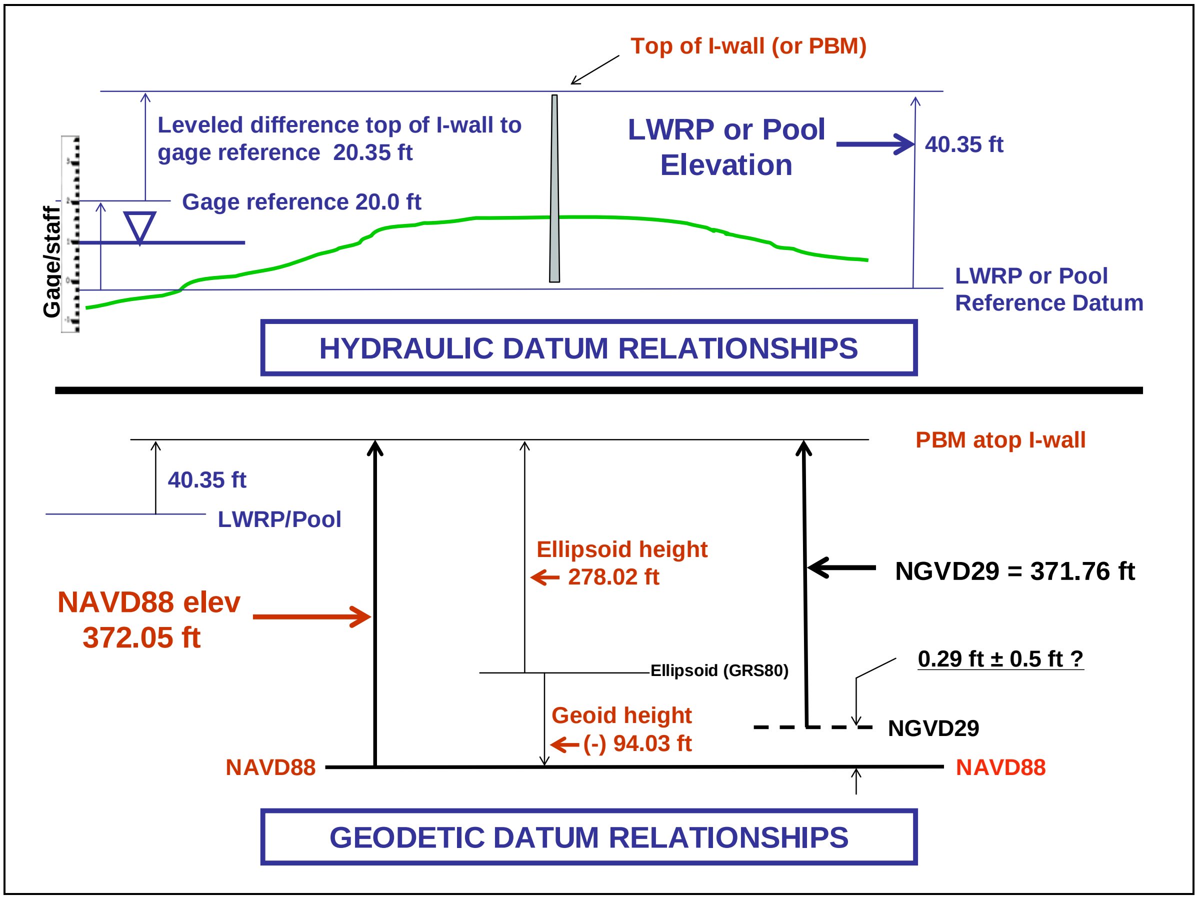


Figure 6-1. Orthometric height and hydraulic reference datum relationships on a floodwall.

6-3. Procedures for Connecting Inland River, Pool, or Reservoir Gages to the National Spatial Reference System. Inland river gages that are used to reference and model USACE floodwall design elevations should be firmly referenced to NAVD88, and be included within the NSRS. This is intended to insure these gages are on the same regional (nationwide) vertical datum used for hydrologic and hydraulic studies performed by USACE and other agencies. Gages referenced to unknown, legacy, or superseded datums (e.g., MSL 1912, NGVD29) must also be referenced to NAVD88.

a. Reference bench marks at river gages. A minimum of three bench marks shall be established around a river gage that is used to reference flood protection elevations on nearby levees or floodwall systems. Only one of these reference points needs to be connected to and published in the NSRS, using either CORS baseline observations, differential levels, or DGPS baseline observations (e.g., static or fast/rapid static methods). The remaining river gage bench marks and the gage zero reference can be surveyed using Third-Order differential leveling methods from the primary bench mark. Data for the gage's primary reference mark (PPCP) shall be incorporated into the NSRS database.

b. Reference bench mark descriptions—periodic inspections. The description for the gage's reference bench mark must contain, in addition to the standard location data, full metadata associated with that bench mark and its nearby river gage. For example:

Bench Mark:	*USED RIVER GAGE 12345 1955*
River Gage:	*[River gage name/file designation]*
Elevation:	*419.63 ft NAVD88 ± 0.22 ft [2008 03 21 adjustment]*
Elevation:	*40.35 ft above LWRP 20XX [2008 03 21]*
Elevation:	*20.35 above river gage zero reference [2008 03 21]*
Elevation:	*3.38 ft above 12345 RM 1 [2008 03 21]*
Elevation:	*0.97 ft below 12345 RM 2 [2008 03 21]*
Position:	*[SPCS X & Y location/accuracy/date]*
Source:	*[specify NGS "PID" and District file number]*

Subsequent bench mark "Recovery Notes" made at periodic (e.g., semi-annual, annual, biannual) gage inspections should also update the gage reference and adjacent reference bench mark connections. The NSRS datasheet in Figure 6-2 contains a simulated example of an inspection recovery note made for a reference bench mark at a river gage. The last gage inspection in 2009 noted that levels were run between the gage and reference bench marks. The results and differences from the 2008 elevations are included on the description. This example utilizes the NSRS for recording gage inspection recoveries in that the reference bench mark is published. Other documentation formats for gage inspections may also be used—e.g., U-SMART (see Chapter 3). An example U-SMART description for an inflow gage in Pittsburgh District is shown in Appendix H.

```
XX999  DESIGNATION -  CEMVS GAGE 12345 RM 1
XX999  PID         -  XX999
XX999  STATE/COUNTY-  MO/C OF ST LOUIS
XX999  USGS QUAD   -  GRANITE CITY (1998)
XX999                         *CURRENT SURVEY CONTROL
XX999  ___________________________________________________________________
XX999* NAD 83(1986)-  38 00 00.     (N)    090 00 00.     (W)     OPUS
XX999* NAVD 88     -       127.903  (meters)     419.63   (feet)  ADJUSTED
XX999  ___________________________________________________________________
XX999  GEOID HEIGHT-         -31.08  (meters)                      GEOID03
XX999  DYNAMIC HT  -         127.821 (meters)     419.36  (feet)  COMP
XX999  MODELED GRAV-     979,991.3   (mgal)                        NAVD 88
XX999  VERT ORDER  -  FIRST     CLASS II

XX999                          STATION DESCRIPTION  (2008)

XX999'DESCRIBED BY USAED ST LOUIS 2008 03 21 (R MESKO)
XX999'IN ST LOUIS, 1.35 KILOMETERS (0.85 MILE) SOUTH ALONG THE FLOOD WALL OF
XX999'THE MISSISSIPPI RIVER FROM THE GOLDEN ARCH BRIDGE OVER THE RIVER, SET
XX999'VERTICALLY IN THE EAST END OF THE ONLY LARGE SOLID PIER OF A RAILROAD
XX999'OVERPASS THAT LEADS WEST OVER THE TRACKS THAT PARALLEL THE FLOOD WALL,
XX999'AND 19.21 METERS (63.0 FEET) WEST OF THE WEST FACE OF THE FLOOD WALL.
XX999'THE MARK IS THE PRIMARY REFERENCE POINT FOR COE RIVER GAGE NO. 12345
XX999'WHICH IS APPROX 30 FT NORTH OF THE MARK.
XX999'THE MARK IS 1.04 METERS N FROM A WITNESS POST.
XX999'THE MARK IS 1.12 M ABOVE GROUND.
XX999'RIVER GAGE NO 12345 LEVELING REFERENCES RUN 2008 03 21
XX999'THE MARK IS 20.35 FT ABOVE THE ZERO GAGE REFERENCE POINT
XX999'THE MARK IS 40.35 FT ABOVE LWRPXX
XX999'THE MARK IS 3.38 FT ABOVE 12345 RM 2, A COE DISC LOCATED ON THE
XX999'FLOODWALL 45.6 FT NORTH.
XX999'THE MARK IS 0.97 FT BELOW 12345 RM 3, A COE DISC LOCATED ON THE
XX999'FLOODWALL 89.4 FT SOUTH.

XX999                          STATION RECOVERY (2009)

XX999'RECOVERY NOTE BY USAED ST LOUIS 2009 06 05 (R MESKO)
XX999'RECOVERED MARK AND RM1 AND RM2 IN GOOD CONDITION, AS DESCRIBED.
XX999'RELEVELING RESULTS FROM 2009 06 05 GAGE INSPECTION:
XX999'THE MARK IS 20.34 FT ABOVE THE ZERO GAGE REFERENCE POINT
XX999'THE MARK IS 40.34 FT ABOVE LWRPXX
XX999'THE MARK IS 3.39 FT ABOVE 12345 RM 2, A COE DISC LOCATED ON THE
XX999'FLOODWALL 45.6 FT NORTH.
XX999'THE MARK IS 0.97 FT BELOW 12345 RM 3, A COE DISC LOCATED ON THE
XX999'FLOODWALL 89.4 FT SOUTH.
```

Figure 6-2. Simulated NSRS datasheet with gage inspection recovery notes.

6-4. <u>Elevation Accuracy Requirements at Reference Gages</u>.

a. Hydrological and hydraulic accuracy requirements. In order to best define the governing accuracy standard required for connecting primary project control monuments to the regional NSRS, it is necessary to understand the hydraulic engineering requirements and applications for such connections. River, pool, or reservoir gages normally have highly accurate local datums—i.e., at the gage itself. However, the absolute vertical relationship between adjacent gages is often uncertain, as was illustrated in Figure 1-2. For example, two river gages separated by 20 miles may have been referenced to orthometric datums 50 years ago—e.g., SLD29 or NGVD29. Surveys were never performed directly connecting the reference bench marks at these gages. The uncertainty between the connections could exceed 0.5 ft or more—see "*Mapping the Zone: Improving Flood Map Accuracy*" (NRC 2009). Calculated water surface profiles could also vary by as much as foot or more, depending on numerous factors. These uncertainties and variations need to be considered in defining relative NSRS accuracy requirements for reference bench marks at a gage. The following excerpt from NRC 2009 illustrates the considerations needed in determining the NSRS accuracy requirements for adjacent river gages:

> "*... frequency analysis of stage height is not the same thing as frequency analysis of base flood elevation because the BFE is defined relative to an orthometric datum [NAVD88] ... and the stage height is defined relative to an arbitrary gage elevation datum. However, it is not necessary to reconcile these datums because what we are seeking is not the elevation itself, but rather the uncertainty of the elevation. The difference between the stage height and the flood elevation is the fixed datum height that is the same for all measurements and thus does not affect their variations from year to year*."

The nominal ± 0.25 ft relative accuracy standard in Table 3-1 is believed to be adequate for most USACE projects; fully considering hydraulic design models, design criteria, construction tolerances, and other federal and state agency accuracy requirements.

b. Accuracy estimates. Region-wide hydraulic accuracy requirements can also be estimated using the guidance in "*Accuracy of Computed Water Surface Profiles*" (HEC 1986). This study assessed the survey accuracy required to achieve desired profile accuracies, as illustrated in Table 6-1.

Table 6-1. Survey Accuracy Requirements[1] for Specified Profile Accuracies. (HEC 1986)

		Manning's n-value reliability $N_r = 0$		Manning's n-value reliability $N_r = 1$	
Stream Slope (ft/mile)	Profile Accuracy Emean [2] (ft)	Aerial Survey Contour Interval	Topo Map Contour Interval	Aerial Survey Contour Interval	Topo Map Contour Interval
1	0.1	10 ft	n/a	n/a	n/a
1	0.5	10 ft	5 ft	n/a	n/a
1	1.0	> 10 ft	10 ft	10 ft	2 ft
1	1.5	> 10 ft	10 ft	10 ft	5 ft
1	2.0	> 10 ft	10 ft	>10 ft	10 ft
10	0.1	2 ft	n/a	n/a	n/a
10	0.5	10 ft	5 ft	n/a	n/a
10	1.0	10 ft	5 ft	10 ft	n/a
10	1.5	> 10 ft	10 ft	10 ft	2 ft
10	2.0	> 10 ft	10 ft	10 ft	5 ft
30	0.1	2 ft	n/a	n/a	n/a
30	0.5	10 ft	2 ft	n/a	n/a
30	1.0	10 ft	5 ft	10 ft	n/a
30	1.5	> 10 ft	10 ft	10 ft	2 ft
30	2.0	> 10 ft	10 ft	10 ft	5 ft

[1] Denotes maximum survey contour interval to produce desired survey accuracy
[2] Emean is "mean absolute reach error"

c. From Table 6-1, given the allowable error in a water surface profile, and considering other hydraulic factors, the required accuracy of topographic data (e.g., stream cross-sections) can be estimated. Topographic survey accuracies in this older publication are defined relative to a National Map Accuracy Standard (NMAS) contour interval parameter. These can be converted to NSSDA 95% confidence standards. HEC 1986 should be reviewed in order to appreciate the impact (or often lack thereof) of survey accuracy on computed water surface profiles. For example, if a hydrological or hydraulic water surface profile model is sensitive to cross-sectional accuracy at the ± 2 ft (NSRS) level, there would then be no point in requiring control points for these sections to be accurate to the ± 0.1 ft (NSRS) level.

d. Regional gage accuracy for levees and related flood protection projects. The Bois Brule Levee and Drainage District in St. Louis District represents a typical main-stem Mississippi

River levee system (see Appendix I). On this levee segment the river slope drops approximately 13 ft over a 12-mile reach. Given the magnitude of the elevation change over this 12-mile distance, the design levee grades between each end of the system would not need a high level of accuracy relative to the NSRS. A ± 0.25 ft to ± 0.5 ft relative accuracy between the northerly and southerly limits would be adequate for most engineering purposes. These levels of accuracy can be easily achieved with static DGPS, CORS/OPUS, or conventional differential leveling methods.

e. Gage accuracy requirements in low gradient areas. Many floodplain areas with minimal water surface gradients will require more accurate elevation connections with the NSRS in order to measure flow or inundation conditions. An example is the Jacksonville District's Central and Southern Florida (C&SF) project that contains a vast network of levees, canals, control gates, pump stations, and other control structures throughout southern Florida; including portions of the Everglades. In some cases, elevation differences over a few miles may be only 0.2 ft. To provide a consistent geodetic reference framework for this project, the USACE and NGS performed high accuracy "height modernization" control surveys at hundreds of PBMs throughout the region. These geodetic surveys included a network of static GPS baselines and high-order differential leveling. Geodetic survey standards followed NGS specifications and resultant "PPCP" data were adjusted and published by the NGS—see "*Guidelines for Establishing GPS-Derived Ellipsoid Heights (Standards: 2 cm and 5 cm)*" (NOAA 1997) and "*Guidelines for Establishing GPS Derived Orthometric Heights (Standards: 2 cm and 5 cm) version 1.4* " (NOAA 2005). These NSRS PPCPs were subsequently used to establish LPCP elevations near gages and pump stations.

6-5. Levee System Connections to the NSRS. The following guidelines are recommended procedures to establish PPCPs suitable for defining project elevations relative to the NSRS. They are based on the nominal target accuracy standard of ± 0.25 ft relative to the published NSRS. Field survey methods for performing the connections were covered in Chapter 3. These guidelines are also applicable to other water control and inland navigation projects covered in this chapter.

a. General PPCP requirements. Each levee project or segment should have at least one PPCP that has been connected to the NSRS. On large levee projects, PPCPs should be spaced between 15 to 20 mile intervals. This recommended spacing is dependent on local RTK or RTN capabilities, district survey standards, and the distance from the PPCP to the levee. In general, the PPCP should be located relatively close to the project in order to establish supplemental LPCP connections using GPS static or rapid static baseline methods—see Figure 6-3. If the project is covered by an RTN, then a less dense network of PPCPs will be required for RTN site calibration. When multiple PPCPs are established on a levee segment, it is recommended that these PPCPs be interconnected with static GPS baseline observations, along with observed baselines to nearby CORS points.

b. Supplemental LPCPs. LPCPs on a levee segment are connected with the PPCPs using the various survey methods described in Chapter 3. The density of LPCPs is site dependent. In general, static or RTK techniques are most efficient to perform this connection. Descriptions for these points should be included in a district project control database (e.g., U-SMART) for permanent retention. The NSRS elevations determined for these LPCPs should not supersede local construction or legacy elevations—see Chapter 3.

c. Water level gage connections. Reference PPCP bench marks need to be set near each water level gage associated with a levee system or water control project. These gage reference marks will be designated as “Primary Project Control Points" and will be connected to the NSRS using any of the methods covered in Chapter 3. Level ties between the gage reference PBMs are also required and elevation differences and associated metadata should be included in the NSRS bench mark description or the U-SMART database system.

d. Bench mark construction. Bench mark construction for new PPCPs and LPCPs points will follow the guidance in EM 1110-1-1002 (*Survey Markers and Monumentation*). Type C (USACE disk set in existing concrete structure) marks are recommended. Type F and Type G marks (disk attached to shallow rod or rebar), or other suitable type marks/monuments, are also acceptable as bench marks. Geodetic survey quality deep-driven mark stability is not required. At least three reference bench marks are required at each gage.

e. Summary of PPCP and LPCP connections. Figure 6-3 summarizes the general NSRS connection requirements for PPCP control on a levee segment. PPCP elevations were established by GPS or leveling observations to surrounding NSRS bench marks. LPCP connections with the PPCPs are also indicated. Topographic site plan surveys would be performed relative to the nearest LPCPs on the levee.

f. Application projects. The following appendices contain examples of USACE inland projects that have been connected to the NSRS.

(1) Appendix H. East Branch Clarion River Dam and Spillway Control Surveys (Pittsburgh District),

(2) Appendix J. Establishing NSRS Elevations on 15 Dam and Reservoir Projects in Pittsburgh District.

(3) Appendix I. St. Louis District: Control Surveys--Bois Brule Levee and Drainage District.

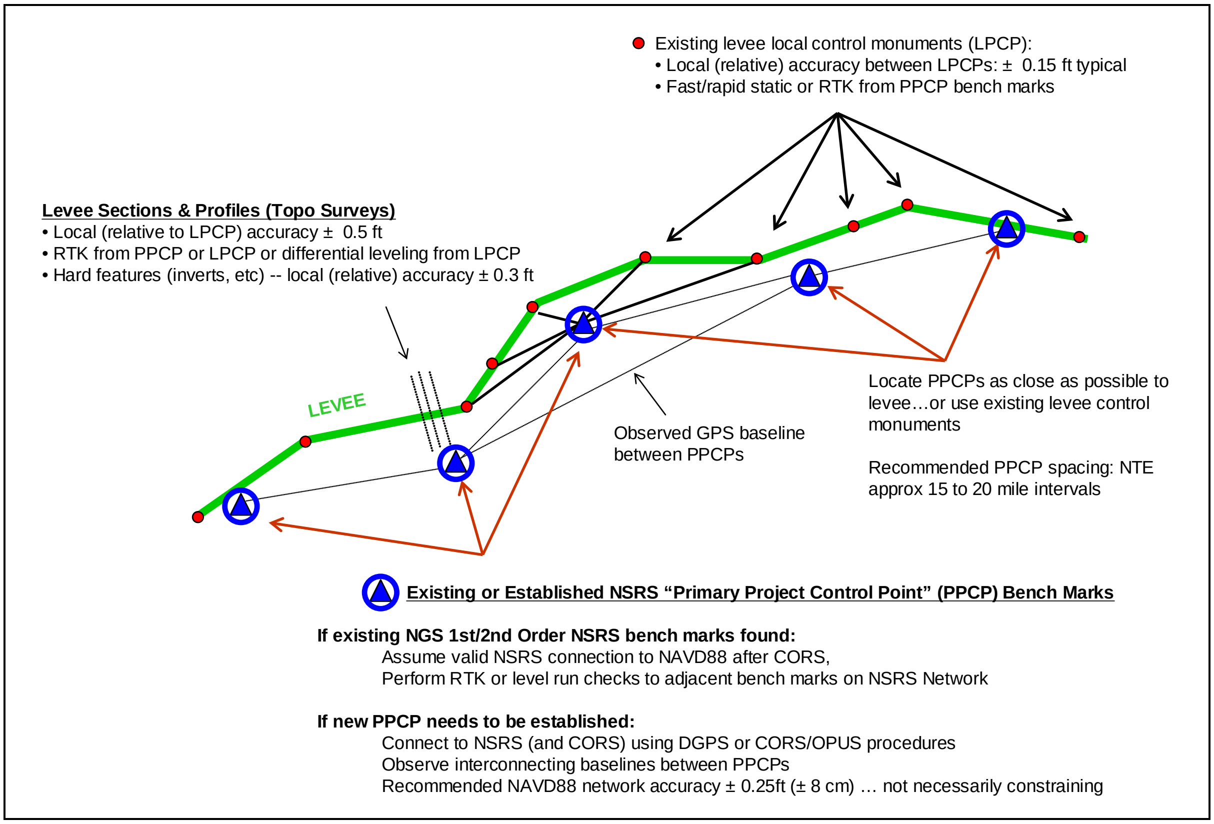


Figure 6-3. NSRS and local reference bench marks on a levee system.

6-6. Dam and Reservoir Connections to the NSRS. For most dam projects, multipurpose hydropower projects, and related impoundment reservoirs, only one PPCP needs to be tied in to the NSRS, as illustrated in Figure 6-4. This point will usually be one of the deformation and settlement monitoring bench marks used on periodic inspection surveys. NSRS referenced elevations for all the other monitoring points can be computed from the elevation differences in the most recent periodic inspection report—these differences are generally accurate to 0.005 ft tolerances when precise geodetic leveling techniques are employed. The PPCP at a dam and reservoir project can be referenced to the NSRS using the survey techniques covered in Chapter 3. In general, CORS/OPUS observations will be adequate to provide a reliable reference to the NSRS. In some instances, networked baseline connections may be selected.

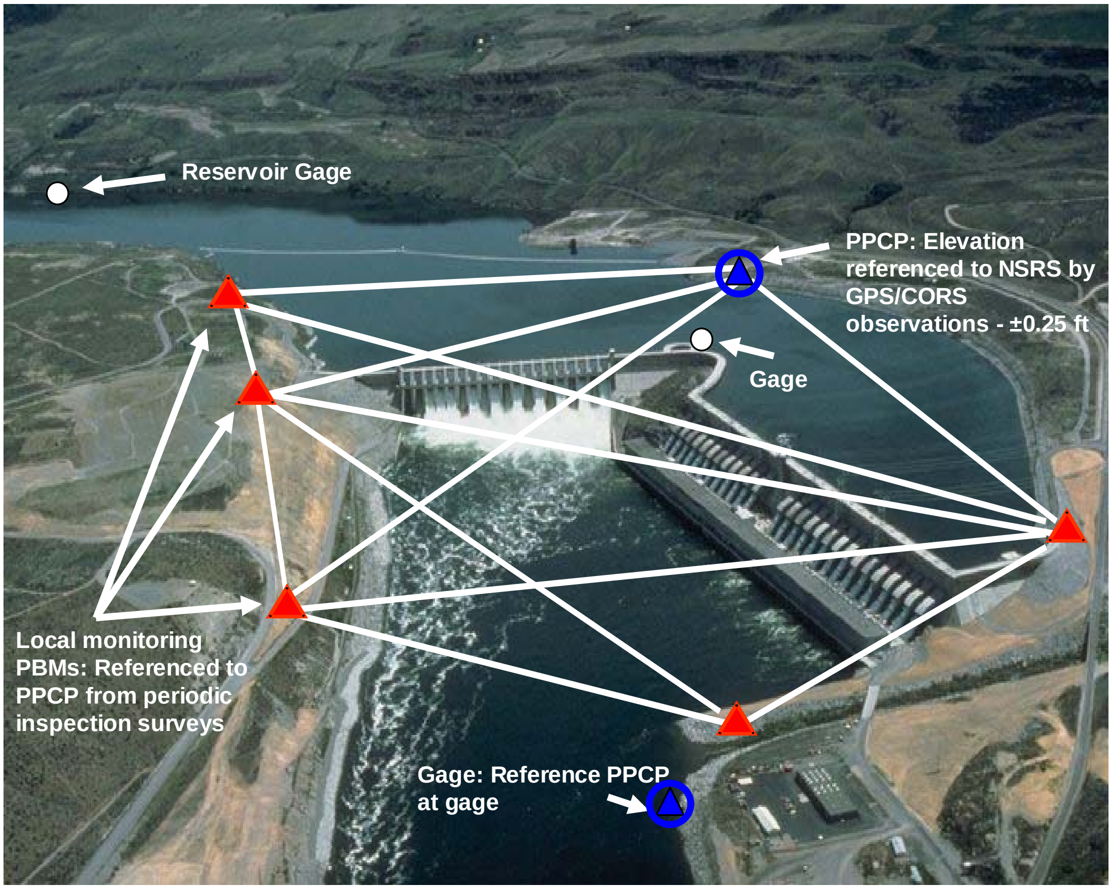


Figure 6-4. NSRS connections at a multipurpose hydropower project.

a. The guidance in this section is not intended to modify local datums used on periodic deformation monitoring inspections or for water control purposes. The intent is to add a reliable NSRS reference to these structure points and any gages near the dam and reservoir. This allows pool, intake, spillway, and crest elevations to be externally referenced to the current NSRS reference datum. The nominal ± 0.25 ft relative NSRS accuracy will normally be suitable for most projects. Exceptions may exist for large impoundment reservoirs. Detailed guidance for monitoring relative elevations around dams is covered in EM 1110-2-1009 (*Structural Deformation Surveying).*

b. Vertical settlement monitoring networks are often referenced to an arbitrary datum—i.e., 100.00 ft or 1,000.00 ft at the primary reference point held fixed in the network. Relative elevation accuracies (not NAVD88 accuracies) are most critical. For example, a monitoring plug on a concrete monolith may have a local vertical precision of ± 0.001 ft relative to an adjacent monolith plug, and perhaps ± 0.005 ft relative to the external monitoring network some 500 ft to 1,000 ft distant. In many cases, the monolith point is also referenced to a superseded legacy vertical datum, such as NGVD29. Thus, a monolith plug could have two defining elevations—e.g., 104.678 ft on the deformation monitoring network datum and 786.3 ft (± ? ft) on the legacy NGVD29 datum. Once the network is updated to the NSRS, the NAVD88 elevation for this same monolith point might be 784.82 ft ± 0.2 ft. This NSRS elevation may have been obtained from static GPS baseline observations to NSRS points 5 to 10 miles distant, and/or CORS points 50 to 150 miles distant.

c. Gages near the dam or in the reservoir must also be referenced to the NSRS elevation datum. Procedures and documentation are similar to that required for levee projects in the above paragraphs. The relationships with legacy orthometric datums and reservoir pool datums must be documented at each gage.

d. Appendix H contains an example of geodetic surveys that were performed to establish primary project control on Pittsburgh District's Clarion River (East Branch) Dam and Spillway. See also Appendix J.

6-7. Inland Navigation Lock and Dam Connections to the NSRS. Inland lock structure elevations (and related gate structures, sills, guide walls, dams, etc.) are connected to the NSRS similarly to dam and reservoir projects described above. Figure 6-5 depicts a control scheme for a typical lock structure. Upstream and downstream gages should be referenced to the NSRS at this lock. As with dam and reservoir projects described above, NSRS (NAVD88) elevations will not supersede local construction, water control, or deformation monitoring elevations on LPCPs used at the site. The relationships between NAVD88 and legacy orthometric datums and controlled pool datums upstream and downstream of the lock must be documented at each gage.

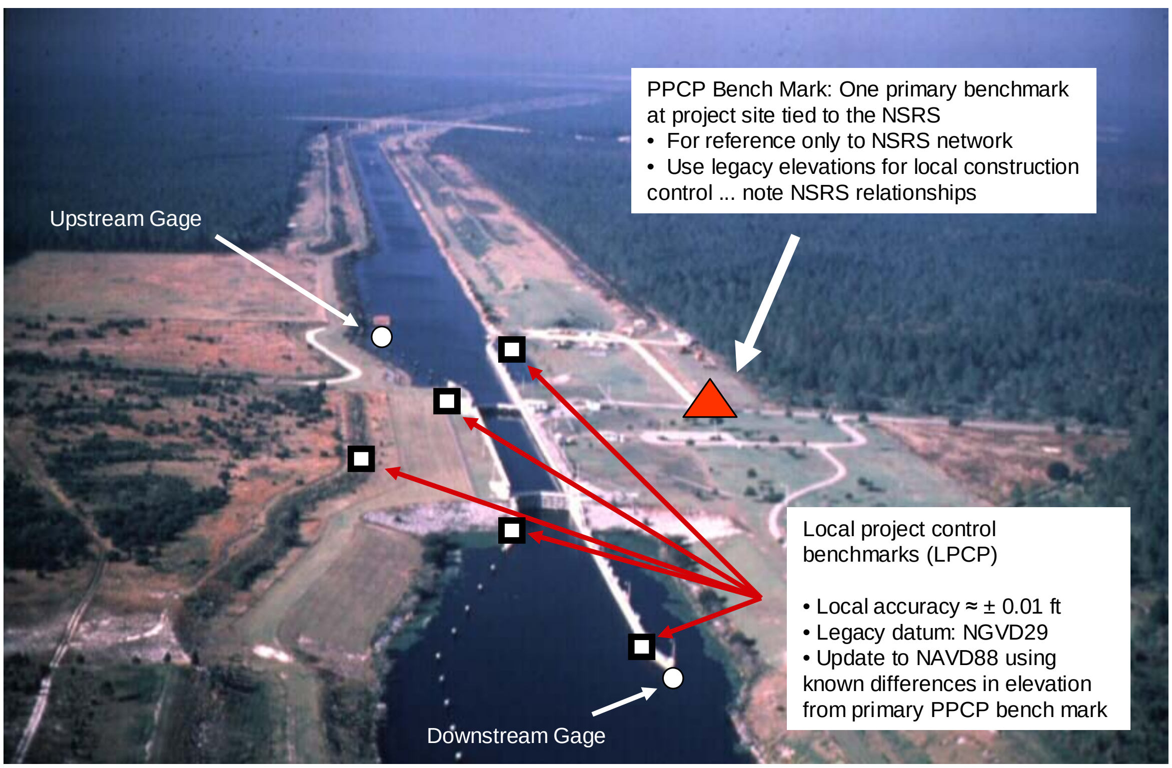


Figure 6-5. NSRS connections at an inland lock project.

6-8. Referencing Projects on the Great Lakes and Connecting Waterways. Navigation and shore protection projects on the Great Lakes and connecting waterways are referenced to the International Great Lakes Datum (IGLD). IGLD is specified by the year of the adjustment (currently 1985). Each of the five Great Lakes has its own independent IGLD low water reference datum, as listed in Chapter 2 (Table 2-2). The IGLD85 and its relationship to NAVD88 is determined and defined by the International Joint Commission—see "*Establishment of International Great Lakes Datum--1985*" (IJC 1995). IGLD elevations of bench marks are published in the NSRS.

a. The datum references in the connecting channels (i.e., Niagara, Detroit, St. Clair, and St.Marys Rivers) between the fixed datums at each lake are developed by the local USACE districts. See a complete listing in Appendix K.

b. Primary project control point connections to the NSRS (i.e., IGLD85 or NAVD88) would follow similar guidance outlined for navigation, shoreline protection, and flood risk management projects in this manual.

c. Both NAVD88 and IGLD85 are referenced to the same primary bench mark at Pointe-au-Père/Rimouski, Quebec. The only difference is that the IGLD85 elevations are published as dynamic heights and the NAVD88 elevations are published as Helmert orthometric heights.

d. Due to inaccuracies in NAVD88 leveling adjustments in the Great Lakes region, a “hydraulic corrector” must be applied at subordinate points on the Great Lakes in order to obtain a reference datum for engineering, construction, or navigation projects. These hydraulic correctors at gage sites are published by the IJC Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data.

e. Appendix K contains additional technical information on IGLD85 dynamic heights and hydraulic correctors. It also includes an example of a Detroit District navigation project in Lake Superior that is referenced to IGLD85.

CHAPTER 7

Vertical Datums Applicable to Regulatory Permitting

7-1. Purpose and Applicability. This chapter provides general background information on reference elevation grades that may be cited on permit applications or mitigation site plans. ER 1110-2-8160 (*Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*) notes that vertical reference datums on permit applications should conform to accepted local and state requirements; thus, legacy or local reference datums will usually supersede requirements to reference projects to the NSRS (currently NAVD88) or NOAA NWLON tidal datums. Therefore, the technical guidance in this chapter is intended to cover only those permit applications or mitigation projects where the reference to a nationwide vertical NSRS or NWLON datum may be applicable.

7-2. Vertical Reference Datums Used in Regulatory Activities. Table 7-1 lists some of the reference datums that may be applicable to permitting or mitigation projects. These reference datums are also illustrated in Figures 7-1 and 7-2 and further described in subsequent sections. In tidal areas, established datum limits defined by NOAA are measured based on gage readings—e.g., MSL, MLW, MHW. Reference data at these gages may also be related to NSRS (NAVD88) datums. Limits defined by more subjective observations (OHWM, MHT) do not necessarily have hydraulic or tidal gage measurement definitions; however, OHWM or MHT points or contours on permit applications may have been surveyed relative to established NSRS (NAVD88) datums. Many permit applications reference water surface and excavation/fill elevations to superseded legacy orthometric datums (e.g., NGVD29) under the erroneous assumption this datum relates to the current epoch of LMSL.

7-3. Ordinary High Water Mark (OHWM) Determination. OHWM is a jurisdictional benchmark for administering the USACE regulatory program in navigable waterways under Section 10 of the Rivers and Harbors Appropriations Act of 1899 (33 U.S.C. 403) and Section 404 of the Clean Water Act of 1977 (33 U.S.C. 1344). The term "Ordinary High Water Mark" is defined in 33 CFR 328.3 (*Definitions of Waters of the United States*) to "mean that line on the shore established by the fluctuations of water and indicated by physical characteristics such as clear, natural line impressed on the bank, shelving, changes in the character of soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas."

a. OHWM interpretation. Various interpretations of OHWMs exist throughout CONUS, given the subjective site-dependent nature of the definition—e.g., estimated vegetation line limits. For example, a OHWM is also defined as "where the banks of a body of water are relatively steep, the OHWM is coordinate with the limit of the bed of the water; and that, only, is to be considered the bed which the water occupies sufficiently long and continuously to wrest it from vegetation and destroy its value for agricultural purposes."

Table 7-1. Vertical Datums used in Regulatory Permitting Authorities.

Authority [1]	Geographic Area	Activity	Typical Reference Datums
SECTION 10 Rivers & Harbor Act of 1899	Navigable Waters of United States	All work over, through, and under navigable waters (e.g., dredging, docks, beach renourishment)	Ordinary High Water Mark (OHWM) Mean High Tide (MHT) Mean Low Water (MLW)
SECTION 404 Clean Water Act of 1977	Waters of the United States, including Wetlands	Fill (e.g., roads, home sites, beach renourishment)	OHWM High Tide Line (HTL) Vegetation lines
SECTION 103 Marine Protection, Research & Sanctuaries Act	Ocean	Transportation of dredged material for the purpose of ocean disposal	Mean Sea Level (MSL) MLLW NAVD88

[1] References are listed in Section 7-12.

(1) When the river banks are low and flat, OHWM is considered "the point up to which the presence and action of the water is so continuous as to destroy the value of the land for agricultural purposes by preventing growth of vegetation."

(2) On navigable lakes and rivers the U.S. Government holds an easement for riparian lands up to the OHWM. Thus, it is essential that the OHWM demarcation line be referenced to reliable vertical datums—to include the NSRS (NAVD88). In controlled pools, the relationship between the OHWM and pool discharge controlling elevations should be related to a common and consistent reference datum.

b. Open water. Open water is defined as an area that, during a year with normal patterns of precipitation, has standing or flowing water for sufficient duration to establish an OHWM. Aquatic vegetation within the area of standing or flowing water is either non-emergent, sparse, or absent. Vegetated shallows are considered to be open waters. The term 'open water' includes rivers, streams, lakes, and ponds.

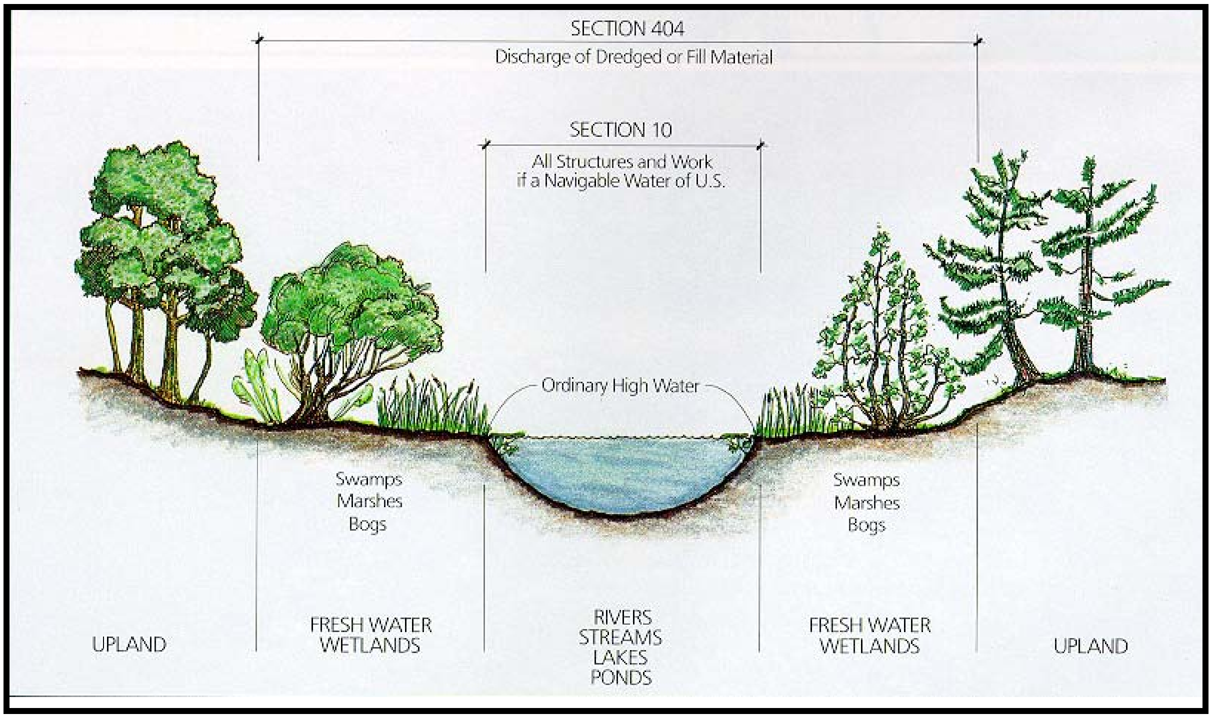


Figure 7-1. Regulatory datums for various permit authorities—inland rivers and lakes.

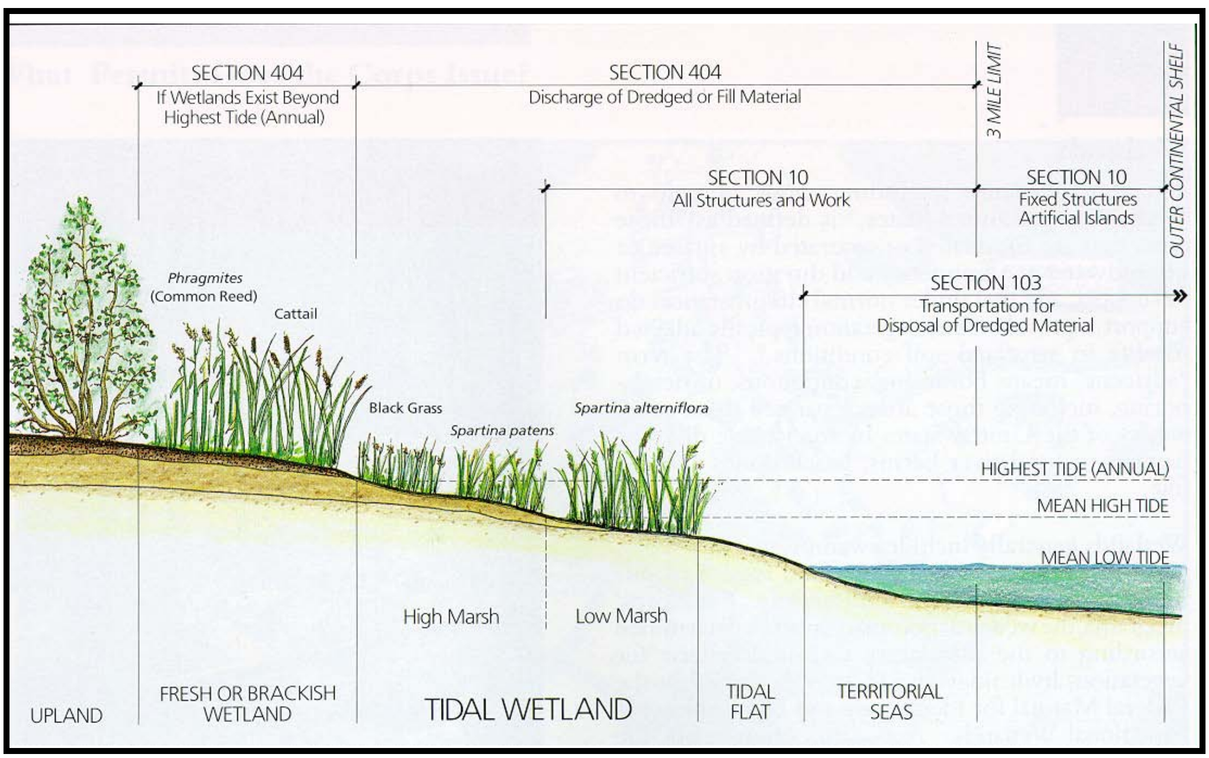


Figure 7-2. Regulatory datums for various permit authorities—tidal areas. In this figure "Highest Tide (Annual)" approximates HTL or MHWS, "Mean High Tide" approximates MHW, and "Mean Low Tide" approximates MLW.

c. Stream bed. A stream bed is defined as the substrate of the channel between the OHWMs. The substrate may be bedrock or inorganic particles that range in size form clay to boulders. Wetlands contiguous to the stream bed, but outside of the OHWMs, are not considered part of the stream bed.

d. Survey methods. Once OHWMs are delineated (staked out) in the field, various survey methods outlined in Chapter 3 may be used to control horizontal positions and elevations. RTK/RTN methods are usually the most efficient and provide adequate accuracy—typically less than ±0.2 ft relative to a nearby vertical reference PPCP—a gage or published NSRS bench mark. OHWM points will usually be marked with TBM hubs. Permanent marks (PBMs) will often be offset (set back) from the OHWM to higher ground, similar to erosion control lines or construction setback lines. Elevations will normally be referenced to NAVD88; however, legacy elevations or river/pool stages may be used. The horizontal location of OHWMs will normally be referenced to the local site plan (e.g., local boundary corners and structures). NAD83 coordinates may also be referenced on the site plan, either SPCS or geographical.

e. Example. Figure 7-3 is an example from a permit application that adequately references the proposed structure elevation to the local IGLD85 and NAVD88 datums at Lake Michigan. The relationship between the datums is clearly indicated on the drawing. Also shown is the established OHWM at the project site—in this case, differing federal and state OHWM determinations.

7-4. High Tide Line (HTL) and Related High Water Definitions. The high tide elevations depicted in Figure 7-2 are not consistently defined or measured throughout CONUS. High water tidal reference datums may be based on observed gage data, predicted tide models, or, in some cases, on visual estimates. Tidal datums based on reduced observations from federal or state gages are the most reliable and defensible. Predicted high tide levels based on NOAA models (TCARI, VDatum, etc.) would be next in reliability. Estimated tidal datum levels would be the most difficult to authenticate.

a. 33 CFR 328.3 definition of High Tide Line. The term "High Tide Line" (labeled as "Highest Tide Annual" in Figure 7-2) is defined in 33 CFR 328.3 as "... the line of intersection of the land with the water's surface at the maximum height reached by a rising tide. The HTL may be determined, in the absence of actual data, by a line of oil or scum along shore objects, a more or less continuous deposit of fine shell or debris on the foreshore or berm, other physical markings or characteristics, vegetation lines, tidal gages, or other suitable means that delineate the general height reached by a rising tide. The HTL encompasses spring high tides [MHWS] and other high tides that occur with periodic frequency but does not include storm surges in which there is a departure from the normal or predicted reach of the tide due to the piling up of water against a coast by strong winds such as those accompanying a hurricane or other intense storm."

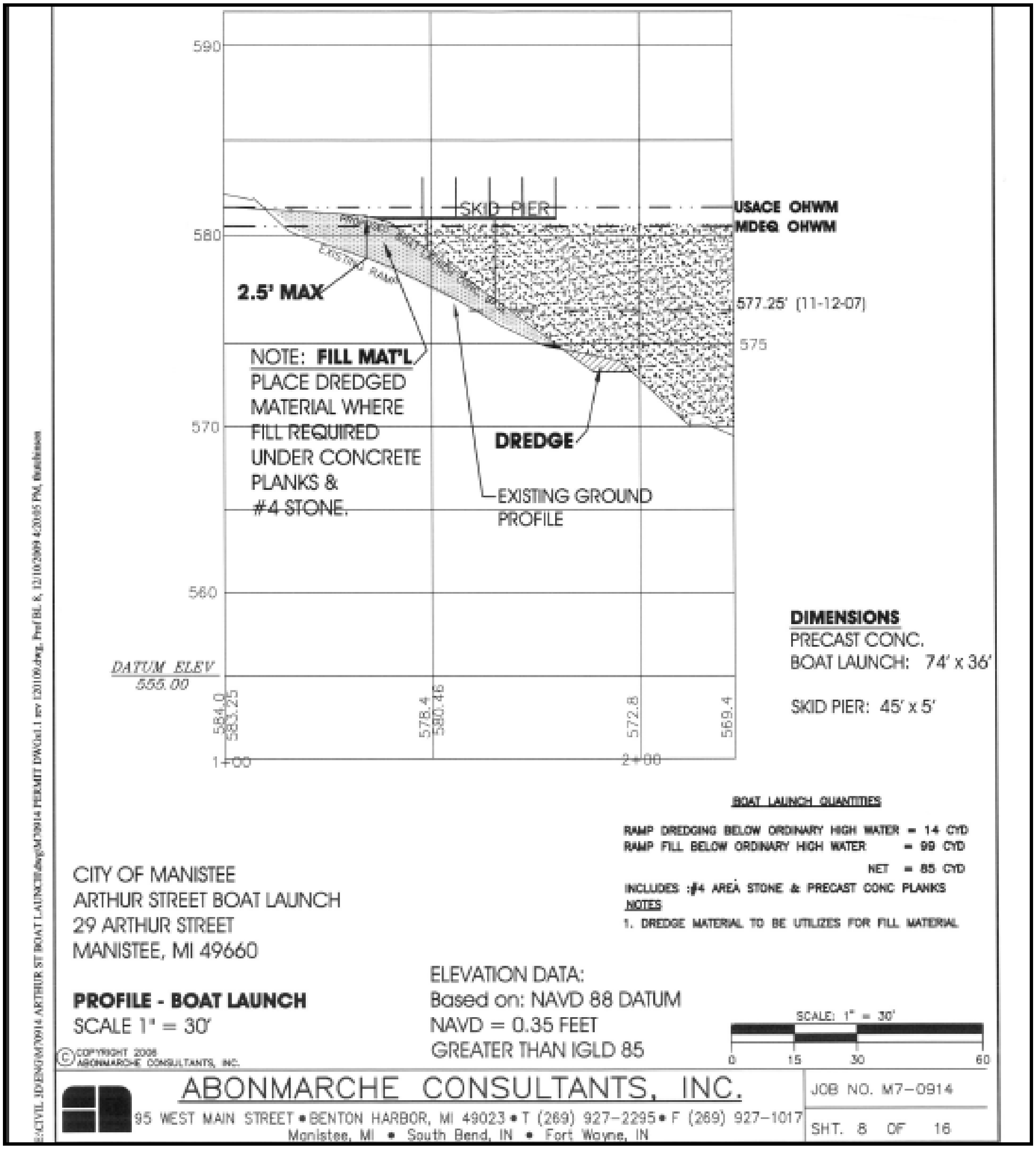


Figure 7-3. Federal (USACE) and State (Michigan Department of Environmental Quality) OHWM datums at Manistee, Michigan permit application site. (USACE OHWM—elevation 581.5 ft). Note that NAVD88 is 0.35 ft greater than IGLD85 (577.5 ft). (Detroit District)

(1) The preceding 33 CFR 328.3 definition implies High Tide Lines may be based on measured Spring Tides—i.e., Mean High Water Springs, or MHWS—at a local gage station.

33 CFR 328.3 notes "other high tides that occur with periodic frequency." These "other high tides" might be locally interpreted to include Mean High Water (MHW), Mean Higher High water (MHHW), or perhaps Highest Astronomical Tide (HAT). MHHW is below MHWS. HAT would represent the maximum tidal heights exclusive of storm surges. Local usage will determine which of these tidal datums should be used to define a permit limit on the shoreline. Regardless of the reference datum, it should be based on a firm mathematical computation from gage data, as illustrated for Spring High Water Tides (SHWT) below.

(2) It should be noted that MHW and MHHW datums are determined from all observed tides, including storm surge or other weather effects—the 33 CFR 328.3 definition excludes storm surge tides. By using mean values over a 19-year period, the effect of tides biased by weather effects (both extreme highs and extreme lows) are averaged out.

(3) Highest Astronomical Tide (HAT) is not based on observed gage data but is defined as the individual highest and lowest predicted tides over the NTDE 1983-2001 time period using the NOAA tide prediction procedures. NOAA predicted tides, including HAT, do not include daily and weekly weather effects in their elevations, but do include annual and semi-annual constituents that are driven by average seasonal changes in mean sea level.

b. Spring High Water Tides. Spring tides of increased range occur semimonthly as the result of the Moon being new or full. The spring range of tide is the average range occurring at the time of spring tides and is most conveniently computed from the harmonic constants. It is larger than the mean tide range (Mn) where the type of tide is either semi diurnal or mixed, and is of no practical significance where the type of tide is predominantly diurnal. The average height of the high waters of the spring tides is called Spring High Water Tides (SHWT) or Mean High Water Springs (MHWS), and the average height of the corresponding low waters is called Spring Low Water or Mean Low Water Springs (MLWS). Reference "*Tidal Datums and Their Applications*" (NOAA 2001).

(1) Historically, the international community, when it used Mean Low Water Springs (MLWS) as a chart datum, derives it by MLWS = Z0 - (M2 + S2), or the sum of the amplitudes of the semidiurnal (Z0) and solar M2 and S2 harmonic constituents below a mean value. Likewise, MHWS = Z0 + (M2 + S2). So high water spring tide datums are not based on tabulation of observations, but on harmonic analysis of observations.

(2) NOAA does not publish MHWS at its NWLON tide gage stations. However, they do publish the Spring Tide Ranges for selected prediction stations. The Spring Tide Range can be used to approximate the MHWS range by adding half the Spring Tide Range to the Mean Tide Level published for a gage station. This is only applicable for semi-diurnal tides—Spring Tide Ranges are not published for areas of diurnal tides (most of the Gulf of Mexico, etc.). In diurnal or mixed tide regions, the Highest Astronomical Tide (HAT) could be considered in lieu of MHWS because it can be determined the same way regardless of type of tide.

c. Mean High Water datum.. Mean High Water (MHW) is a reference for Section 10 boundaries— labeled as "Mean High Tide" in Figure 7-2. MHW datum is defined as the average height of all high waters at a tide station referenced to a 19-year period—see Chapter 2. MHW datum is always below HTL.

d. Non-tidal wetland. A non-tidal wetland is defined as a wetland (i.e., a water of the United States) that is not subject to the ebb and flow of tidal waters. Non-tidal wetlands contiguous to tidal waters are located landward of the high tide line (e.g., the Spring High Tide Line).

e. Tidal wetland. A tidal wetland is a wetland (i.e., a water of the United States) that is inundated by tidal waters—reference 33 CFR 328.3. Tidal waters end where the rise and fall of the water surface can no longer be practically measured in a predictable rhythm due to masking by other waters, wind, or other effects. Tidal wetlands are located channel ward of the high tide line (i.e., spring high tide line) and are inundated by tidal waters two times per lunar month, during spring high tides.

f. Tidal datum computations. Methods for determining tidal datums using water level gage comparison techniques are covered in state and federal publications. The primary source for most tidal datum computations is in the NOAA "*Computational Techniques for Tidal Datums Handbook Computational Techniques*" (NOAA 2003). This reference outlines methods for establishing tidal datums from gage observational data, including simultaneous comparison methods (e.g., Range-Ratio) used to transfer tidal datums from an established gage site to a remote project (permit) site. Typically, MHW datums are transferred using these simultaneous comparison methods. This NOAA manual does not cover establishment of an "Ordinary High Water Mark" (OHWM) since an OHWM demarcation is not always based on direct gage observations.

(1) The period of simultaneous gage comparisons is dependent on the distance from the site to the NWLON gage, the mean tide range, and local or individual state requirements. Simultaneous gage comparison periods of 3, 7, to 30 days are common in CONUS. Temporary staff gages may be used for short-term comparisons.

(2) Many coastal states have statutory and regulatory requirements for defining and observing new tidal datums at a project site. One example is in Florida Statutes, Chapter 177, Part II of the "*Florida Coastal Mapping Act of 1974*." Where these statues are applicable, surveyors portraying tidal elevations on permit drawings must have met the minimum technical standards prescribed in the statutes. For example, the Florida Department of Environmental Protection Chapter 177 statutes are relatively rigid regarding establishment of tidal datums, as extracted below. (Other coastal states have similar statutory or regulatory requirements.)

> *177.36 Work to be performed only by authorized personnel.--The establishment of local tidal datums and the determination of the location of the mean high-water line or the mean low-water line must be performed by qualified personnel licensed by the Board of Professional Surveyors and Mappers or by representatives of the United States Government when approved by the department.*
> *177.37 Notification to department.--Any surveyor undertaking to establish a local tidal datum and to determine the location of the mean high-water line or the mean low-water*

line shall submit a copy of the results thereof to the department [Department of Environmental Protection].within 90 days after the completion of such work, if the same is to be recorded or submitted to any court or agency of state or local government.

177.38 Standards for establishment of local tidal datums.--
(1) Unless otherwise allowed by this part or regulations promulgated hereunder, a local tidal datum shall be established from a series of tide observations taken at a tide station established in accordance with procedures approved by the department. In establishing such procedures, full consideration will be given to the national standards and procedures established by the National Ocean Service [NOAA CO-OPS].
(2) Records acquired at control tide stations, which are based on mean 19-year values, comprise the basic data from which tidal datums are determined.
(3) Observations at a tide station other than a control tide station shall be reduced to mean 19-year values through comparison with simultaneous observations at the appropriate control tide stations. The observations shall be made continuously and shall extend over such period as shall be provided for in departmental regulations.
(4) When a local tidal datum has been established, it shall be preserved by referring it to tidal bench marks in the manner prescribed by the department.

g. Survey procedures. Once a computed tidal datum at a site is established on a gage reference PBM, the MHW, MLW, or MHWS demarcation line can be staked out using total stations, differential levels, or RTK/RTN methods. On critical projects, the local gage PBMs should be set at stable locations for future reference and use in construction. Surveyed horizontal relationships between demarcation lines and PBMs to property corners are also normally required. CORS/OPUS techniques may be employed if a general NAD83 mapping reference is required—see Chapter 3.

7-5. <u>Boundary Uncertainties Due to Water Level Datum Errors</u>. The uncertainty in the value of a water level datum (e.g., MLW, MHW, OHWL, MTL) translates into a horizontal uncertainty of the location of a marine boundary when the datum line is surveyed to the land—reference "*Tidal Datums and Their Applications*" (NOAA 2001). Table 7-2 expresses the uncertainty in the marine boundary as a function of the slope (or grade) of the land. The greatest errors in the determination of a marine boundary occur for relatively flat terrain, which is characteristic of broad sections of the Atlantic and Gulf Coasts.

Table 7-2. Error in Position of Marine Boundary as a Function of the Slope of the Land given a 0.1 ft Vertical Datum Error. (Source: NOAA 2001)

Slope %	Degree of Slope (degrees)	Horizontal Error [1] (feet)
0.1	0.05	106
0.2	0.1	49
0.5	0.3	20
1.0	0.6	10
2.0	1	5
5.0	3	2
10.0	6	1
50.0	27	0.2
100.0	45	0.1

[1] error = 0.1 ft x cot (slope in degrees)

For example, a ±0.1 ft error in transferring a tidal datum from a gage to a project site on a 2% grade will equate to a ±5 ft horizontal error of the boundary demarcation line. This ± 0.1 ft "relative" uncertainty in a tidal datum does not include the regional (or global) uncertainty of the datum at the master gage. See Chapters 4 and 9 for discussions on the absolute and relative accuracies and uncertainties of tidal datums. If the tidal epoch at the project site has not been updated to the current epoch, then the error at this 2% grade site could be (+) 14 ft—14 ft landward given a 0.25 ft apparent sea level rise between epochs. The impact of these horizontal uncertainties (or biases) may or may not be significant, depending on the nature of a dredge or fill permit.

7-6. Section 10 Authority: Geographic and Jurisdictional Limits of Oceanic and Tidal Waters. Section 10 of the Rivers and Harbors Appropriations Act of 1899 (33 U.S.C. 403) requires approval prior to the accomplishment of any work or placement of any structure in navigable waters of the United States, or which affects the course, location, or condition of such waters with respect to navigable capacity. Typical activities requiring Section 10 permits include construction of piers, wharves, bulkheads, dolphins, marinas, ramps, floats intake structures, cable or pipeline crossings, and dredging and excavation. A Section 10 permit is required for all work, including structures, seaward of the "annual high water line" (e.g., MHW) in navigable waters of the United States, defined as waters subject to the ebb and flow of the tide, as well as a few of the major rivers used to transport interstate or foreign commerce. An example of a Section 10 project is shown in Figure 7-4.

a. Ocean and coastal waters. The navigable waters of the United States over which USACE regulatory jurisdiction extends include all ocean and coastal waters within a zone three geographic (nautical) miles seaward from the baseline (The Territorial Seas). Wider zones are recognized for special regulatory powers exercised over the outer continental shelf. (See 33 CFR 322.3(b)).

(1) Baseline defined. Generally, where the shore directly contacts the open sea, the line on the shore reached by the ordinary low tides (e.g., MLW) comprises the baseline from which the distance of three geographic miles is measured. The baseline has significance for both domestic and international law and is subject to precise definitions. Special problems arise when offshore rocks, islands, or other bodies exist, and the baseline may have to be drawn seaward of such bodies.

(2) Shoreward limit of jurisdiction. Regulatory jurisdiction in coastal areas extends to the line on the shore reached by the plane of the mean (average) high water—i.e., Mean High Water (MHW). Where precise determination of the actual location of the line becomes necessary, it must be established by survey with reference to the available tidal datum, preferably averaged over a period of 18.6 years. Less precise methods, such as observation of the "apparent shoreline" which is determined by reference to physical markings, lines of vegetation, or changes in type of vegetation, may be used only where an estimate is needed of the line reached by the MHW.

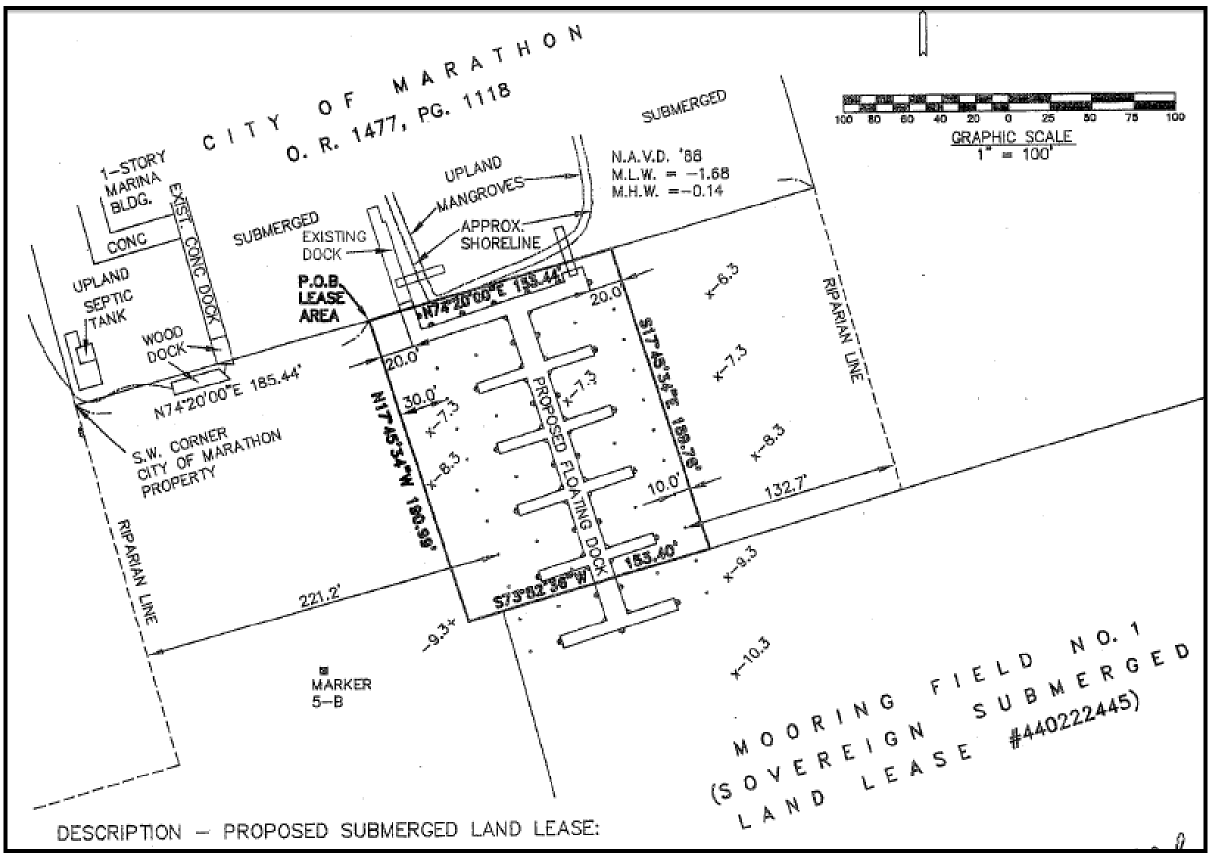


Figure 7-4. Example of Section 10 permit application with tidal datums and elevations referenced to NAVD88. (Jacksonville District)

b. Bays and estuaries. Regulatory jurisdiction extends to the entire surface and bed of all water bodies subject to tidal action. Jurisdiction thus extends to the edge of all such water bodies, even though portions of the water body may be extremely shallow, or obstructed by shoals, vegetation, or other barriers. Marshlands and similar areas are thus considered "navigable in law," but only so far as the area is subject to inundation by the mean high waters. The relevant test is therefore the presence of the mean high tidal waters, and not the general test described above, which generally applies to inland rivers and lakes.

c. Tidal datum computations. Methods for determining MHW tidal datums at remote sites using simultaneous comparison techniques were outlined in paragraph 7-4. Required tidal observation periods are highly site dependent—based on tide range and distance between the comparison gage and site.

7-7. Section 404 Authority: Limits of Jurisdiction—Dredged or Fill Material. Section 404 of the Clean Water Act of 1977 (33 U.S.C. 1344) regulates the discharge of dredged, excavated, or fill material in wetlands, streams, rivers, and other U.S. waters. Jurisdictional boundaries relative to tidal and non-tidal waters are defined below. Typical activities requiring Section 404 permits include depositing of fill or dredged material in waters of the U.S. or adjacent wetlands, site development fill for residential, commercial, or recreational developments (see Figure 7-5), construction of revetments, groins, breakwaters, levees, dams, dikes, and weirs, and placement of riprap and road fills.

a. Territorial Seas. The limit of jurisdiction in the territorial seas is measured from the baseline in a seaward direction a distance of three nautical miles. (Reference 33 CFR 329.12)

b. Tidal Waters of the United States. The landward limits of jurisdiction in tidal waters:

(1) Extends to the "high tide line" (HTL), or

(2) When adjacent non-tidal waters of the United States are present, the jurisdiction extends to the limits identified in paragraph (c) of this section.

c. Non-Tidal Waters of the United States. The limits of jurisdiction in non-tidal waters:

(1) In the absence of adjacent wetlands, the jurisdiction extends to the OHWM, or

(2) When adjacent wetlands are present, the jurisdiction extends beyond the OHWM to the limit of the adjacent wetlands.

(3) When the water of the United States consists only of wetlands, the jurisdiction extends to the limit of the wetland.

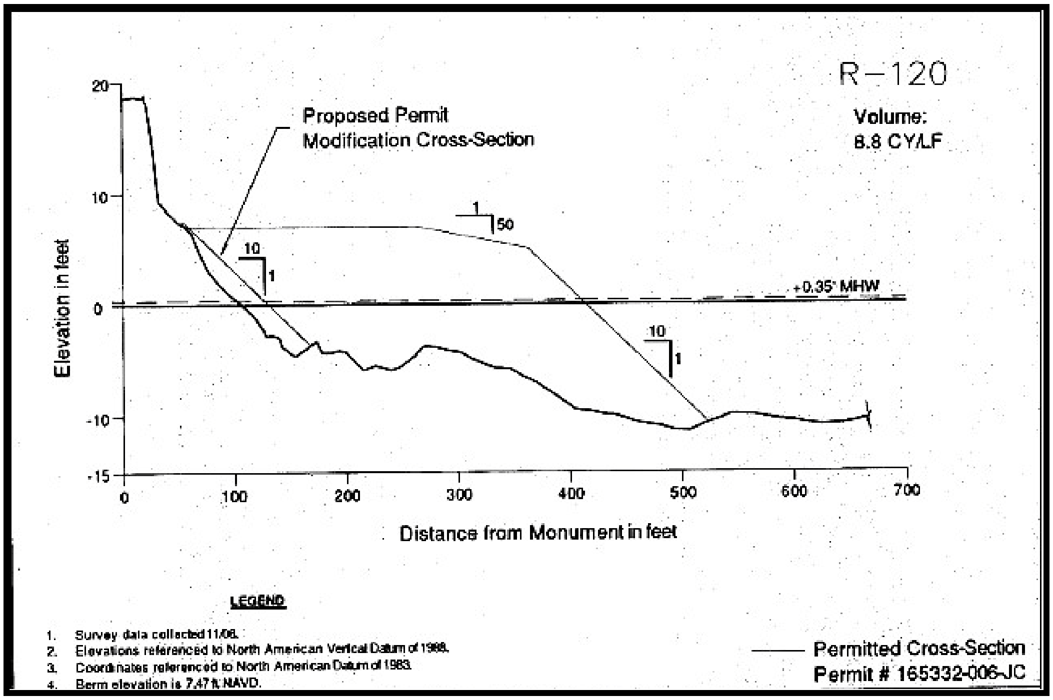


Figure 7-5. Sample Section 404 and Section 10 permit application—beach renourishment project. Cross-section and berm elevations are referenced to NAVD88. MHW relationship above NAVD88 is indicated. (The HTL is not indicated).

7-8. Section 103 Authority: Ocean Dumping of Dredged Material. Section 103 authority in the Marine Protection, Research and Sanctuaries Act (33 U.S.C. 1413), and related statutes, involves permits for ocean dumping in confined disposal sites. Vertical reference datums on these permits may be either orthometric or tidal. It is desirable that the datums be on the latest epochs and the orthometric-tidal relationship be specified. Section 103 permits can also involve a variety of site dependent parameters and restrictions. Those significant to geodetic datums involve dredge positioning/monitoring systems that record dredge/scow positions and changes in draft—e.g., the USACE "Silent Inspector" system. Currently many dredge control systems are positioned using RTK techniques. Thus, dredge draft and/or hopper drag arm elevations can be directly related to the orthometric, ellipsoidal, or tidal datum. Ocean disposal restrictions may involve both horizontal and vertical height restrictions, in addition to various turbidity and biologic criteria. Deep ocean sites (i.e., > 100 ft) may require periodic monitoring surveys to check for material dispersion.

7-9. Marine Boundaries in Coastal Areas Defined by Tidal Datums. The following material in this section is excerpted from NOAA 2001. It provides an overview of the marine boundaries defined by tidal datums in the various states.

a. General. Chart datum, MLLW, is the elevation of the baseline for many marine boundaries, including most which are recognized by the United Nations Convention on the Law of the Sea. However, baselines may differ in position for the purposes of different statutes. The baselines (see Figure 7-6) usually consist of points or line segments on these tidal datum lines from which the marine boundaries are measured and constructed.

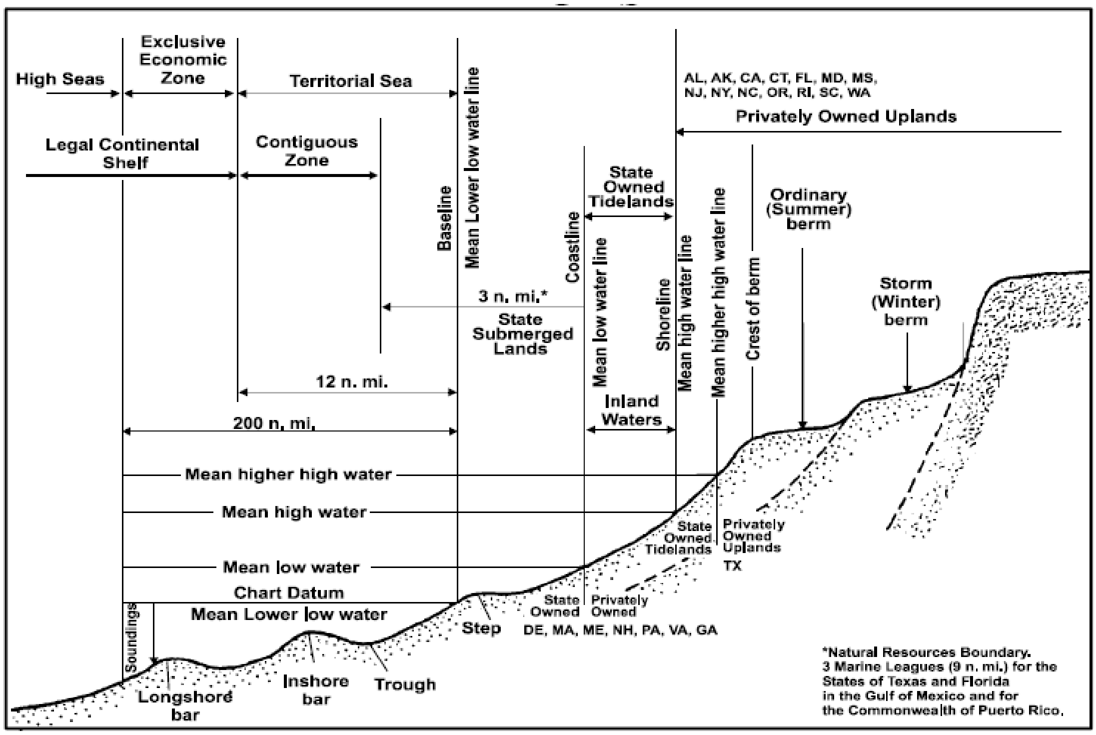


Figure 7-6. The principal tidal datums related to a beach profile. The intersection of the tidal datum with land determines the landward edge of a marine boundary.

b. Marine boundaries. As delineated in Figure 7-6, the marine boundaries of the U.S. are:

(1) Private U.S. property exists in most cases landward of MHW.

(2) State-owned tidelands exist between MHW and MLW in most cases. Refer to Figure 7-6 for individual cases. U.S. Inland Waters are concurrently defined to exist between MHW and MLW for the purpose of marine navigation.

(3) A state's "Submerged Lands Boundary" extends seaward three nautical miles from MLW, except for Texas and the Gulf coast of Florida where it terminates at nine nautical miles. In this band, plus the state-owned tidelands, the states exercise the "Public Trust Doctrine," subject to federal supremacy.

(4) The "Territorial Sea Boundary" extends 12 nautical miles seaward of MLLW. It is also known as the Marginal Sea, Marine Belt, Maritime Belt, 12-Mile Limit, and Adjacent Sea Boundary. Historically, this boundary was three nautical miles; it was changed to its present 12-mile limit in 1988. In the Territorial Sea, the sovereignty of the nation extends to the airspace above, the subsoil, the water, and the resources.

(5) The "Contiguous Zone Boundary" occurs at 12 nautical miles from MLLW. In the U.S., the Territorial Sea and Contiguous Zone are coterminous. In the contiguous zone, the nation may exercise rights to protect its interests, but does not exert sovereign control. The main purpose of the contiguous zone is to exert control over shipping near a nation's coast. Under the United Nations Convention on the Law of the Sea, a coastal nation may declare a Contiguous Zone between 12 and 24 nautical miles.

(6) The 200-mile "Fishery Conservation Zone" extends seaward from MLLW.

(7) The Presidential Proclamation 5030 of March 1983, established the "Exclusive Economic Zone" (EEZ), which claimed rights to living and mineral resources and jurisdiction of approximately 3.9 billion acres. The baseline for demarcation of the EEZ is the MLLW boundary of the Territorial Sea and extends 200 nautical miles. It should be noted that different coastal nations have different definitions of their ordinary low water. These definitions are not usually consistent with NOS definitions.

c. Mean High Water Line. The Mean High Water Line (MHWL) is the coastal boundary between private and state property with the following exceptions:

(1) Maine, New Hampshire, Massachusetts, Pennsylvania, Delaware, Virginia, and Georgia use the Mean Low Water Line (MLWL).

(2) Texas uses the Mean Higher High Water Line (MHHWL) when Spanish or Mexican grants are involved.

(3) Louisiana has adopted the civil law boundary of the line of highest winter tide.

(4) In Hawaii, the upland owner has title to the upper reaches of the wash of the waves.

d. Demarcation of MHWL. In order to map tidal boundaries such as MHWL or MLWL, and determine the latitude and longitude coordinates of their intersection with the coast, the surveyor performs the following basic procedures:

(1) Obtain the published bench mark information at or near the location.

(2) Find the tidal bench marks and run a closed line or loop of differential levels from the bench marks to that part of the shore where the boundary is to be located, run levels along the shoreline, and mark or stake points at intervals along the shore in such a manner that the ground at each point is at the elevation of the tidal datum.

(3) If the boundary is to be mapped, the horizontal distances and directions, or bearings, between each of these points and between those points or features in the area, and between the points and the horizontal control stations are measured so that the boundary may be plotted on a plat or map to the exact scale ratio and in true relation to other boundaries.

7-10. Permit Application Checklist. Table 7-3 outlines some reference datum issues that may warrant review on a permit application.

Table 7-3. Permit Application Checklist for Issues Relating to Geospatial Datums.

Reference Bench Mark for Project Site Surveys
- Datum noted
- Elevation of bench mark noted
- NSRS PID noted, if applicable
- If legacy reference datum (e.g., NGVD29) relationship to NAVD88 identified
- If pool/reservoir/river stage, relationship to NAVD88 identified

Topographic Surveys
- Reference datum identified
- Reference bench mark identified
- Horizontal reference datum identified
- Quantity take off metadata/source noted
- Survey date & source metadata noted

Boundary Survey
- Survey conforms to state minimum technical standards, as applicable

Tidal Datum Transfers
- Primary reference gage identified
- Tidal epoch noted
- Local site gage PBM noted
- Datum transfer to project site meets state minimum technical standards, as applicable
- Datum transfer computation metadata identified

7-11. Example: Sections 10 and 404 Permit Application Involving Tidal Limits. The following example is excerpted from a permit application submitted to the Jacksonville District. The permit involved a bulkhead relocation and fill in Florida tidal waters. This permit is typical of applications for fill in navigable waters or wetlands below the high tide line or MHW line. Some aspects in this example are simulated since this particular permit application did not detail the specific procedures used in transferring tidal datums from an established NOAA NWLON gage.

a. Background. The following general description of the project is excerpted from the permit application and subsequent District technical reviews and findings.

> *"Project Description: The applicant proposes to construct a bulkhead 9 to 27 feet water ward in front of an existing bulkhead and fill approximately 1,660 square feet of waters of the United States with approximately 187.5 cubic yards of fill material to extend the yard.*

Statutory authority: Section 10 of the Rivers and Harbors Act of 1899 and Section 404 of the Clean Water Act of 1972, as amended.

The existing project area consists of open water and inundated floodplain classified as estuarine, subtidal, unconsolidated bottom. The on-site vegetation consists of water oak (Quercus nigra), arrowhead (Sagittaria lancifolia), wild taro (Colocasia esculenta) and other emergent vegetation. The onsite vegetative communities were classified according to the Florida Department of Transportation's Florida Land Use, Cover and Forms Classification System.

The waters of the United States (wetlands) at the site consist of the tidal floodplain of Doctors Lake, a navigable water of the United States. Doctors Lake is an elongated 3,500 acre embayment situated on the west side of the St. Johns River. It is situated in the tidal, brackish part of the Lower St. Johns River Basin (LSJRB). The lake is about five (5) miles long by one (1) mile wide, connected to the St. Johns River by an approximately 0.25 mile wide opening at its northeast end. Doctors Lake has no freshwater tributaries, making tidal exchange with the St. Johns River the lake's largest source of water ... Tidal currents play an important role in determining estuarine water quality. Tidal currents that flow into Doctors Lake on flood tide and out during ebb tide dilute and transport pollutants over each tidal cycle and thereby flush the system. Tidal circulation provides the dominant flushing mechanism in the lake. The tidal prism or the volume of water exchanged during each half tidal cycle plays an important role on tidal circulation. Reduction of intertidal shoreline can reduce the total prism of water exchanged."

Figure 7-7 depicts the project location relative to the St. Johns River, which flows into the Atlantic Ocean some 30 miles downstream of the permit site. An historic NOAA tide gage station is located directly across the lake from the permit site. Tidal bench marks at this gage site have been connected to the NSRS and have reliable NAVD88 adjusted elevations.

b. Permit site plan details. Figure 7-8 (excerpted from the original permit application) shows the plan layout of the existing bulkhead and proposed extended bulkhead into tidal waters. The "MHW" contour in depicted in plan, along with Section A-A. The section A-A view notes the elevations of MHW and HTL. The source of the elevations was noted as a tidal PBM at NOAA tide gage 872 0406 (DOCTORS LAKE, PEORIA POINT). Elevations were referenced to NGVD29. NAVD88 control is readily available—a previous permit may have been referenced to NGVD29. There was no indication of the source of the HTL elevation (1.71 ft above NGVD29). No metadata was included in the permit detailing the source of the Mean High Water contour depicted in the plan view. It is presumed this underwater contour was derived from a topographic/boundary survey of the site performed prior to the permit application.

Figure 7-7. Permit site and reference NOAA gage locations in tidal inlet.

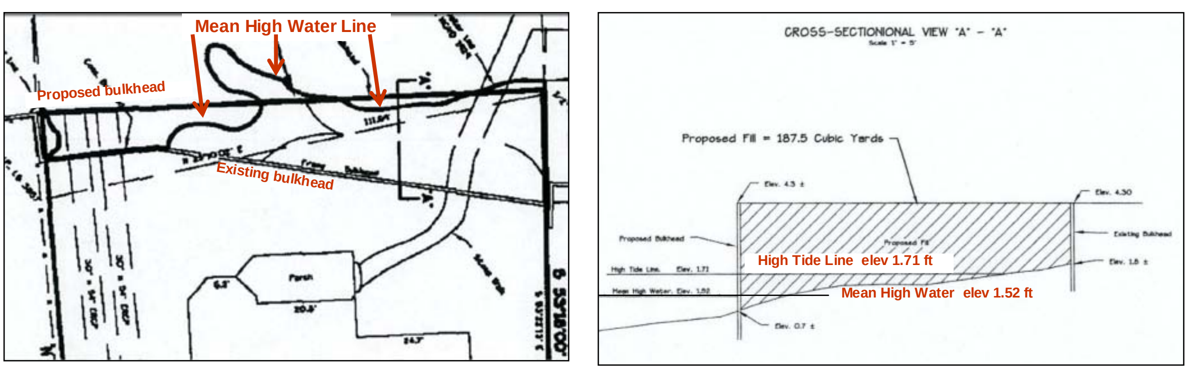


Figure 7-8. Permit site: plan and section views.

Figure 7-9. Apparent surface elevations above surveyed MHW and HTL elevations.

c. NOAA datasheet for gage station 872 0406. The data in Figure 7-10 is taken from the NOAA tide gage directly across the lake from the project site. This data would be used to compare or transfer tidal datums from the gage site to the permit site, as outlined below.

```
 Tidal datums at DOCTORS LAKE, PEORIA POINT based on:

     LENGTH OF SERIES:      5 MONTHS
     TIME PERIOD:           June 1978 - October 1978
     TIDAL EPOCH:           1983-2001
     CONTROL TIDE STATION:  8720496 GREEN COVE SPRINGS, ST. JOHNS R.

Elevations of tidal datums referred to Mean Lower Low Water (MLLW), in METERS:

     MEAN HIGHER HIGH WATER (MHHW)                =  0.278 [0.91 ft]
     MEAN HIGH WATER (MHW)                        =  0.257 [0.84 ft]
     MEAN TIDE LEVEL (MTL)                        =  0.136 [0.45 ft]
     MEAN SEA LEVEL (MSL)                         =  0.130
     NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)    =  0.117 [0.38 ft]
     MEAN LOW WATER (MLW)                         =  0.014
     MEAN LOWER LOW WATER (MLLW)                  =  0.000

National Geodetic Vertical Datum (NGVD 29)
Bench Mark Elevation Information           In METERS above:

     Stamping or Designation               MLLW        MHW

     0406 C 1978                             3.878    3.621
     0406 A 1978                 [10.12 ft]  3.084    2.827
     0406 D 1978                             6.638    6.381
```

Figure 7-10. NOAA datasheet for tide gage 872 0496 (Doctors Lake, Peoria Point).

d. NGS datasheet for Tidal Bench Mark 872 0406A. Figure 7-11 is excerpted directly from the NGS datasheet for PBM "872 0406A." This datasheet provides reference NAVD88 relationships at the tide gage from which differential leveling or RTK surveys can be performed. This sheet also provides the relationship between NAVD88 and NGVD29, which can be transferred to the project site.

```
BC1437  TIDAL BM     -  This is a Tidal Bench Mark.
BC1437  DESIGNATION -  872 0406 A
BC1437  PID          -  BC1437
BC1437  STATE/COUNTY-  FL/CLAY
BC1437  USGS QUAD    -  MIDDLEBURG (1993)
BC1437                          *CURRENT SURVEY CONTROL
BC1437  ___________________________________________________________________
BC1437* NAD 83(1986)-  30 07 11.     (N)    081 45 33.     (W)     SCALED
BC1437* NAVD 88      -          2.973  (meters)       9.75   (feet)  ADJUSTED
BC1437  ___________________________________________________________________
BC1437  GEOID HEIGHT-          -28.13  (meters)                      GEOID09
BC1437  DYNAMIC HT   -            2.969 (meters)       9.74  (feet)  COMP
BC1437  MODELED GRAV-      979,343.2   (mgal)                        NAVD 88
BC1437  VERT ORDER   -  FIRST     CLASS II
BC1437                           SUPERSEDED SURVEY CONTROL
BC1437
BC1437  NAVD 88 (06/15/91)    2.964  (m)         9.72   (f) UNKNOWN     1 2
BC1437  NGVD 29 (09/01/92)    3.289  (m)        10.79   (f) ADJUSTED    1 2
BC1437
BC1437.Superseded values are not recommended for survey control.
BC1437.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
BC1437.See file dsdata.txt to determine how the superseded data were derived.
```

Figure 7-11. NGS datasheet for tidal bench mark 872 0406A.

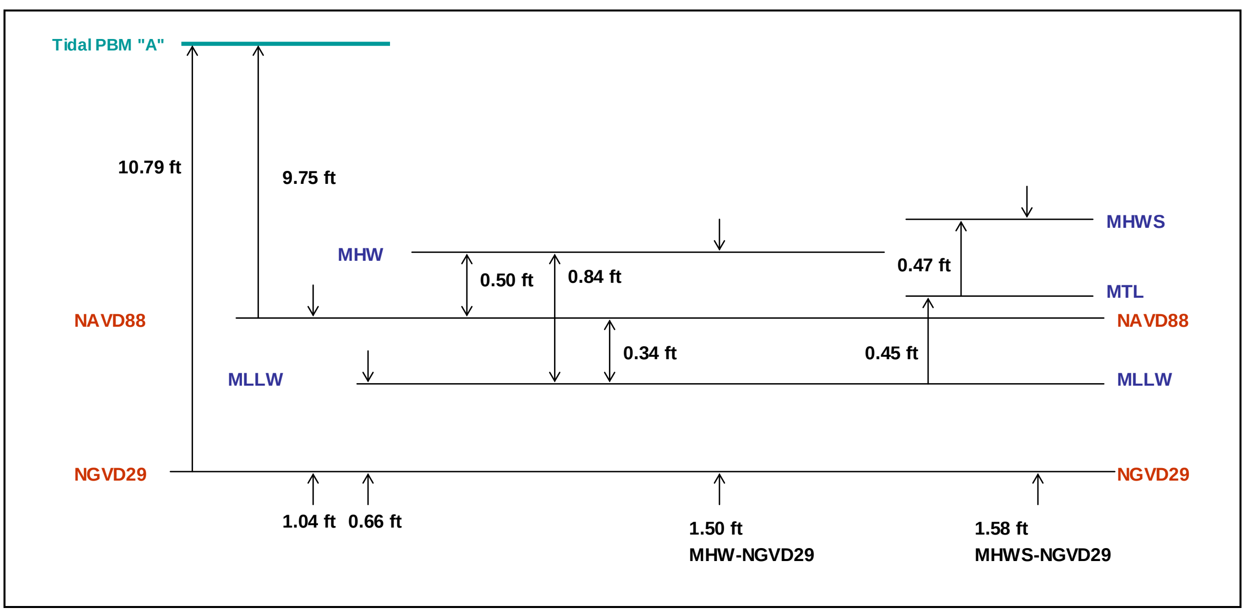


Figure 7-12. Datum relationships at the NOAA DOCTORS LAKE, PEORIA POINT gage.

e. Determining tidal datum relationships to orthometric datums. Using data from the NOAA CO-OPS gage site and the NGS tidal bench mark, the tidal reference datums at the permit site can be referenced to an orthometric datum (NGVD29 or NAVD88) as indicated in Figure 7-12 and the following computations.

(1) MHW elevation relative to NGVD29.

Tidal PBM "A" elevation	10.79 ft above NGVD29 (from NGS Datasheet)
Tidal PBM "A" elevation	9.75 ft above NAVD88 (from NGS Datasheet)

NAVD88 - NGVD29:	(+)	1.04 ft (concluded from above)
NAVD88 - MLLW:	(-)	0.38 ft (from CO-OPS Datasheet)
MHW - MLLW:	(+)	0.84 ft (from CO-OPS Datasheet)
MHW above NGVD29:		1.50 ft

Similarly, MHW datum can be related to NAVD88:

MHW - MLLW:	(+)	0.84 ft (from CO-OPS Datasheet)
NAVD88 - MLLW:	(-)	0.38 ft (from CO-OPS Datasheet)
MHW above NAVD88:		0.46 ft

(2) High Tide Line (HTL) determination. As stated earlier, the permit application did not indicate the source of the HTL elevation (1.71 ft above NGVD29) shown on the section view. Since the HTL is approximately related to MHWS datum, NOAA station prediction data may be used to estimate the MHWS elevation. The following data in Figure 7-13 is taken from the NOAA tidal predictions for this gage site:

FLORIDA, St. Johns River Predictions Station	Latitude	Longitude	Mean Range (ft)	Spring Range (ft)	Mean Tide Level (ft)
Peoria Point, Doctors Lake	30° 07.2'	81° 45.5'	0.80	0.93	0.45

Figure 7-13. NOAA tidal predictions for Doctors Lake.

(3) Estimated High Tide Line based on NOAA published Spring Tide Range relative to Mean Tide level published on the CO-OPS station Datasheet.

One-half of 0.93 ft Spring Tide Range above Mean Tide Level:	(+)	0.47 ft	(from NOAA Station Predictions)
Mean Tide Level above MLLW:	(+)	0.45 ft	(from CO-OPS Datasheet)
MLLW above NGVD29:	(+)	0.66 ft	
Mean High Water Spring above NGVD29:		1.58 ft	(slightly above 1.57 ft MHHW) (the diurnal range is 0.91 ft)
(or MHWS is 0.58 ft above NAVD88)			

(4) An elevation difference of 0.13 ft exists between the permit HTL (1.71 ft) and the HTL computed from NOAA MHWS estimate (1.58 ft). If this elevation disparity were deemed significant at a project site, then NOAA CO-OPS would need to be contacted to obtain a MHWS computation from the original gage observations—in this case, from the 1978 series.

f. Transferring tidal datum elevations to a permit site. Three options would exist to transfer tidal datums from a remote gage to the project site—the permit site in this example.

(1) Perform simultaneous tide gage comparisons between NOAA gage and a temporary gage at the permit site. Given the relatively short distance (1 mile) between the gage site and the permit site, an accurate "water level transfer" of datums between the sites could be easily accomplished. In this jurisdiction, State of Florida approval for a gaging transfer would be required. Three-day staff readings would likely suffice for the datum transfer, given the short distance. For more remote project sites that are distant from the reference gage, longer comparison tide readings may be required.

(2) Run differential levels from the NOAA gage site to the permit site. In this example, a six to seven mile level line would be required around the lake to connect the sites—a 12 to 14 mile level line loop. Maintaining 0.1 ft loop closures over this distance would be problematic; thus, differential leveling over this distance may not be as effective as water transfer or GPS methods.

(3) Perform differential GPS surveys to transfer elevations from the tidal PBMs to the permit site. A static GPS baseline would effectively and accurately transfer tidal datums over this short 1-mile distance. Given the short distance, rapid/fast-static methods would also suffice. Longer baseline lengths would require full static GPS baseline observations.

On this particular site, option (3) above would likely represent the most cost-effective method for transferring tidal datums to the permit site. This transfer could be performed in less than 4 hours time. Leveling would take at minimum 2 days and water level transfer 3 days. For more distant project sites from the reference gage, option (1) would represent the preferred choice, given use of hydraulic comparisons as opposed to geodetic comparisons. Option (2) is only effective over short distances.

7-12. References.

33 U.S.C. 403
Rivers and Harbors Act of 1899 (Section 10)

33 U.S.C. 1344
Clean Water Act of 1972, as amended, (Section 404)

33 U.S.C. 1413
Marine Protection, Research, and Sanctuaries Act of 1972 (Section 103)

33 CFR 328
Navigation and Navigable Waters, Definition of Waters in the United States

CHAPTER 8

Monitoring Flood Protection Elevation Grades in High Subsidence Areas

8-1. Purpose. This chapter provides technical guidance for referencing project elevation grades in areas subject to relative sea level change, land subsidence, or crustal uplift. Sea level rise, coupled with subsidence, reduces protection elevations on HSPP structures; while at the same time increasing the depths of authorized/maintained navigation project grades, resulting in over-dredging. The reverse effects occur in crustal uplift regions where apparent mean sea level is falling. (Subsequent references to "subsidence" in this chapter are intended to apply to "uplift" regions as well).

a. Much of the information in this chapter is abstracted from Volume II of the "Interagency Performance Evaluation Taskforce" report (IPET 2007) that was published following Hurricane Katrina in August 2005. Volume II of this IPET study, performed jointly by USACE, NGS, and CO-OPS, focused on the development and application of a high-accuracy, time-dependent geodetic network in a high subsidence area in Southern Louisiana. This Southern Louisiana example is, therefore, applicable to other USACE project areas experiencing subsidence issues.

b. Districts involved with projects subject to significant subsidence uncertainties should closely coordinate with the NGS to ensure a suitable vertical reference framework is established to monitor elevation changes. This effort can be accomplished following NGS "height modernization" standards and specifications. In some cases, time dependent vertical networks outlined in this chapter can be established to periodically update control in unstable regions. In coastal regions, additional coordination with NOAA CO-OPS is recommended to monitor sea level datum changes in the project region. Reference marks at NOAA or local tide gages must be directly linked to NGS vertical networks in order to monitor sea level changes relative to orthometric/geodetic datums.

8-2. Background. Published elevations relative to the vertical datums in subsidence or uplift areas must be used with caution. This applies not only to NSRS or local district PBMs but also to topographic survey data derived from these bench marks (e.g., floodwall and levee protection elevations). Surveyed or published map elevations will have uncertainties due to the uneven temporal and spatial movement of the land. Thus, any geodetic or terrestrial-based elevation on a bench mark or control structure is not constant, and elevations must be periodically reobserved and adjusted for local subsidence. Likewise, hydraulic or sea level based reference datums are subject to variations due to subsidence and/or sea level change at each gage site. Sea level datums also have time varying astronomical components making their reference definition more complex than terrestrial based datums. Hydraulic low water reference datums used to define navigation grades on the Lower Mississippi River may also be subject to subsidence and other long-term variations: thus, these datums are spatially and temporally variable, and are periodically revised.

a. Long-term primary bench mark subsidence. Figure 8-1 illustrates the varying orthometric and sea level elevations recorded at a primary control bench mark at Lake Pontchartrain in New Orleans, LA. In this high subsidence area, the published orthometric elevation of this bench mark has been readjusted numerous times over the 50+ year period—with an elevation change of over 2 ft since 1951. These readjustments include datum conversions from NGVD29 to NAVD88. Evaluating settlement rates, masked with sea level rise, from legacy elevations on this bench mark is not straightforward in that leveling networks were adjusted between other unstable marks. If a tide gage had been continuously operated at this site over the 50-year period, a better estimate of relative settlement could have been made. Levee or floodwall design elevations in a high subsidence area must factor in the elevation datum uncertainties and subsidence rates at the primary bench mark.

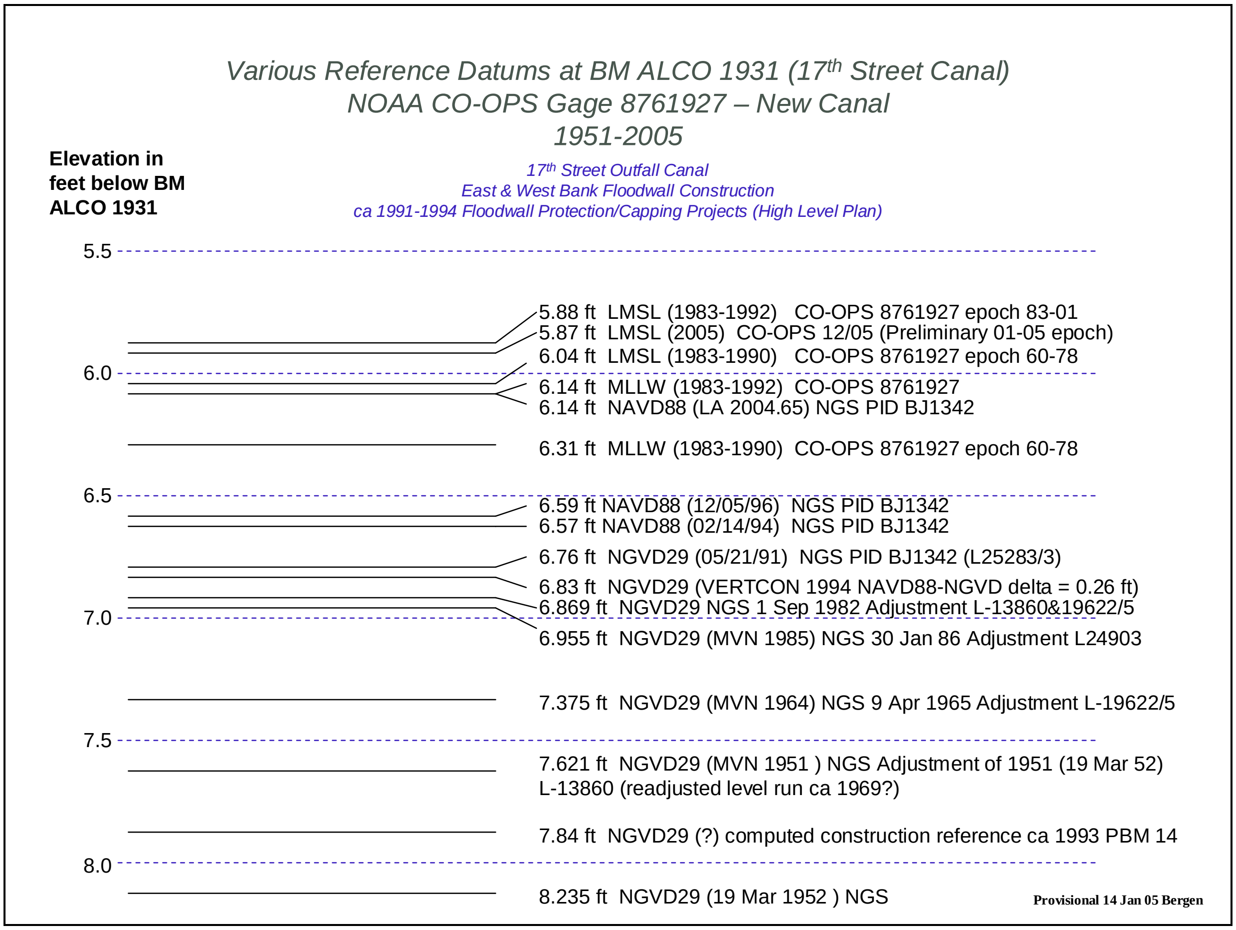


Figure 8-1. Elevations recorded at NSRS bench mark ALCO in New Orleans: 1951 to 2005.

b. Relative mean sea level. Relative or local mean sea level is the average water surface as measured by a tide gage with respect to the land upon which it is situated. Relative sea level change occurs where there is a local change in the level of the ocean relative to the land, which might be due to ocean rise and/or land level subsidence. In areas subject to rapid land level uplift, relative sea level can fall. These sea level changes result from a variety of processes, several of which can occur simultaneously. Part II of EM 1110-2-1000 (*Coastal Engineering*

Manual) lists the following processes that can contribute to long-term relative mean sea level change:

(1) Eustatic rise. Refers to a global change in the oceanic water level. Contributors to eustatic rise include melting of land-based glaciers and the expansion of near-surface ocean water due to global ocean warming.

(2) Crustal subsidence or uplift from tectonic uplifting or downwarping of the earth's crust. These changes can result from uplifting or cooling of coastal belts, sediment loading and consolidation, subsidence due to volcanic eruption loading, or glacial rebound.

(3) Seismic subsidence. Caused by sudden and irregular incidence of earthquakes.

(4) Auto-subsidence. Due to compaction or consolidation of soft underlying sediments such as mud or peat.

(5) Climatic fluctuations. May also create changes in sea level; for example, surface changes produced by El Niño due to changes in the size and location of high-pressure cells.

c. Subsidence. The subsidence effects listed above are the major contributor to elevation changes in most unstable regions. It is especially pronounced in portions of Central California (Sacramento and San Joaquin River Basins), Southwestern California, and coastal portions of Texas and Louisiana. Uplift, or apparent sea level rise, is evident in Northwest CONUS and portions of Alaska. In Southern Louisiana, subsidence is occurring at a rate of up to 0.1 foot every three years in some areas. There are many potential factors that contribute to subsidence, such as the geologic composition of the area and withdrawal of ground water and oil.

d. Subsidence at reference bench marks. Bench marks set on deep-driven rods to "apparent refusal" or bedrock will often exhibit relative subsidence to the local land surface, as shown in Figure 8-2. This relative chance is not necessarily a definitive measure of local settlement as the bench mark’s refusal point may also have settled at a differing rate. Thus, the difference in elevation between the deep-driven bench mark and the TBM in the figure may or may not represent the local subsidence. The apparent subsidence of the mark on the lower left figure may be due more to local levee settlement as opposed to subsidence. The Shell Beach tidal bench mark in the figure was set in 1982 on then dry land, illustrating the rapid subsidence (or apparent sea level rise) that has occurred over the intervening 23 years in this region.

Figure 8-2. Local subsidence relative to deep-driven bench marks.

e. Monitoring elevation changes in subsidence areas with "Vertical Time Dependent Positioning" (VTDP) geodetic surveys. Changes in elevation caused by subsidence can be measured and/or periodically monitored using a combination of conventional leveling procedures and GPS techniques. Determining subsidence rates requires long-term observations and considerable analysis.

(1) Prior to the use of GPS observations, subsidence estimates were made using regional leveling networks. Presumed "most stable" bench marks were held fixed for different leveling campaigns and settlement estimates were made based on the elevation changes in these campaigns. The ability to measure accurate relative elevation differences to < 0.1 ft with GPS over long baselines (>100 miles) now provides a mechanism to connect unstable areas with reference bench marks in known stable regions

(2) As an example, Figure 8-3 shows the difference in elevations in southern Louisiana due to the differences between NGVD29 and NAVD88 adjustments along with regional subsidence. The leveling for this line was performed by NOAA in 1984 and adjusted to the NGVD29 datum at that time. In 1991, NGS adjusted the entire CONUS to the NGVD29 datum in preparation for the NAVD88 adjustment. In Southern Louisiana, an extensive “GPS Derived Height” network was completed in 2004, establishing new heights (elevations) for 85 bench marks in Southern Louisiana. The differences are shown in Figure 8-4. This "Vertical Time Dependent Positioning (VTDP)" adjustment, known as NAVD88 (2004.65), held control outside of the subsidence area to establish new NAVD88 adjusted heights for the 85 bench marks.

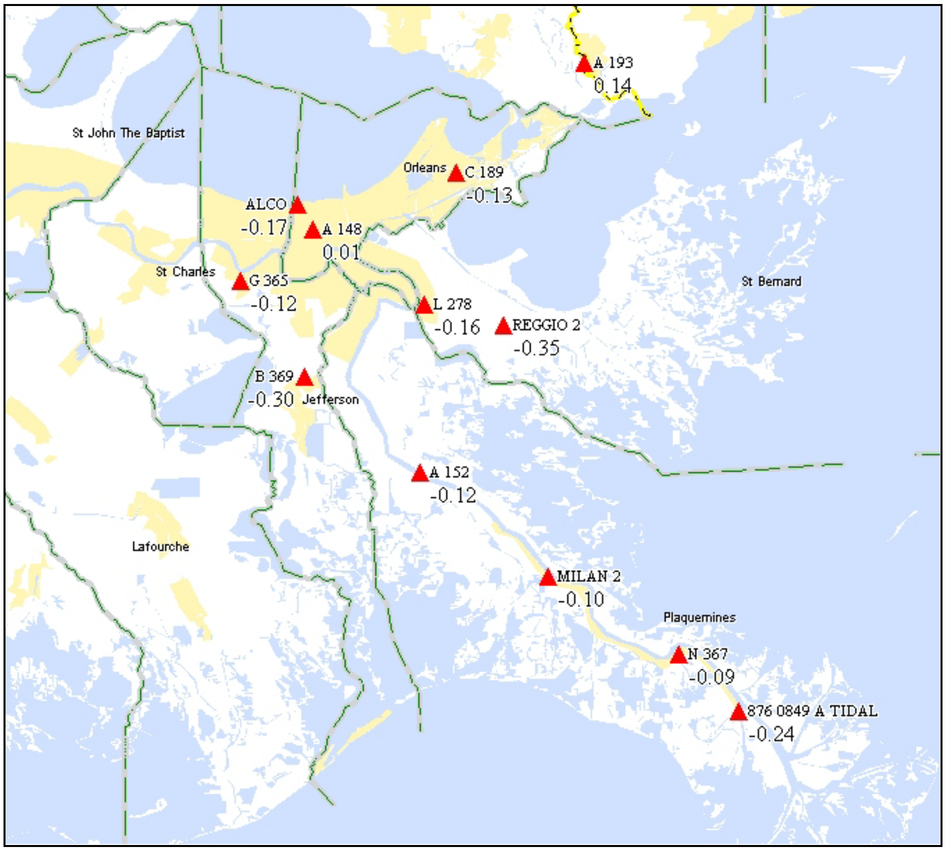


Figure 8-3. Elevation changes (ft) due to datum shift (NGVD29 to NAVD88) and regional readjustment.

(3) Because the 1991 NAVD88 adjustment held control outside of the area, as did the NAVD88 (2004.65) adjustment, the change in the heights reflects the apparent movement of the marks between the observation periods. In order to determine the amount of subsidence from the time the original leveling was done, it is necessary to determine the amount of movement between the original adjustment and the 1991 national readjustment of the NGVD29 and then the amount of movement between the original NAVD88 adjustment and the NAVD88 (2004.65) adjustment.

f. Monitoring elevation changes in subsidence areas with water level gages. Monitoring subsidence or sea level changes on flood risk management, hurricane protection, or coastal (tidal) navigation projects requires continuous leveling or GPS surveys between water level recording gages and fixed NSRS bench marks. Geodetic surveys alone cannot determine sea level changes. Records from these gages, if reasonably well documented, can provide an independent means to investigate and determine reliable rates of local subsidence and/or validate rates determined via a VTDP geodetic survey analysis. Reference PBMs at these gages must be connected to an external geodetic network that is not impacted by subsidence. A New Orleans District study on the use of gage data in evaluating subsidence changes in Southern Louisiana is at Appendix L.

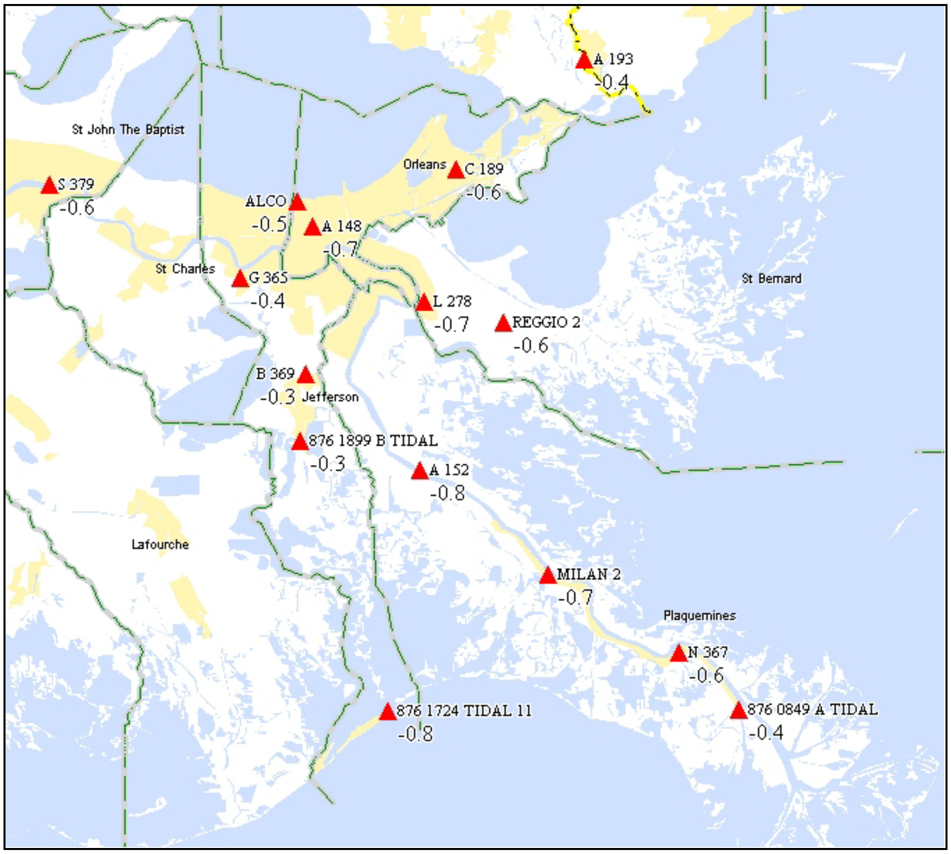


Figure 8-4. Elevation changes (ft) between NAVD88 (1996) adjustment and the VTDP NAVD88 (2004.65) regional readjustment.

g. Applications. EM 1110-2-1100 (*Coastal Engineering Manual*) notes that long-term subsidence must be factored in to the design of flood/hurricane protection structures. It states that "...the [reference] datums described above, and the reported variability of those datums, represent design criteria considerations that directly impact the expected lifetime of a project. If, for example, a coastal project is to be situated in an area of known subsidence, then design elevations need to reflect additional freeboard as a factor-of-safety consideration ..." Estimating future subsidence out 50 or more years, like sea level change, is difficult, and must be based on extrapolated trends from past geodetic surveys and water level gage data.

8-3. Development of a Vertical Time Dependent Positioning Reference Framework to Monitor Bench Mark Subsidence in Southern Louisiana. This section describes the process developed by the NGS for establishing a VTDP network in Southern Louisiana. This VTDP network is used to evaluate subsidence at bench marks in the subsidence area. A VTDP network must be continuously monitored and periodically updated—i.e., the NAVD88 (2004.65) VTDP network was subsequently updated to a NAVD88 (2006.81) epoch. The procedures outlined below are applicable to other USACE projects subject to subsidence.

a. Beginning in 2004, NGS began a series of reobservations in Louisiana for the purpose of updating the NAVD88 published heights in the region in support of hurricane evacuation route mapping. These reobservations included both GPS campaigns and leveling observations. The GPS data were collected according to NGS standards in "*Guidelines for Establishing GPS-Derived Ellipsoid Heights: Standards: 2 cm and 5 cm*" (NOAA 1997) and in "*Guidelines for Establishing GPS Derived Orthometric Heights: Standards: 2 cm and 5 cm*" (NOAA 2005). These guidelines required a set of three 5½ hour static DGPS sessions with at least a 4 hour difference in the starting time of one session on different days. The data collected was processed using the NGS program "PAGES" and adjusted using the NGS program "ADJUST." However, prior to this adjustment, the published orthometric heights of bench marks in the Gulf Coast region from Pensacola, FL west to Houston, TX (which included bench marks occupied in the GPS reobservations in Louisiana) were updated using the most recent subsidence rates as published in NGS Technical Report 50—"*Rates of Vertical Displacement at Benchmarks in the Lower Mississippi Valley and the Northern Gulf Coast*" (NOAA 2004). These rates were applied to previous observation data and adjusted. This readjustment used 151 previously observed level lines connecting across the entire region, consisting of 16,331 bench marks. Rates of all published bench marks included in NGS Technical Report 50 (NOAA 2004) were applied. A total of 85 such bench marks were part of this reobservation campaign, as shown in Figure 8-5.

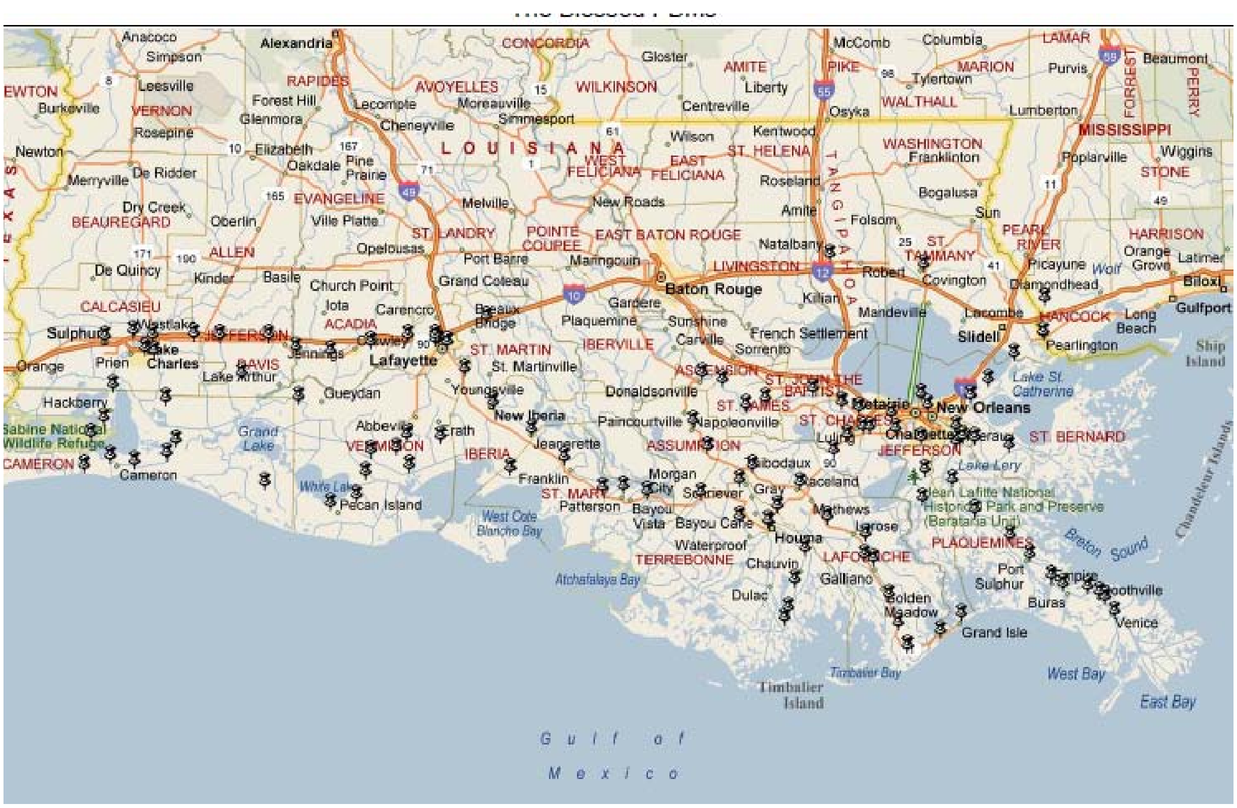


Figure 8-5. Southern Louisiana Vertical Time Dependent Network (adjustment epoch 2004.65).

b. When the GPS-derived orthometric heights were compared with leveling data at these 85 bench marks, as corrected for subsidence rates and tied to non-subsiding bench marks outside the subsidence area, there was a variety of agreements and disagreements. First, 32 of the 85 bench marks showed better than 2 cm agreement between the GPS-derived and leveling-derived orthometric heights, indicating a good estimate of subsidence rates at those points.

c. After finding the 32 points with the most reliable estimated subsidence rates, their heights were then held as stochastic constraints in a constrained adjustment of all 85 bench marks (along with fixing the heights of 4 points outside the subsidence area). The resultant adjustment of 85 heights was given the notation "NAVD88 (2004.65)", where the 2004.65 is the date in years and decimal portions of a year of the midpoint of the observation campaign. The formal accuracy estimates on these 85 bench marks fall in the 2 to 5 cm range. Note that even as these points have been adjusted to 2004.65, they are all susceptible to subsidence, and therefore it will be critical to use CORS data and possibly future re-leveling to re-adjust these heights and recompute their subsidence rates with a higher accuracy than the 2004.65 adjustment produced.

d. The NAVD88 (2004.65) adjustment, again, was not a local adjustment. It went outside of the subsidence area and held fixed what were felt to be stable bench marks. The four bench marks held fixed were: LAKE HOUSTON 2050, which is a galvanized steel pipe driven to a depth of 2050 feet; 872 9816 TIDAL 1 a TIDAL Bench mark in Pensacola, Florida; FOREST EAST BASE in Scott County, Mississippi; and M 237 in Latanier, Louisiana. A free adjustment holding LAKE HOUSTON 2050 fixed was run with the results shown in Table 8-1. The difference between the NAVD88 (1994) and NAVD88 (2004.65) reflects the apparent subsidence of the bench marks due to the procedures used in the adjustment.

Table 8-1. Louisiana VTDP Free Adjustment.

DESIGNATION	PUBLISHED (m)	ADJUSTED (m)	PUB-ADJ (m)
872 9816 TIDAL 1	1.3479	1.3741	-0.0262
FOREST EAST BASE	136.4527	136.4622	-0.0095
LAKE HOUSTON 2050	17.0714	CONSTRAINED	0.0000
M 237	20.3830	20.3422	0.0408

e. The geographical location of these fixed bench marks relative to the Southern Louisiana subsidence area is shown in Figure 8-6.

8-4. <u>Estimating Subsidence Rates in the Southern Louisiana Region from Geodetic Observations</u>. This section focuses on subsidence occurring at bench marks throughout the Southern Louisiana project area. A bench mark's subsidence rate may be different from that occurring in the adjacent ground—see Figure 8-2 where the deep-driven rod bench mark protrudes well above the subsided ground. Over the years, there have been several studies that have been published documenting the subsidence of New Orleans and Southern Louisiana.

Figure 8-6. Location of fixed bench marks defining NAVD88 (2004.65) in Southern Louisiana.

a. A NOAA report “Subsidence in the Vicinity of New Orleans as Indicated by Analysis of Geodetic Leveling Data” (NOAA 1986) used three different adjustments to determine the apparent movement of bench marks in this area. This report does not show sea level rise--only the apparent movement of the benchmarks. It should also be noted that the movement reflected in this report, as well as in NOAA Technical Report 50 (NOAA 2004), reflects the movement of the mark based on leveling observations. Table 8-2 shows not only the apparent subsidence but also that the subsidence is neither linear nor at the same rate based on location and different epochs.

Table 8-2. Apparent Movement (in mm/year) without Sea Level Rise from Three Leveling Networks (1951-1955, 1964, and 1984-85) to Estimate Apparent Crustal Movement. (NOAA 1986)

PBM Designation	1985.0 – 1964.0	1985 – 1951	1964 – 1951
A 148 (AU0429)	-6.88 (21 yr)	-5.57 (34 yr)	-3.1 (13 yr)
PIKE RESET (BH1164)	-1.36	-1.59	-1.97
231 LAGS (BH1073)	-16.39	-10.90	-2.03
A 92 (BH1136)	-2.36	-2.66	-3.13

b. The rate of subsidence varies from epoch to epoch (survey to survey) due to many factors, such as compaction, removal of subsurface fluids, and geologic events. Therefore, one cannot predict future subsidence with any degree of accuracy. Table 8-3 shows the rate of change reflected in at least two different epochs of First-Order, Class II leveling, as published in NOAA Technical Report 50 (NOAA 2004).

Table 8-3. Apparent Movement from Two Epochs of Leveling Data. (NOAA 1986)

PBM Designation	Rates of Movement (mm/year)
A 148 (AU0429)	-11.01
PIKE RESET (BH1164)	-6.99
231 LAGS (BH1073)	-16.08
A 92 (BH1136)	-7.39

c. The average rate of apparent subsidence across the region was found to be about 0.6 ft subsidence per 10 years. This indicates that elevations published in the 1960's, 70's, 80's, and early 90's may have changed even more than 1 ft. A long-term objective is to continually improve upon the vertical reference system in Southern Louisiana—e.g., NAVD88 (2004.65) was later updated to NAVD88 (2006.81). This provides a consistent framework from which the monitoring of previously constructed and proposed flood risk management and hurricane protection structures can be performed.

d. Figure 8-7 depicts estimated subsidence rates occurring at 18 benchmarks in the New Orleans region based on the adjusted elevations. The subsidence rates were computed using the difference between the published NAVD88 (2004.65) and superseded values and dividing them by the number of years between the adjustments. These rates, compared with those published in NOAA Technical Report 50 (NGS 2004), do not all agree since the adjusted elevations contain distributed errors from the adjustment computations. Therefore, Figure 8-7 illustrates the need to use unadjusted values in determining subsidence rates as documented in NOAA Technical Report 50.

8-5. Sea Level Trends in Southern Louisiana. Long term tide station records provide estimates of local relative sea level trends as opposed to the absolute rates of global sea level that are the subject of basic research in climate change. These local relative sea level trends from tide stations are a combination of global sea level variations, regional climate scale water level variations, and local vertical land movement due to local or regional subsidence. Thus the tide stations provide the information because they provide direct information on variations of water levels relative to the local land elevations.

a. Figure 8-8 depicts the apparent sea level increase (i.e., mostly subsidence) over a 60-year period at the USACE Florida Avenue gage on the Inner Harbor Navigation Canal (IHNC) in New Orleans, LA. The apparent sea level rise at this gage supports independent geodetic observations and observed elevation decreases on hurricane protection structures in this area.

PID	Designation	Rate (m/yr)	NAVD88 2004.65 (m)	Procedure	Sup/Date	Sup (m)	Sup (ft)	Leveling Year	NAVD88 (2004.65) minus Sup (ft)
BH1119	C 189	-0.016	0.63	LEVELING(2004.65)	12/5/1996	0.794	2.60	1994	-0.54
AU2163	B 369	-0.015	1.84	LEVELING(2004.65)	12/5/1996	1.975	6.48	1995	-0.44
AU2310	876 1899 B TIDAL	-0.015	0.01	LEVELING(2004.65)	12/5/1996	0.141	0.46	1995	-0.43
AU0429	A 148	-0.015	1.77	GPS OBS(2004.65)	12/5/1996	1.915	6.28	1994	-0.48
BJ1342	ALCO	-0.014	1.87	LEVELING(2004.65)	12/5/1996	2.008	6.59	1994	-0.45
AT0804	REGGIO 2	-0.012	1.52	GPS OBS(2004.65)	2/14/1994	1.714	5.62	1988	-0.64
BH1212	A 193	-0.012	0.75	LEVELING(2004.65)	2/14/1994	0.879	2.88	1993	-0.42
AU2110	G 365	-0.011	0.24	GPS OBS(2004.65)	12/5/1996	0.342	1.12	1995	-0.33
AT1390	876 0849 A TIDAL	-0.011	0.85	GPS OBS(2004.65)	8/31/2001	0.972	3.19	1993	-0.40
AT0407	A 152	-0.010	0.67	GPS OBS(2004.65)	2/14/1994	0.870	2.85	1984	-0.66
BJ3744	S 379	-0.010	4.31	GPS OBS(2004.65)	2/14/1994	4.482	14.70	1986	-0.56
AT0376	R 194	-0.008	1.39	GPS OBS(2004.65)	2/14/1994	1.554	5.10	1984	-0.54
AT0357	D 194	-0.008	1.68	LEVELING(2004.65)	2/14/1994	1.835	6.02	1984	-0.51
AT0200	MILAN 2	-0.008	-0.15	GPS OBS(2004.65)	2/14/1994	0.005	0.02	1984	-0.51
AT0332	L 278	-0.007	2.11	LEVELING(2004.65)	2/14/1994	2.253	7.39	1984	-0.47
AT0231	EMPIRE AZ MK 2 1934 1966	-0.007	-0.01	GPS OBS(2004.65)	2/14/1994	0.129	0.42	1984	-0.46
AT0247	C 279	-0.007	-0.23	GPS OBS(2004.65)	2/14/1994	-0.100	-0.33	1984	-0.43
AT0731	N 367	-0.007	0.34	GPS OBS(2004.65)	2/14/1994	0.470	1.54	1984	-0.43

Figure 8-7. Estimated subsidence rates at selected bench marks in New Orleans Region. (IPET 2007) (Note: "Sup" = superseded)

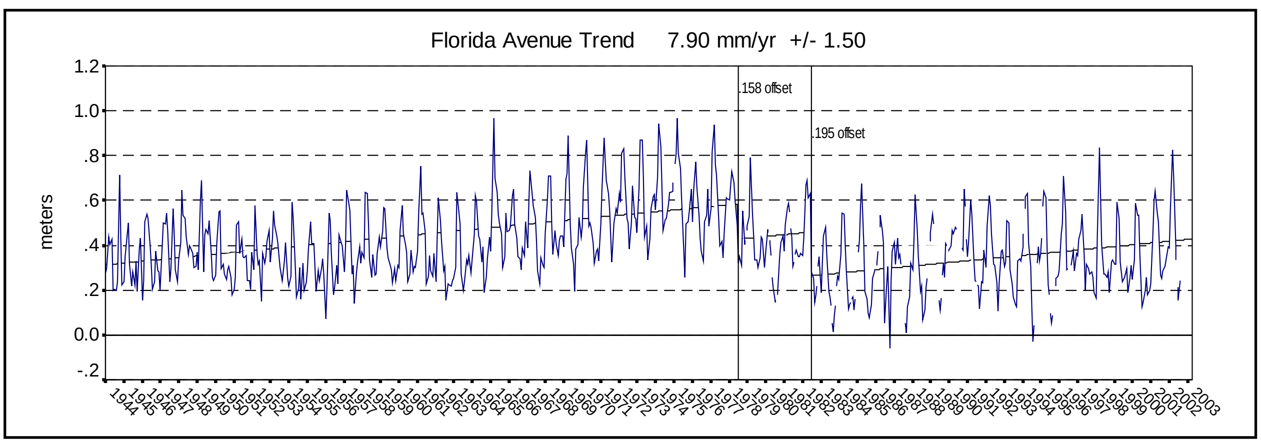


Figure 8-8. Apparent sea level rise at Corps IHNC Florida Ave. gage from 1944 to 2003. Gage zero adjustments were estimated. (IPET 2007)

b. NOAA CO-OPS has performed analyses of relative sea level trends for all of the long-term NWLON stations in their network. Unfortunately, the New Orleans area and Lake Pontchartrain are geographical areas of data gaps for locations with measurements of sea level variations necessary to estimate sea level trends with high certainty. The closest NWLON stations in this category are Dauphin Island, AL; Pensacola, Fl; and Grand Isle, LA. The analyses done for estimating relative sea level trends in the New Orleans area include using a 23-year monthly mean time series pieced together from Waveland, MS (3.52 mm/yr with a ± 2.6 mm/yr 95% confidence interval) and a 10-year monthly mean time series at New Canal, LA (3.98 mm/yr with an 95% confidence interval > ± 3.0 mm/yr). Historical once-per-day readings from long term USACE stations have also been analyzed; however, there have been many adjustments to the gages that were not readily available for this review.

(1) Analysis of the USACE record at Florida Avenue, New Orleans, LA provides a composite estimate of 7.90 mm/yr with a 95% confidence interval of ± 1.5 mm/yr.

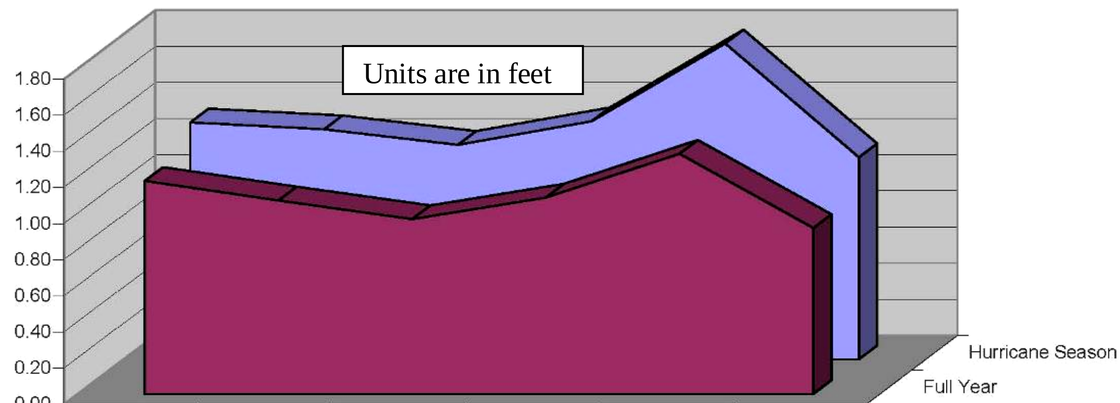
(2) Using an assumption of similar ratio relationships of shorter period trends to longer period trends, the relative sea level trend at NWLON New Canal gage was estimated to be 6.83 mm/yr for a 23-year period (comparing with Waveland trends).

(3) By performing a difference of the simultaneous monthly mean sea levels between New Canal and Waveland, a trend fit to the differences shows that relative LMSL is rising 1.9 mm/yr faster at New Canal than at Waveland. Adding the 1.9 mm/yr rate to the 3.98 mm/yr estimate for 10-months gives an estimate of 5.88 mm/yr.

c. Although limited by the 10-year period length and with a spread of 2 mm/yr, these three independent estimates of the relative sea level trend at the New Canal gage are consistent with independent estimates of local subsidence in the region based on NOAA Report 50 (NOAA 2004), which relied on repeat geodetic surveys.

d. The results of the analyses used to estimate relative sea level trends for the Southern Louisiana study area provide corroboration of the drawbacks of estimating sea level trends from only a few decades of measurement, and the need to look at simultaneous time periods when comparing trends across a region.

8-6. Seasonal Variation in Mean Sea Level in Southern Louisiana. The average seasonal cycles in monthly local mean sea level can show wide variations depending on the seasonal variations in water temperature, winds, and circulation patterns currents in the nearby coastal ocean. Figure 8-10 shows four plots of monthly local mean sea levels the coastal region from Pensacola west to Grand Isle. It can be seen that there is as low progression from a single mode of a seasonal high and low sea level stand at Pensacola (high in September, low in January) to a bi-modal variation at Galveston, TX with secondary high and low in May and July respectively. Hurricane season, from June through November, coincides with the periods of high monthly local mean sea levels--this generally adds to the elevation of storm surge. Seasonal variations in the New Orleans IHNC are shown in Figure 8-9. This data were constructed by computing average water surface elevations for selected years at the USACE Florida Avenue gage. Elevations are in feet and are referred to approximate LMSL or NGVD29 (1983 adjustment). Figure 8-10 clearly shows a quarter-foot bias in average surface elevation during the fall hurricane season. Hydrodynamic modeling, risk analysis, and design criteria need to consider this seasonal bias in evaluating flood protection elevations.



	1998	1999	2000	2001	2002	2003
Full Year	1.18	1.07	0.97	1.09	1.33	0.92
Hurricane Season	1.31	1.27	1.19	1.32	1.75	1.12

Figure 8-9. Seasonal variations (in feet) at the New Orleans IHNC Florida Avenue gage.

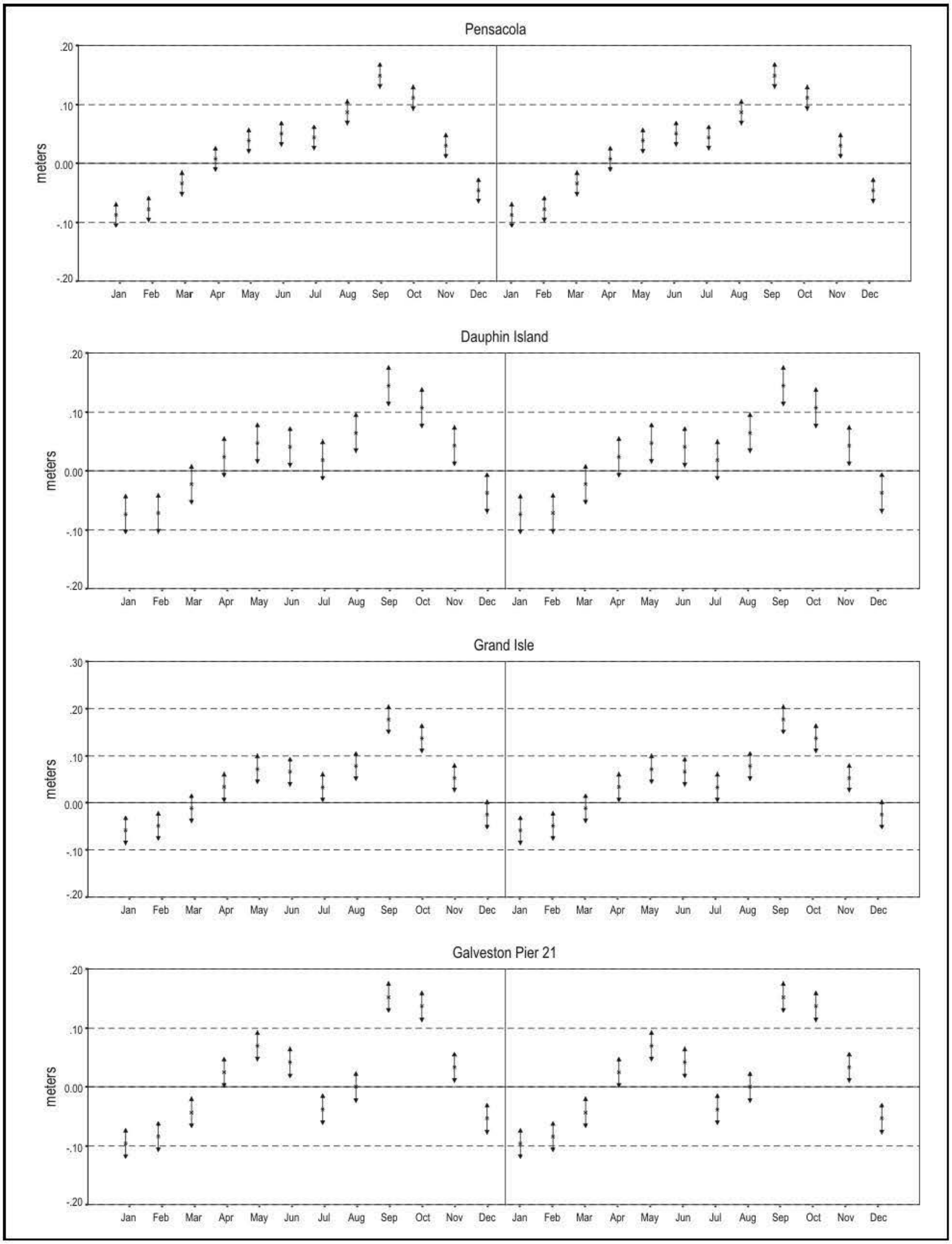


Figure 8-10. Monthly local Mean Sea Levels from Pensacola to Galveston.
(in meters above LMSL)

CHAPTER 9

Checklist for Assessing Project Datums and Elevation Uncertainties through Project Phases

9-1. Purpose. This chapter provides guidance for evaluating the adequacy of elevation grades and reference datums through the life cycle of a project. This includes evaluations or assessments to ensure the project is referenced to the current NSRS and/or NWLON framework. Procedures for estimating project grade or depth measurement uncertainties are also outlined.

9-2. Planning and PED Phases—Reference Datum Checklist. During the planning and/or detailed design phases (e.g., PED), water level datums, geodetic datums, and topographic elevation references shall be clearly defined and established throughout the project area. This entails setting Primary Project Control Points (PPCPs) at a spacing sufficient to densify supplemental (local) control for subsequent engineering and construction surveys, as outlined in previous chapters of this manual. All PPCPs must be published in the NSRS. The project area includes not only the planned location of a flood protection structure but also related flood plain mapping on the protected side of the control structure and perhaps hydrographic surveys on the flood side. Navigation projects may include external confined disposal and beach renourishment sites. These design reference surfaces must be established prior to performing basic site plan mapping, aerial mapping, LIDAR elevation mapping, hydrographic surveys, geotechnical investigations, and related preliminary design requirements. The main issues to be evaluated and resolved during the preliminary planning and/or design phases are listed in Table 9-1.

Table 9-1. Reference Datum Checklist—Planning and PED Phases (Navigation, Flood Risk Management, and HSPP Projects).

PROJECT ELEMENT	ACTION
Establish Primary Project Control PBMs	Use existing (published) NSRS PBM or survey new PBM and submit/publish in NSRS—see Chapter 3
Reference datums	NAD83, NAVD88, & hydraulic/tidal
Accuracy required	see nominal standards in Table 3-1
Density of Primary Control PBMs	see Chapter 6 (Inland projects)
Recommended survey procedures	see Chapter 3
PPCP satellite visibility	Verify horizon clearances

Table 9-1 (Continued). Reference Datum Checklist—Planning and PED Phases (Navigation, Flood Risk Management, and HSPP Projects).

PROJECT ELEMENT	ACTION
Local construction reference PBMs (LPCP)	Survey connections made directly from PPCP PBMs
Datum	NAD83, NAVD88, & hydraulic/tidal
Local construction datum	Note relationship to NSRS (NAVD88)
Density of LPCPs	Ensure spacing within leveling or RTK ranges to project
Local relative accuracy	see Chapter 3
PBMs indicated in contract documents	minimum of three required for construction plans & specs
Legacy Datums	Document reference to NSRS (NAVD88)
Protection Grade Elevation References	Ensure referenced to NSRS (NAVD88) from PPCP/LPCP ties
Subsurface investigation boring reference elevation	NSRS/NAVD88—connected from PPCPs or local PBMs
Site plan mapping reference datums	NAD83 and NAVD88 (current adjustments and epochs) and Local Station-Offset system
Detailed topographic site plan accuracies (hard features, ground shots, etc)	See Chapter 3 (total station or RTK methods relative to PBMs)
Hydraulic/tidal gage reference PBMs	Directly referenced to river/tidal gage reference datum
Minimum number of gage reference PBMs	3 (one PBM must be connected to/published in NSRS)

Table 9-1 (Concluded). Reference Datum Checklist—Planning and PED Phases (Navigation, Flood Risk Management, and HSPP Projects).

PROJECT ELEMENT	ACTION
Navigation MLLW datum modeling	VDatum or spatially interpolated—document model
Navigation RTK base station	Documented in plans and published in NSRS
Navigation tidal PBM calibration points	Documented in plans and connected to CO-OPS network
Metadata	Design memorandums, project drawings, CADD files, studies, reports, flood profile diagrams and related framework documents contain full and complete metadata on the reference elevation datum, primary project control PBMs, and local construction control PBMs; including the relationships and estimated accuracies of legacy reference datums, bench marks, and designed protective elevations.

9-3. Construction Phase Checklist.

a. Minimum construction stakeout criteria. Local horizontal alignment and vertical control PBMs established during the detailed design phase and shown on the contract plans shall be thoroughly verified during the initial construction stakeout. This verification entails checks to a minimum of three PBMs shown on the contract drawings. Checks between the local reference points should generally agree to within ±0.05 ft. Checks on horizontal alignment control points or bench marks exceeding these tolerances shall be thoroughly investigated and resolved prior to construction stake out. The government construction inspector shall review in progress (on site) initial construction stakeout work and shall thoroughly review the contractor's stake out notes for both the basic control check and the site stake out.

b. Machine control system calibration. Machine control positioning systems on graders and bulldozers must be verified on site to ensure horizontal and vertical grading references check with fixed project control bench marks. Machine control RTK networks must also be adequately "site calibrated" prior to excavation or grading, ensuring fixed calibration bench marks surround the construction site.

c. Verification of as-built floodwall cap elevations. Post-construction profile or topographic surveys of floodwalls shall be made to verify as-built controlling elevations and horizontal location. These surveys may be performed using total stations, levels, or RTK methods. Surveys must originate from the reference control PBMs shown in the contract plans. Elevations of sheet pile or floodwall caps should be recorded to the nearest 0.1 ft.

d. Navigation project control verification. RTK or RTN horizontal positioning calibration shall be checked or site calibrated at independent PBMs. RTK/RTN water surface elevation measurements shall be calibrated to the local river or tide gages. In tidal areas verify the latest MLLW gradient model is being used. Levels should be run between tidal reference PBMs to verify stability. Staff gages should be set by leveling to a minimum of two reference PBMs.

9-4. Post-Construction (Operation and Maintenance) Phase—Periodic Reassessments or Evaluations of Controlling Reference Elevations. Periodic reevaluations of project reference elevations and related datums shall be included as an integral component in the various civil works inspection programs of completed projects. The frequency that these periodic reevaluations will be needed is a function of estimated magnitude of geophysical changes that could impact designed protection grades. Most USACE projects are in relatively stable areas and can be evaluated at less frequent intervals. Some criteria for determining resurvey frequencies might include: (1) protected population areas, (2) known insufficient datums, (3) known settlement problems, (4) known subsidence or crustal uplift, (5) District or sponsor priority, (6) type for flood protection structure, or (7) structure height. Navigation project grades or flood protection elevations that are referenced to tidal datums will have to be periodically coordinated with and/or reviewed by NOAA CO-OPS to ensure the latest tidal hydraulic effects are incorporated and that the project is reliably connected with the NSRS. For dams, levees, and related structures, a complete reevaluation of the vertical datums should be conducted at the frequency specified in the O&M Manual for the project; typically ranging from 2 to 5 years in high subsidence areas to 10 or more years in stable areas. Any uncertainties in protection levels that are identified during the inspection should be incorporated into any applicable risk/reliability models developed for the project. Technical guidance on periodic inspection monitoring surveys is found in EM 1110-2-1009 (*Structural Deformation Surveying*).

a. Reference bench mark verification. Periodic resurveys shall be performed relative to the PPCPs established for the project. The NSRS datasheet shall be reviewed to determine if NGS has revised the elevation for the primary mark. The stability of the PPCPs shall be verified by GPS observations or differential level runs to adjacent NSRS reference bench marks. Checks to ± 0.1 ft would be a reasonable tolerance. The PPCP should normally be used as the base station when GPS RTK surveys are performed at the project site.

b. Topographic survey methods. Topographic surveys of floodwall caps, levee or floodwall profiles, inverts, pump stations, etc. should generally meet the tolerances indicated in Chapter 3, which are relative to the LPCPs and/or indirectly to the NSRS PPCPs. Differential leveling (spirit or digital), GPS RTK, or total station methods should yield ± 0.1 ft relative accuracies on surveyed points relative to LPCPs. Reference also topographic surveying methods in EM 1110-1-1005 (*Control and Topographic Surveying*).

c. Profile surveys of levee grades and floodwall caps. Periodic topographic surveys of levee and floodwall elevations shall be performed to verify the current protection elevations. Either differential leveling or GPS RTK methods may be used—RTK normally being the most efficient method given 3D coordinates are directly observed at each point. Shot points are taken at 50-ft or 100-ft intervals along the structure, breaks in grade, gate structures, monoliths, and at other features as designated.

d. Topographic sections on protected or flood sides of floodwalls. Floodwalls set atop or around bridges, levees, pump stations, and other facilities may require periodic topographic surveys of the surrounding berms, revetments, chords, or water depths. Subsurface hydrographic surveys may be required in adjacent canals or rivers to check for scour into the levee revetment or floodwall base. The density of such surveys will depend on the potential scour or settlement being monitored. Typically, 50- to 100-ft sections will be surveyed using standard topographic survey methods, such as GPS RTK.

(1) Hydrographic surveys of deeper water on the flood side can be performed following the techniques outlined in EM 1110-2-1003 (*Hydrographic Surveying*). In shallow river or canal areas (i.e., < 15 ft water depth), standard leveling or total station topographic survey methods may be used with a 25-ft expandable level rod. Typical cross-section spacing is 50 ft or 100 ft c/c.

(2) Acoustic depths may be taken from a boat using inexpensive single-beam survey methods. If 100% bottom coverage is required to evaluate scour or other anomalies in a floodwall or levee footing, then either multi-transducer or multibeam survey systems may be employed, depending on water depth and other factors. Other high-definition acoustic devices may also be used.

e. Deformation and deflection measurements. Many of the precise survey procedures used for large dams outlined in EM 1110-2-1009 (*Structural Deformation Surveying*) may be applied to levees and floodwalls—on an isolated basis given the large geographical extent of floodwalls as compared to dams. This would include precise differential leveling to monitor regional subsidence and settlement, and crack or monolith lateral movement using micrometers. A number of options exist to monitor relative (internal) horizontal deflections of individual floodwall sections. Overall (global) lateral deformation or translation requires monitoring from undisturbed permanent reference points.

f. Navigation and coastal shore protection structures. Coastal navigation, shore protection, and hurricane protection projects need to be periodically evaluated to check for updates to the reference sea level datum. This is normally performed during the development of maintenance dredging plans & specifications.

(1) A periodic assessment of these projects is intended to verify (1) that the design/constructed sea level reference datum is current (i.e., latest tidal datum epoch and model) and (2) that the local project control has been connected with the latest NSRS (NAVD88) adjustment.

(2) Many shore protection projects were originally designed to sea level datums based on interpolated or extrapolated references from gages. Depending on the type of gage, tidal range, and the distance from the gage, this interpolation or extrapolation may no longer be valid or sufficiently accurate—i.e., generally within ± 0.25 ft of the reference water level datum. With sea level rise, the crest elevation of structures may be below that originally designed. However, the original design documents should be checked to verify that allowance for sea level rise was considered in the design elevation and is consistent with the current condition.

g. Coastal navigation project reference datums. Reference tide gages should be checked for periodic datum updates or corrections by NOAA CO-OPS. Updates to VDatum models should also be checked to make sure the latest revisions are accounted for in the model.

h. Checklist. Table 9-2 summarizes some of the items that should be evaluated during periodic inspections or resurveys of levees and floodwalls.

Table 9-2. Summary of Requirements for Referencing Levee and Floodwall Elevations during Post-Construction Maintenance (Periodic Evaluation) Phase.

Post-construction Operation & Maintenance	Periodic inspection and verification of reference hydraulic/tidal and geodetic NSRS datums, subsidence, and sea level changes
Verification of Primary/local PBM relative to NSRS regional network	Check tolerance: ± 0.1 ft (3D)—see criteria in Chapter 3
Topographic inspection survey density:	
Floodwall cap profile surveys	25 to 100 ft shot points (typical) plus breaks in grade
Cross-section topo/hydro surveys	50 or 100 ft c/c typical
Resolution	± 0.1 ft (3D) typical

9-5. Sample PED Evaluation Report on a Hurricane Protection Project's Reference Datums. The following example report contains excerpts from an evaluation of reference datum connections in a New Orleans District Design Report. This evaluation checklist and report reviewed the reference datums used for various engineering disciplines covered in the report. The initial checklist indicates areas that will require additional field survey or design review effort.

Comprehensive Evaluation of Project Datum--Quality Control Checklist (New Orleans District)

Title: LPV-12.2, Hurricane Protection Project, Jefferson Lakefront, Fronting Protection, Duncan Pumping Station, Jefferson Parish, Louisiana - Design Report

Prepared by: mh
Checked By: rmf
Date: 31-May-07

General Checklist

[No] Gages Referenced To both NAVD88 and Latest Epoch Tidal Datum (MLLW, LMSL)
[???] Gage Inspection Current
[No] Do Plans Document 3 PBMs
[No] Is A PBM Tied To NAVD88 and Tidal Datum
[N/A] Is Navigation Project Tied To MLLW
[???] Has Subsidence and Sea Level Rise Been Considered
[Yes] Are Units Specified (US Survey Foot)
[Yes] State Plane Zone Specified
[No] Do Project PBMs Indicate Epoch, Datum, Description, Elevation

Comments [Excerpted]:

<u>Executive Summary</u>

3.0 Site Survey Plan
This is very good. It makes it clear which horizontal and vertical systems were used in the preparation of the topographic survey and also makes it clear that older pump station plans are referenced to Cairo 1910 and incorporated into the report for "informational purposes only."

<u>Design Report</u>

3.0 Site Survey Plan
Good. Includes additional information that confirms Geoid03 (revised for South Louisiana in Oct '05) was used in GPS processing. Refer to Appendix C below for more detail.

10.0 Preliminary Cost Estimate
Drawings in 10.2 Demolition indicate "NGVD" in margin. Should be NAVD88 (2004.65)

<u>Appendix A - Scope of Work</u>

7. Site Surveys and Mapping
All federally funded Hurricane Protection, Flood Control, Shore Protection, and Navigation projects require documentation of the following:
1) Reference to accurate and current hydraulic/tidal datum (e.g., LWRP, LMSL, MLLW) based upon an adequate gage network with ties to NOAA using latest tidal epoch.

2) Three bench marks at each gage relative to NAVD88 (latest adjustment/epoch with at least one PBM directly tied to the NSRS) and from which rigorous gage inspections are performed and documented (bench mark ties to 3rd order)

3) Reference/relationship from latest epoch of NAVD88 to construction/design datum if other than NAVD88 or current hydraulic/tidal datum (e.g., relationship from NAVD88 to MLG, NGVD29, Cairo, etc.).

A total of three bench marks needs to be identified or established at the project site in accordance with CEMVN-ED-SS-06-01, "USACE New Orleans District Guide for Minimum Survey Standards for Performing Hydrographic, Topographic, and Geodetic Surveys" and the location, identification and elevation of these bench marks needs to be shown on all relevant project sheets/drawings (see Appendix C note below).

8. Geotechnical Explorations, Test, and Analysis
Soil Borings and Cone Penetrometer test to be tied to baseline with station and offset and X/Y and elevation given with respect to project reference systems (see Appendix D note below).

Appendix B - Plates

Plate #4 - Confirm survey baseline referenced to Geodetic North Azimuth and add stationing, PI Coordinates, source, ID, etc. or remove completely.

Plate #27
" Existing elevations are per the design drawings.*
Actual as-built elevations are +/-0.8' lower"
This is a very good and helpful note.

Appendix C - Site Survey Plan

Need to show site map and data sheets for control points "082806GD" and NGS "BUICK" (pictures, description, location, references, elevation, etc.) and establish a third bench mark with all of this information.

Appendix D - Geotechnical Investigation

Only two soil boring logs shown (of eight proposed). Ground elevations for two soil boring locations shown on log, margin info indicates NGVD (confusing/ambiguous).

Station/offset info for proposed soil boring and CPT test sites unclear. Need coordinates and elevations of all soil boring and CPT sites.

General - Include statement: All surveys shall be conducted in accordance with CEMVN-ED-SS-06-01, "USACE New Orleans District Guide for Minimum Survey Standards for Performing Hydrographic, Topographic, and Geodetic Surveys" and shall be submitted to ED-SS. The guidance is available at http://www.mvn.usace.army.mil/ed/edss/surveyingguidelines.asp

9-6. Elevation Uncertainty Estimates of Reference Grades. The surveyed elevation of a flood protection structure or navigation grade has an uncertainty due to the propagated errors of all the uncertainties in the components that derived the elevation. These include the regional geodetic PPCP datum uncertainties, hydraulic or tidal datum uncertainties, water level gage references, local LPCP datum uncertainties, topographic/hydrographic survey errors, feature irregularities, etc.

a. For example, if the PPCP for a levee project has an estimated NSRS accuracy of ± 0.2 ft, and topographic surveys or the levee profile are performed through local LPCPs on the levee, then the resultant NSRS accuracy of a ground shot atop the levee (or a first-floor elevation in the flood plain) could propagate to as much as ± 0.5 ft. Likewise, the resultant elevation accuracy of a navigation project grade or HSPP structure elevation depends on reliability of the tidal datum, sea level change estimates, and the depth measurement process.

b. Uncertainties in navigation depths will normally range between ± 0.5 ft and ± 1 ft, or larger in some projects. These propagated uncertainties must be estimated for each project and factored in to the risk analysis or design of a protection grade or navigation channel design grade—see EM 1110-2-1619 (*Risk-Based Analysis for Flood Damage Reduction Studies*). Table 9-3 lists typical elevation uncertainty estimates for inland and coastal projects.

Table 9-3. Typical Elevation Uncertainty Estimates of Gages and Project Features Relative to NSRS.

Feature	Elevation Uncertainty Project (Standard Deviation 95%)
River Gages	
Gages directly connected to NSRS based on direct leveling or DGPS satellite observations	± 0.05 ft to ± 0.2 ft
Gages on legacy datums with firm (published NSRS) relationships	± 0.15 ft to ± 0.4 ft
Gages on legacy datums without firm (or unknown) connections to national vertical network	± 0.5 ft to ± 2.0 ft

Table 9-3 (Concluded). Typical Elevation Uncertainty Estimates of Gages and Project Features Relative to NSRS.

Feature	Elevation Uncertainty Project (Standard Deviation 95%)
Topographic Feature Elevations (Propagated Errors)	
Levee/floodplain/first-floor elevations based on direct connections with current NSRS bench marks	± 0.2 ft to ± 0.3 ft
Levee/floodplain/first-floor elevations based on legacy datums and uncertain PBM origins	± 0.5 ft to ± 1 ft
Levee/floodplain/first-floor elevations based on legacy datums but firmly related to current NSRS vertical network	± 0.3 ft
Coastal Project Grades	
Tide Gages (function of period of record, epoch, etc.—see Chapter 4)	± 0.2 ft to ± 0.5 ft
Tidal model at project site hydrodynamically modeled to local NOAA LMSL datum	± 0.1 ft
Tidal model at project site estimated based on unknown or outdated tidal datum (uncertainty function of tide range and distance from original gage)	± 0.2 ft to ± 0.5 ft
Navigation channel depth or HSPP grade (propagated error)	± 0.5 ft to ± 1.0 ft

c. Appendix M *(Uncertainty Model for Orthometric, Tidal, and Hydraulic Datums for use in Risk Assessment Models)* discusses methods for estimating overall datum and survey uncertainties on USACE project grades, and the statistical factors that should be considered in arriving at risk assessments associated with datum uncertainties. This appendix contains practical examples of the factors (such as those in Table 9-3) that must be incorporated in datum uncertainty computations.

9-7. Computing Elevation Uncertainties in the Design of Flood Protection and HSPP Structures. Uncertainty is defined as the result of imperfect knowledge concerning the present or future state of a system, event, situation, or (sub) population under consideration. Datum and resultant elevation uncertainties of reference PBMs and gages must be factored in the design of protection elevations on inland or coastal flood protection structures. Uncertainty "allowances" also factor in to risk-based design of protection elevations, which involves estimating the probability and severity of undesirable consequences of a failure, e.g., loss of life, threat to public safety, environmental and economic damages. Risks associated with datum and subsidence uncertainties would involve potential overtopping during flood stages. General guidance on these design considerations is summarized below.

> *Loss of protection due to lowering of the top of flood barrier relative to design water levels shall be accounted for in any flood risk management project with site geology that is undergoing long term regional settlement [subsidence] and/or on coastlines where future sea level rise is occurring. For the system to be reliable, the top of the flood protection must be able to provide the required design height over the service life of the project. In areas where subsidence is a concern, a comparative analysis shall be performed ...To ensure reliability of the system, and to account for local settlement caused by the weight of levees, or from general lowering of an area relative the water level due to regional subsidence and/or sea level rise, flood risk management projects shall be initially constructed to a height sufficient to maintain the required height for all future conditions. Flood risk management projects shall also be constructed to the design level for current conditions with allowance for raising in the future to meet design heights as settlement and/or subsidence occurs.*

a. An additional freeboard allowance can be estimated that will account for geodetic datum uncertainties and long-term subsidence. The floodwall depicted in Figure 9-1 depicts freeboard allowances for uncertainties in the regional geodetic datum and regional subsidence. These allowances may be estimated from the ranges shown in Table 9-3 and from the uncertainty estimates listed in Table 9-4 in the following section.

b. Application of these uncertainty allowances are outlined in EM 1110-2-1619 *(Risk-Based Analysis for Flood Damage Reduction Studies*). .

Figure 9-1. Allowances for geodetic datum and subsidence in risk-based design.

9-8. Site Information Classifications and Requirements. Table 9-4 provides general site information classifications for reference datums, based on various levels of potential adverse site conditions. These classifications apply to the design of new protection structures or an evaluation of existing projects. "Well-Defined" or "Ordinary" classifications are considered acceptable. "Limited" site information will require additional field survey data. Datum or subsidence uncertainty estimates shown in the table should be factored into design risk assessment models and floodwall height overbuild computations.

Table 9-4. Site Information Classifications and Uncertainty Estimates (95% Confidence Levels) of the Primary Project Control Point (PPCP).

Condition	Well-Defined	Ordinary	Limited
Connection to existing NSRS PBM	1st Order PBM in NSRS	2nd Order NSRS PBM	3rd or 4th NSRS PBM
Surveyed connection method with NSRS	1st/2nd Order differential levels	2/5 cm NGS GPS standards 3rd Order levels GPS CORS/OPUS	GPS RTK or unknown method
Reference orthometric datum	NAVD88	NAVD88	NGVD29
Published in NSRS	Yes	Yes	No
Estimated network orthometric datum accuracy relative to NSRS	± 0.02 ft to < ± 0.10 ft	> ± 0.10 ft to < ± 0.25 ft	> ± 0.25 ft
Estimated regional hydraulic/tidal water level datum accuracy at gage reference PBM	± 0.05 ft to < ± 0.10 ft	> ± 0.10 ft to < ± 0.25 ft	> ± 0.25 ft
Uncertainty in 50-year subsidence forecast predictions (95%) in high subsidence areas	< ± 0.1 ft	> ± 0.1 ft to < ± 0.5 ft	> ± 0.5 ft
Uncertainty in 50-year sea level forecast predictions (95%)	< ± 0.1 ft	> ± 0.1 ft to < ± 0.5 ft	> ± 0.5 ft

9-9. Estimating Uncertainties on Coastal Navigation Project Grades. The design navigation grade or required dredging template needs to contain an allowance for uncertainties in the reference datum, tidal models, and survey accuracies. This allowance is dependent on a statistical analysis of the "total propagated uncertainty" (TPU) of individual depth measurements made by the acoustic measurement system, along with estimated hydrodynamic, meteorological, and environmental conditions occurring at a specific project site. Statistical uncertainties in the overall depth measurement process at a specific project site should be reviewed and evaluated during the PED phase. These will include local system variables (e.g., positional uncertainties, acoustic calibration precisions, vessel motion correction, acoustic depth resolution, sound velocity and outer beam refraction, etc.) and other systematic biases (tidal phase variations, tidal MLLW modeling variations, etc.) that may be present in the propagated depth error budget—TPU.

a. Indeterminate biases. Indeterminate biases include biases in tidal models, tidal epoch latencies, reference datum biases, tidal bench mark settlement, sea level change, acoustic bottom reflectivity, reference datum adjustments, geoid readjustments, and other largely indeterminate factors. These are biases that are difficult or nearly impossible to measure or correct for. They are generally not factored in dredge clearance assessment. This is because these biases are present in all repeated surveys over the project, assuming the same vertical reference tidal bench mark is used on a given project. They do, however, enter into the estimated uncertainty of a reported channel clearance to the public and cost estimates for dredging.

(1) For example, sea level rise occurring between tidal epoch updates could be as much as 0.2 ft. Thus, the MLLW datum at the reference bench mark would have a constant bias of 0.2 ft and the reported channel clearance constantly off by that same amount. This equates to overdredging the project by a constant 0.2 ft, which may have significant budget impacts.

(2) The use of outdated or undefined local reference datums will also cause systematic biases in the maintained or reported project depth. Datum biases of upwards of 2 ft have been known to occur, resulting in incorrectly reported or interpreted channel clearance depths.

(3) Tidal bench mark elevations used to reference measurement, payment, and clearance surveys at a project are also subject to uncertainties. The stability of the bench mark could be subject to regional settlement or uplift. The MLLW datum has an uncertainty dependent on the length of the time the gage was in place, the distance from a primary gage, and other factors. The uncertainty of the computed MLLW datum at a gage site can range from ± 0.1 ft to as much as ± 0.25 ft—see Chapter 4. It is also assumed that a primary reference bench mark is used to control all surveys performed at a given project site. If different bench marks are used, and inconsistencies between these bench marks exist (height or MLLW datum), then these errors would be propagated into the TPU estimates. An example would be uncertainties in a tidal zoning model.

(4) Tidal datum variations over a project may be subject to uncertainties if not minimized by some form of hydrodynamic modeling, such as those used in developing VDatum tidal datum fields.

(5) Geoid undulations occurring over a project must be modeled if RTK methods are used to measure the water surface elevation. Geoid model uncertainties in coastal areas are typically at the 1 to 3 cm range, with predicted uncertainties slightly larger (5 cm) in offshore entrance channels. There are no practical methods of refining the model in offshore models; however, since these errors are systematic to all users of the same model, survey repeatability (or more importantly, reproducibility) is not impacted.

(6) The accumulation of these global uncertainties can range from 0.1 to 0.5 ft. The addition of these global uncertainties can propagate to an overall uncertainty in the reported

project clearance. For example, a project with an estimated local survey confidence of ± 0.25 ft relative to a fixed bench mark/gage and an estimated global uncertainty of ± 0.25 ft would have an overall uncertainty of nearly ±0.4 ft. Given these uncertainties, reporting project clearances to an implied 0.1 ft confidence level is problematic.

b. Water surface correction uncertainty due to unmodeled tidal phase lags. Aside from vessel motion corrections (roll, pitch, yaw, heave), the largest portion of the depth error budget (TPU) is attributable to unmodeled tidal phase lags—i.e., surface slope gradients between the reference gage and the project site. This error is significant is tidal estuaries, rivers, or when inshore gage readings are extrapolated out into a coastal entrance channel—see Chapter 4. If RTK-derived water surface elevations are measured, coupled with GPS-aided IMU systems to correct vessel motions (e.g., POS/MV), then the uncertainty of the water surface elevation measurement at the project site may be estimated.

c. General measurement uncertainties. Uncertainty estimates in the design and maintenance of navigation grades in a typical navigation project of limited geographical extent are summarized in Table 9-5. This table differentiates between the survey procedures used to measure the water surface at the offshore project site—(1) unmodeled surface elevation extrapolation from a shore-based tide gage or (2) direct RTK surface elevation measurement at the project site. This table is not inclusive of all the measurement factors that make up a depth measurement—see the TPU factors in Figure 9-3.

Table 9-5. Estimated Uncertainties in Measuring Navigation Project Grades in a Typical Navigation Project.

Measurement Factor	Uncertainty Range
Tidal gage MLLW datum accuracy	0.1 - 0.2 ft
Tidal epoch latency (update lag during 19-year period)	0.05 - 0.1 ft
Projected gage/tidal PBM elevation (RTK):	
RTK geoid prediction	0.1 - 0.2 ft
RTK accuracy	0.1 - 0.15 ft
Projected gage/tidal PBM elevation: (extrapolated from gage to work site)	
MLLW range gradient (unmodeled/estimated)	0.1 - 0.3 ft
Tidal phase lag (gage to work site)	0.2 - 2 ft +
Acoustic depth measurement uncertainties:	
Depths < 15 ft	0.05 – 0.1 ft
Depths 15 ft to 40 ft	0.1 – 0.3 ft
Depths > 40 ft	0.3 – 0.5 ft

(1) The applicable uncertainties in Table 9-5 are statistically propagated to determine the resultant uncertainty of a depth measurement and uncertainty in the dredged clearance estimate.

(2) As an example, given a Gulf Coast 45-ft deep-draft navigation project located 5 miles distant from the reference tide gage. The reference gage datum computation was based on 90 days of observations 30 years ago. The tide readings at the gage are extrapolated out to the project site without any tide range or phase correction. The mean tide range is 8 ft at the offshore project site and 6 ft at the gage. The phase lag between the project site and gage is estimated at 45 minutes. The TPU of the measured grade would be estimated as follows:

Estimated Uncertainty Factor in TPU	Uncertainty in ± ft (95%)	
Tidal gage MLLW datum accuracy	0.3 ft	(Chapter 4)
Tidal epoch latency (update lag)	0.05 ft	(1993 to 2009)
Extrapolated (projected) surface from gage		
MLLW range gradient	0.2 ft	(unmodeled MLLW reference)
Tidal phase lag (average ebb/flood)	0.7 ft	(average random deviations)
Acoustic depth measurement	0.3 ft	(from above table)
Total Propagated Uncertainty:	0.8 ft	RMS (95%)

d. This implies that the uncertainty of the measured or cleared navigation grade is uncertain at the ± 0.8 ft (95%) confidence level. This uncertainty allowance should be factored in the tolerances used in the original studies that determine the authorized navigation depth for a project—see EM 1110-2-1613 (*Hydraulic Design of Deep-Draft Navigation Projects*). This uncertainty allowance (or tolerance) can also play in the evaluation of dredge clearance survey data and in the significant figure (rounding) resolution of recorded depths and clear grades. Figure 9-2 illustrates the uncertainty allowance estimate relative to (i.e., above and below) a nominal or required clearance grade. This uncertainty may or may not be significant on soft bottom maintenance dredging projects; however, on new work or rock-cut channels, this allowance may need to be applied to the overdepth allowance to provide additional confidence that the final channel clearance is to grade.

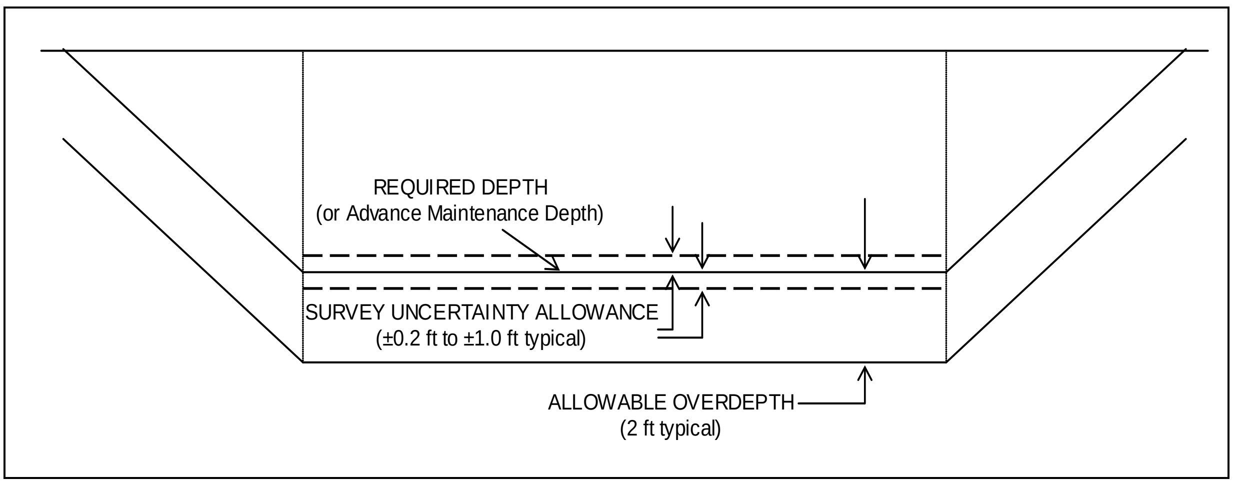


Figure 9-2. Propagated uncertainty allowance on a typical maintenance dredging template.

e. Approximate estimates of TPU in deep-draft navigation projects. Table 9-6 provides another example of general estimates for survey TPUs under nominal deep-draft project conditions, accounting for various measurement conditions largely dependent on the water surface measurement correction. These ranges may be used to estimate the TPU for a specific navigation project. Given the main variable in the table is dependent on the gage location relative to the project site (non-RTK measurements) the magnitude of this error needs to be estimated based on actual tidal range and phase conditions.

Table 9-6. Estimated TPU Allowances for Deep-Draft Navigation Projects.

Typical TPU	Water Surface Elevation Measurement Procedure	Tidal regime hydrodynamically modeled
Hard Bottom Materials		
± 0.20 foot	Determined from carrier phase GPS (RTK)	Yes
± 0.25 foot	Determined from carrier phase GPS (RTK)	No
± 0.20 foot	Estimated from gage less than 1 mile from project site	Yes
± 0.25 foot to ± 0.50 foot	Estimated from gage 1 to 5 miles from project site	No
± 0.50 foot to ± 1.0 foot	Estimated from gage > 5 miles from project site	No
± 0.50 foot to ± 2.0 foot	Estimated from gage > 10 miles from project site	No
Soft Bottom Materials (Maintenance Dredging)		
± 0.25 foot	Determined from carrier phase GPS (RTK)	Yes
± 0.25 foot to ± 1.0 foot	Estimated from gage 1 to 10 miles from project site	No
± 0.50 foot to ± 2.0 foot	Highly variable acoustic reflectivity due to suspended sediment, fluff, dense bottom vegetation, etc.	Yes

f. Methods for directly computing TPU of depth measurements. A more refined estimate of the TPU in measured depths (and clearance grades) in a navigation project may be computed using algorithms developed by the Canadian Hydrographic Service (CHS) for the US Naval Oceanographic Office—see "*Error Budget Analysis for US Naval Oceanographic Office (NAVOCEANO) Hydrographic Survey Systems: Final Report for Task 2, FY 01*" (NAVOCEANO/Hare 2001). A screen capture of a TPU calculator using these algorithms is shown in Figure 9-3. This TPU calculator provides user input of the estimated accuracies of over 50 parameters making up the total (propagated) depth error budget. It is applicable to either multibeam or single-beam hydrographic systems. This calculator compares the resultant TPU with both USACE EM 1110-2-1003 (*Hydrographic Surveying*) accuracy standards and International Hydrographic Organization "*Special Publication S-44*" (IHO 1998) accuracy standards. In addition, positional errors and target detection resolutions are estimated, as shown in the figure.

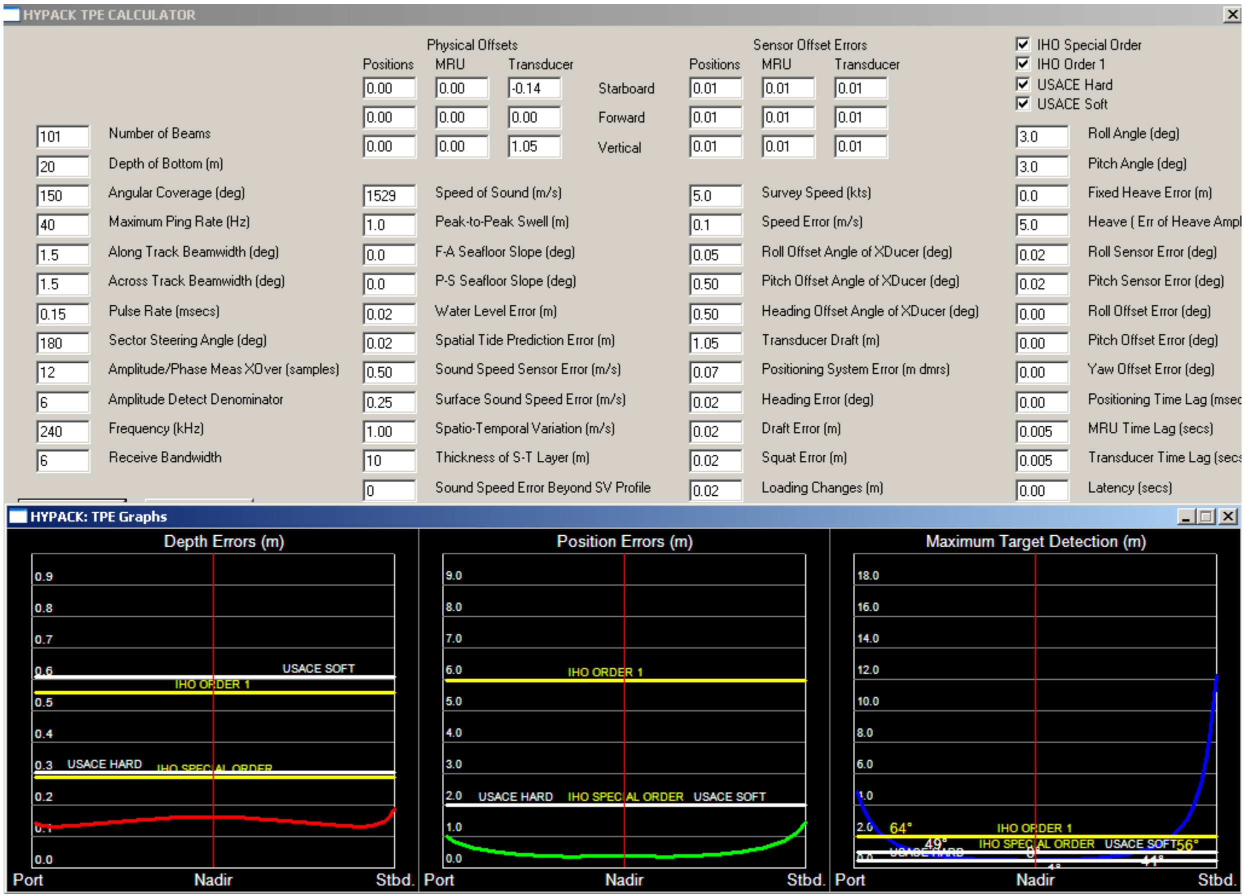


Figure 9-3. Total Propagated Uncertainty calculator for depth, position, and object detection. Values shown are for example only—users must insert estimated uncertainties for each parameter specific to their survey systems, procedures, and project. (HYPACK, Inc.)

APPENDIX A

References

A-1. Required References.

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Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums

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Survey Markers and Monumentation

EM 1110-1-1003
NAVSTAR Global Positioning System Surveying

EM 1110-1-1005
Control and Topographic Surveying

EM 1110-2-1003
Hydrographic Surveying.

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Part III: Height Systems, SaLIS Vol. 66, No. 2, pages 149–160, June 2006.
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A National Vertical Datum Transformation Tool, Parker, B., Hess, K. W., Milbert, D. G., and Gill, S., 2003; Sea Technology, Volume 44, Number 9, pp. 10-15, September 2003.

A-2. Related References.

ER 1110-1-8156
Policies, Guidance, and Requirements for Geospatial Data and Systems

ER 1110-2-100
Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures

ER 1130-2-520
Navigation and Dredging Operations and Maintenance Policies

ER 1130-2-530
Flood Control Operations & Maintenance Policies

EP 1130-2-520
Navigation and Dredging Operations and Maintenance Guidance and Procedures

EM 1110-1-2909
Geospatial Data and Systems

EM 1110-2-1416
River Hydraulics

EM 1110-2-1601
Hydraulic Design of Flood Control Channels

EM 1110-2-1607
Tidal Hydraulics

EM 1110-2-1614
Design of Coastal Revetments, Seawalls, and Bulkheads

EM 1110-2-1913
Design and Construction of Levees

APPENDIX B

Geodetic Reference Datums and Coordinate Systems

B-1. Purpose and Background. This Appendix provides general background information on geodetic reference datums, coordinate systems, and local horizontal reference systems that are used to georeference USACE civil works and military construction projects. The primary focus of this appendix is on geospatial reference systems that define horizontal locations on the Earth. The use of State Plane Coordinate Systems (SPCS) is covered in detail Section II since these systems are most commonly used to reference topographic site plan surveys of local projects. Transformations between geospatial datums and coordinate systems are also discussed. Vertical datums (i.e., orthometric, tidal, hydraulic) are not included here as they were covered in Chapter 2.

a. Most USACE site plan surveys for PED require “control surveys” to bring in a geodetic reference network to the local project site where detailed topographic surveys are performed. It is important that the correct geodetic reference network is used, and that it is consistent with the overall installation or project reference system. It is also important that these reference systems conform to the most up to date regional or nationwide reference systems (i.e., NAD83).

b. Other topographic surveys outside Army installations or Corps civil project areas may require rigid references to established property boundaries (corner pins, section corners, road intersections/centerlines, etc.). These ties to legal boundaries and corners will thus establish the reference system by which all topographic survey features are detailed. Regional geodetic networks may or may not be required on such surveys, depending on local practice or statute.

SECTION I

Geodetic Reference Systems

B-2. General. The discipline of surveying consists of locating points of interest on the surface of the earth. The positions of points of interest are defined by coordinate values that are referenced to a predefined mathematical surface. In geodetic surveying, this mathematical surface is called a datum, and the position of a point with respect to the datum is defined by its coordinates. The reference surface for a system of control points is specified by its position with respect to the earth and its size and shape. Control points are points with known relative positions tied together in a network. Densification of the network refers to adding more fixed control points to the network. Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network. Reference systems can be based on the geoid, ellipsoid, or a plane. The earth's gravitational force can be modeled to create a positioning reference frame that rotates with the earth. The geoid is such a surface (an equipotential surface of the earth's gravity field) that best approximates MSL. The orientation of this surface at a given point on geoid is defined by the plumb line. The plumb line is oriented tangent to the local gravity vector. Surveying instruments can be readily oriented with respect to the gravity field because its physical forces can be sensed with simple mechanical devices.

B-3. Geodetic Coordinates. A coordinate system is defined by the location of the origin, orientation of its axes, and the parameters (coordinate components) which define the position of a point within the coordinate system. Terrestrial coordinate systems are widely used to define the position of points on the terrain because they are fixed to the earth and rotate with it. The origin of terrestrial systems can be specified as either geocentric (i.e., origin at the center of the earth, such as NAD83) or topocentric (i.e., origin at a point on the surface of the earth, such as NAD27). The orientation of terrestrial coordinate systems is described with respect to its poles, planes, and axes.

a. Geocentric coordinates. Geocentric coordinates have an origin at the center of the earth, as shown in Figure B-1. GPS coordinates are initially observed on this type of reference system. For example, a coordinate on such a system might be displayed on a GPS receiver as:

X = 668400.506 m
Y = - 4929214.152 m
Z = 3978967.747 m

GPS receivers will transform these geocentric coordinates into a geographic coordinate system described below.

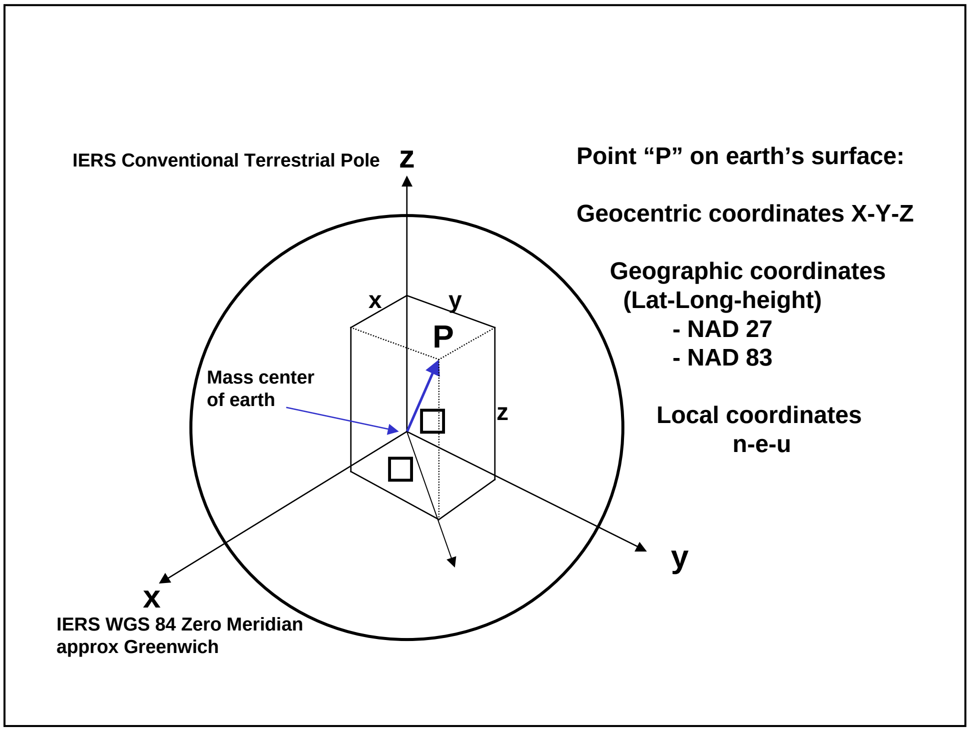

Figure B-1. Earth-centered earth-fixed coordinate reference frames.

b. Geodetic or Geographic coordinates. Geographic coordinate components consist of latitude (ϕ), longitude (λ), and ellipsoid height (h). Geodetic latitude, longitude, and ellipsoid height define the position of a point on the surface of the Earth with respect to some "reference ellipsoid." The most common reference ellipsoid used today is the WGS84, which will be described in more detail in a later section.

(1) Geodetic latitude (ϕ). The geodetic latitude of a point is the acute angular distance between the equatorial plane and the normal through the point on the ellipsoid measured in the meridian plane (Figure B-1). Geodetic latitude is positive north of the equator and negative south of the equator.

(2) Geodetic longitude (λ). The geodetic longitude is the angle measured counter-clockwise (east), in the equatorial plane, starting from the prime meridian (Greenwich meridian), to the meridian of the defined point (Figure B-1). In the continental United States, longitude is commonly reported as a west longitude. To convert easterly to westerly referenced longitudes, the easterly longitude must be subtracted from 360 degrees as shown below.

East-West Longitude Conversion

λ (W) = [360 - λ (E)]

For example:

λ (E) = 282 ^{d} 52 ^{m} 36.345 ^{s} E
λ (W) = [360 ^{d} - 282 ^{d} 52 ^{m} 36.345 ^{s} E]
λ (W) = 77 ^{d} 07 ^{m} 23.655 ^{s} W

(3) Ellipsoid Height (h). The ellipsoid height is the linear distance above the reference ellipsoid measured along the ellipsoidal normal to the point in question. The ellipsoid height is positive if the reference ellipsoid is below the topographic surface and negative if the ellipsoid is above the topographic surface.

(4) Geoid Separation (N). The geoid separation (or often termed "geoidal height") is the distance between the reference ellipsoid surface and the geoid surface measured along the ellipsoid normal. The geoid separation is positive if the geoid is above the ellipsoid and negative if the geoid is below the ellipsoid.

(5) Orthometric Height (H). The orthometric height is the vertical distance of a point above or below the geoid.

(6) The relationships between the ellipsoid height geoid height, and the orthometric height were illustrated in Chapter 2.

B-4. Datums. A datum is a coordinate surface used as reference for positioning control points. Both horizontal and vertical datums are commonly used in surveying and mapping to reference coordinates of points in a network.

a. Geodetic datum. Five parameters are required to define an ellipsoid-based datum. The semi-major axis (a) and flattening (f) define the size and shape of the reference ellipsoid; the latitude and longitude of an initial point; and a defined azimuth from the initial point define its orientation with respect to the earth. The NAD27 and NAD83 systems are examples of horizontal geodetic datums. Such a reference surface is developed from an ellipsoid of revolution that best approximates the geoid. An ellipsoid of revolution provides a well-defined mathematical surface to calculate geodetic distances, azimuths, and coordinates.

b. Horizontal datum. A horizontal datum is defined by specifying (1) the geometric surface (plane, ellipsoid, sphere) used in coordinate, distance, and directional calculations, (2) the initial reference point (origin), and (3) a defined orientation, azimuth or bearing from the initial point. The “horizontal datum” for most topographic surveys is usually defined relative to the fixed control points (monuments and/or bench marks) that were used to control the individual shots. These “control points” may, in turn, be referenced to a local installation/compound control network and/or to a national NSRS CORS station.

c. Project datum. A project datum is defined relative to local control and might not be directly referenced to a geodetic datum. Project datums are usually defined by a system with perpendicular axes, and with arbitrary coordinates for the initial point, and with one (principal) axis oriented toward an assumed north. A chainage-offset system may also be used as a reference, with the PIs (points of intersection) either marked points or referenced to some other coordinate system.

d. Vertical datum. A vertical datum is a reference system used for reporting elevations. The two most common nationwide systems are the NGVD29 and the NAVD88. See Chapter 2 for details on these orthometric datums.

e. The National Spatial Reference System (NSRS). The NSRS is that component of the National Spatial Data Infrastructure (NSDI) that contains all geodetic control contained in the NGS database. (See Chapter 2 for details on the NSRS).

B-5. WGS84 Reference Ellipsoid. The GPS satellites are referenced to the WGS84 ellipsoid. The origin of the WGS84 Cartesian system is the earth's center of mass, as shown in Figure B-1. The Z-axis is parallel to the direction of the Conventional Terrestrial Pole (CTP) for polar motion, as defined by the Bureau International Heure (BIH), and equal to the rotation axis of the WGS84 ellipsoid. The X-axis is the intersection of the WGS84 reference meridian plane and the CTP's equator, the reference meridian being parallel to the zero meridian defined by the BIH and equal to the X-axis of the WGS84 ellipsoid. The Y-axis completes a right-handed, earth-centered, earth-fixed orthogonal coordinate system, measured in the plane of the CTP equator 90 degrees east of the X-axis and equal to the Y-axis of the WGS84 ellipsoid. The DOD continuously monitors the origin, scale, and orientation of the WGS84 reference frame and references satellite orbit coordinates to this frame. Updates are shown as WGS84 (GXXX), where "XXX" refers to a GPS week number starting on 29 September 1996.

a. It is a common misconception that the resultant position of a GPS survey is referenced to WGS84. While this would be the case if we were using GPS in an absolute mode (no reference/base station), in the differential GPS mode, the geospatial coordinates have been shifted from the WGS84 ellipsoid to the GRS80 ellipsoid when the reference receiver is using NAD83 coordinates.

b. Over the years there have been several reference ellipsoids and interrelated coordinate systems (datums) that were used by the surveying and mapping community. Table B-1 lists just a few of these reference systems along with their mathematical defining parameters. Note that GRS80 is the actual reference ellipsoid for NAD83; however, the difference in the axis between GRS80 and WGS84 ellipsoids is insignificant but the origins differ by over 2 meters. Transformation techniques are used to convert between different datums and coordinate systems. Most GPS software has built in transformation algorithms for the more common datums.

Table B-1. Reference Ellipsoids and Related Coordinate Systems.

Reference Ellipsoid	Coordinate System (Datum/Frame)	Semimajor axis (meters)	Shape (1/flattening)
Clarke 1866	NAD27	6378206.4	1/294.9786982
WGS72	WGS72	6378135	1/298.26
GRS80	NAD83 (XX)	6378137	1/298.257222101
WGS84	WGS84 (GXXX)	6378137	1/298.257223563
ITRS	ITRF (XX)	6378136.49	1/298.25645

SECTION II

Horizontal Coordinate Systems

B-6. General. Geocentric, geographic, or geodetic coordinates described above are rarely used to reference site plan topographic surveys or maps. Engineering site plan drawings are normally referenced to a local SPCS, or in some cases, a metric-based UTM system. They may also be referenced to an arbitrary coordinate system relative to some point on the project--a monument, property corner, road intersection, etc. Most construction drawings also contain "chainage-offset" (stationing) reference systems. In most cases, control surveys performed for setting project control will be computed and adjusted using the SPCS. The following paragraphs describe horizontal coordinate systems commonly used on facility site plan mapping and related control surveys.

B-7. Geographic coordinates. The use of geographic coordinates as a system of reference is accepted worldwide. It is based on the expression of position by latitude (parallels) and longitude (meridians) in terms of arc (degrees, minutes, and seconds) referred to the equator (north and south) and a prime meridian (east and west). The degree of accuracy of a geographic reference (GEOREF) is influenced by the map scale and the accuracy requirements for plotting and scaling. Examples of GEOREFs are as follows:

40° N 132° E (referenced to degrees of latitude and longitude).
40°21' N 132°14' E (referenced to minutes of latitude and longitude).
40°21'12" N 132°14'18" E (referenced to seconds of latitude and longitude).
40°21'12.4" N 132°14'17.7" E (referenced to tenths of seconds of latitude and longitude).
40°21'12.45" N 132°14'17.73" E (referenced to hundredths of seconds of latitude and longitude).

US military maps and charts include a graticule (parallels and meridians) for plotting and scaling geographic coordinates. Graticule values are shown in the map margin. On maps and charts at scales of 1:250,000 and larger, the graticule may be indicated in the map interior by lines or ticks at prescribed intervals (for example, scale ticks and interval labeling at the corners of 1:50,000 at 1minute [in degrees, minutes, and seconds] and again every 5 minutes).

B-8. Horizontal Datums and Reference Frames. The following paragraphs briefly describe the most common datums used to reference CONUS projects.

a. North American Datum of 1927 (NAD27). NAD27 is a horizontal datum based on a comprehensive adjustment of a national network of traverse and triangulation stations. NAD27 is a best fit for the continental United States. The fixed datum reference point is located at Meades Ranch, Kansas. The longitude origin of NAD27 is the Greenwich Meridian with a south azimuth orientation. The original network adjustment used 25,000 stations. The relative precision between initial point monuments of NAD27 is by definition 1:100,000, but coordinates on any given monument in the network contain errors of varying degrees. As a result, relative accuracies between points on NAD27 may be far less than the nominal 1:100,000. The reference units for NAD27 are US Survey Feet. This datum is no longer supported by NGS, and USACE commands have been gradually transforming their project coordinates over to the NAD83 described below. Approximate conversions of points on NAD27 to NAD83 may be performed using CORPSCON, a transformation program developed by ERDC/TEC. Since NAD27 contains errors approaching 10 m, transforming highly accurate GPS observations to this antiquated reference system is not the best approach.

b. North American Datum of 1983 (NAD83). The nationwide horizontal reference network was redefined in 1983 and readjusted in 1986 by the NGS. It is known as the North American Datum of 1983, Adjustment of 1986, and is referred to as NAD83 (1986). (Subsequent adjustments have been made). NAD83 used far more stations (250,000) and observations than did NAD27, including a few satellite-derived coordinates, to readjust the national network. The longitude origin of NAD83 is the Greenwich Meridian with a north azimuth orientation. The fixed adjustment of NAD83 (1986) has an average precision of 1:300,000. NAD83 is based upon GRS80, an earth-centered reference ellipsoid which for most, but not all, practical purposes is equivalent to WGS84. With increasingly more accurate uses of GPS, the errors and misalignments in NAD83 (1986) became more obvious (they approached 1 meter), and subsequent refinements outlined below have been made to correct these inconsistencies.

c. High Accuracy Reference Networks (HARN). (Figure B-2). Within a few years after 1986, more refined GPS measurements had allowed geodesists to locate the earth's center of mass with a precision of a few centimeters. In doing so, these technologies revealed that the center of mass that was adopted for NAD83 (1986) is displaced by about 2 m from the true geocenter. These discrepancies caused significant concern as the use of highly accurate GPS measurements proliferated. Starting with Tennessee in 1989, each state--in collaboration with NGS and various other institutions--used GPS technology to establish regional reference frames that were to be consistent with NAD83. The corresponding networks of GPS control points were originally called High Precision Geodetic Networks (HPGN). Currently, they are referred to as High Accuracy Reference Networks (HARN). This latter name reflects the fact that relative accuracies among HARN control points are better than 1 ppm, whereas relative accuracies among pre-existing control points were nominally only 10 ppm. Positional differences between NAD83 (1986) and NAD83 (HARN) can approach 1 meter.

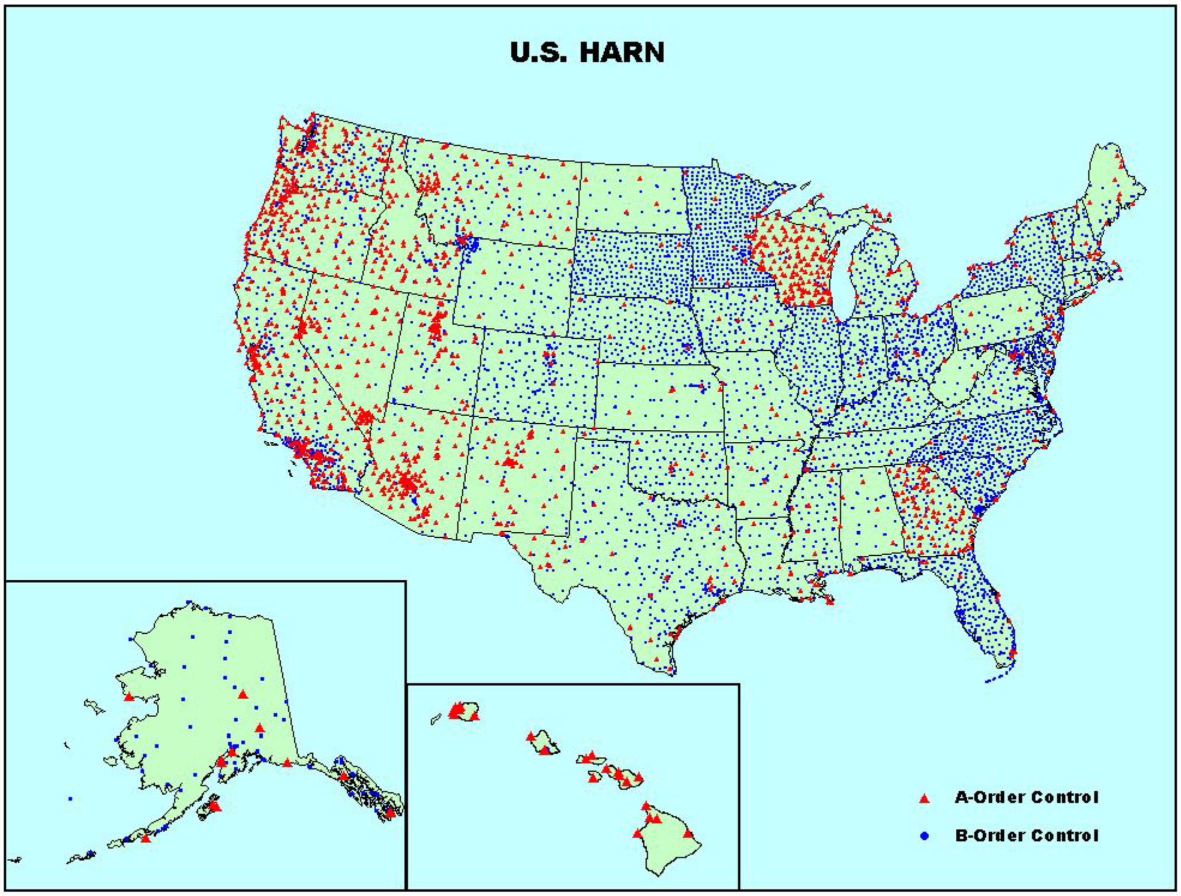

Figure B-2. High Accuracy Reference Network control points.

d. Continuously Operating Reference Stations (CORS). The regional HARNs were subsequently further refined (or "realized") by NGS into a network of Continuously Operating Reference Stations, or CORS. This CORS network was additionally incorporated with the International Terrestrial Reference System (ITRS), i.e. the ITRF. CORS are located at fixed points throughout CONUS and at some OCONUS points--see Figure B-3. This network of high-accuracy points can provide GPS users with centimeter level accuracy where adequate CORS coverage exists. Coordinates of CORS stations are designated by the year of the reference frame, e.g., NAD83 (CORS 96). Positional differences between NAD83 (HARN) and NAD83 (CORS) are less than 10 cm. More importantly, positional difference between two NAD83 (CORSxx) points is typically less than 2 cm. Thus, GPS connections to CORS stations will be of the highest order of accuracy. USACE commands can easily connect and adjust GPS-observed points directly with CORS stations using a number of methods, including the NGS on-line program OPUS (see Chapter 3 and EM 1110-1-1003). CORS are particularly useful when precise control is required in a remote area, from which a topographic survey may be performed. With only 1 to 2 hours of static DGPS observations, reference points can often be established to an ellipsoid accuracy better than ± 0.25 ft in X-Y-Z.

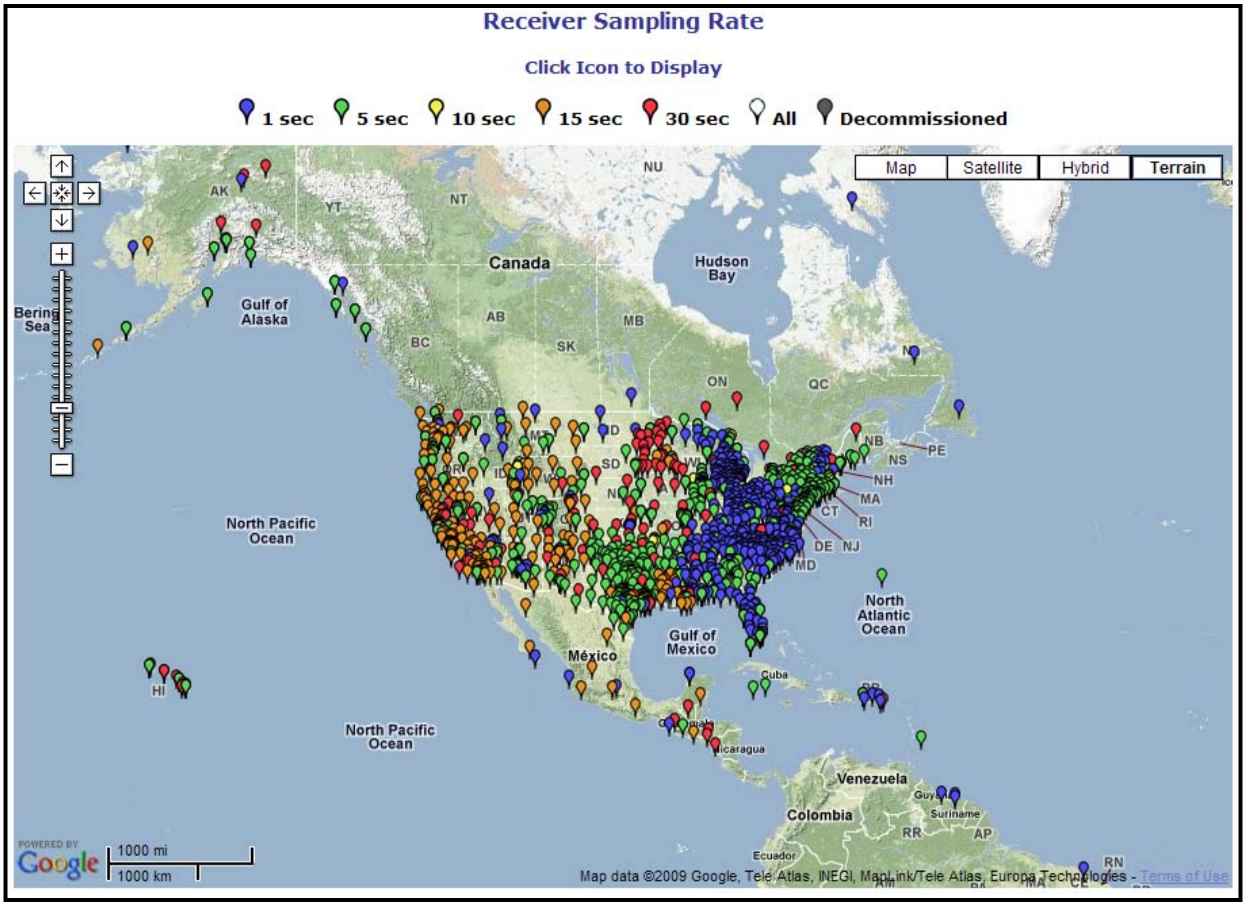

Figure B-3. Continuously Operating Reference Stations as of 2010. (NGS)

e. 2007 National Readjustment. (See Figure B-4). A readjustment of NSRS was completed in 2007 by the NGS. The adjustment was undertaken to resolve inconsistencies between the existing statewide HARNs, the Federal Base Network (FBN) adjustments, and the nationwide CORS system, as well as between states. Individual local and network accuracy estimates were also derived from this effort. This readjustment includes ~70,000 passive geodetic control monuments constrained to the NAD83 (CORS96) realization. NAD83 (NSRS2007) was created by adjusting GPS data collected during various geodetic surveys performed between the mid-1980´s and 2005. For this adjustment NAD83 (CORS96) positional coordinates for ~700 CORS were held fixed. The CORS 2002 epoch was used for all states except AZ, OR, WA, CA, and AK where an epoch of 2007 was used. Derived NAD83 (NSRS2007) positional coordinates should be consistent with corresponding NAD83 (CORS96) positional coordinates to within the accuracy of the GPS data used in the adjustment.

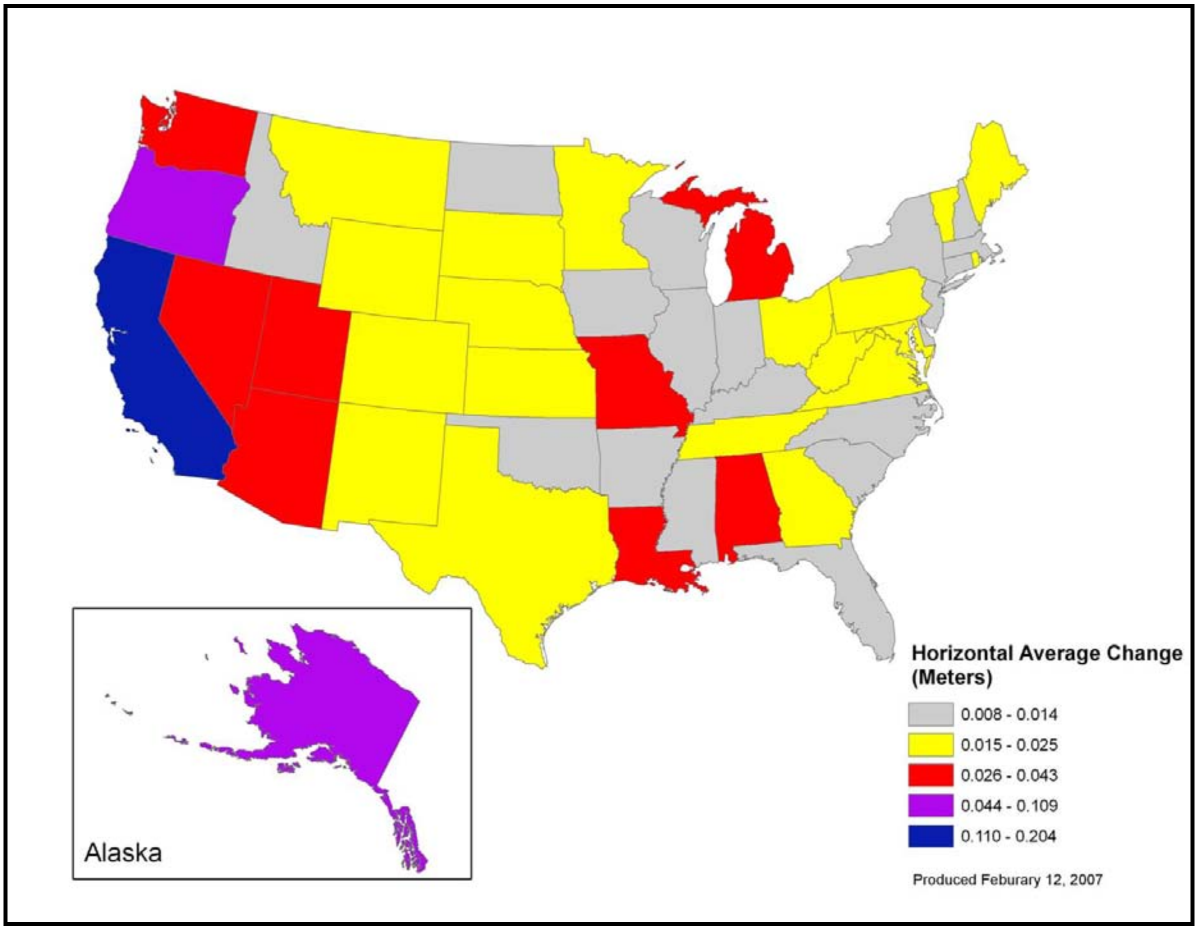

Figure B-4. Horizontal coordinate shifts between NAD83 and NAD83 in meters (NSRS 2007).

f. International Terrestrial Reference Frame (ITRF). The ITRF is a highly accurate geocentric reference frame with an origin at the mass center of the earth. The ITRF is continuously monitored and updated by the International Earth Rotation Service (IERS) using very-long-baseline-interferometry (VLBI) and other techniques. These observations allow for the determination of small movements of fixed points on the earth's surface due to crustal motion, rotational variances, tectonic plate movement, etc. These movements can average 10 to 20 mm/year in CONUS, and may become significant when geodetic control is established from remote reference stations. These refinements can be used to accurately determine GPS positions observed on the basic WGS84 reference frame. NAD83 coordinates are defined based on the ITRF year/epoch in which it is defined, e.g., ITRF 89, ITRF 96, ITRF 2000. For highly accurate positioning where plate velocities may be significant, users should use the same coordinate reference frame and epoch for both the satellite orbits and the terrestrial reference frame. USACE requirements for these precisions on control surveys would be rare, and would never be applicable to local facility mapping surveys. Those obtaining coordinates from NGS datasheets must take care not to use ITRF values. The relationship between ITRF, NAD83, and the geoid is illustrated in Figure B-5.

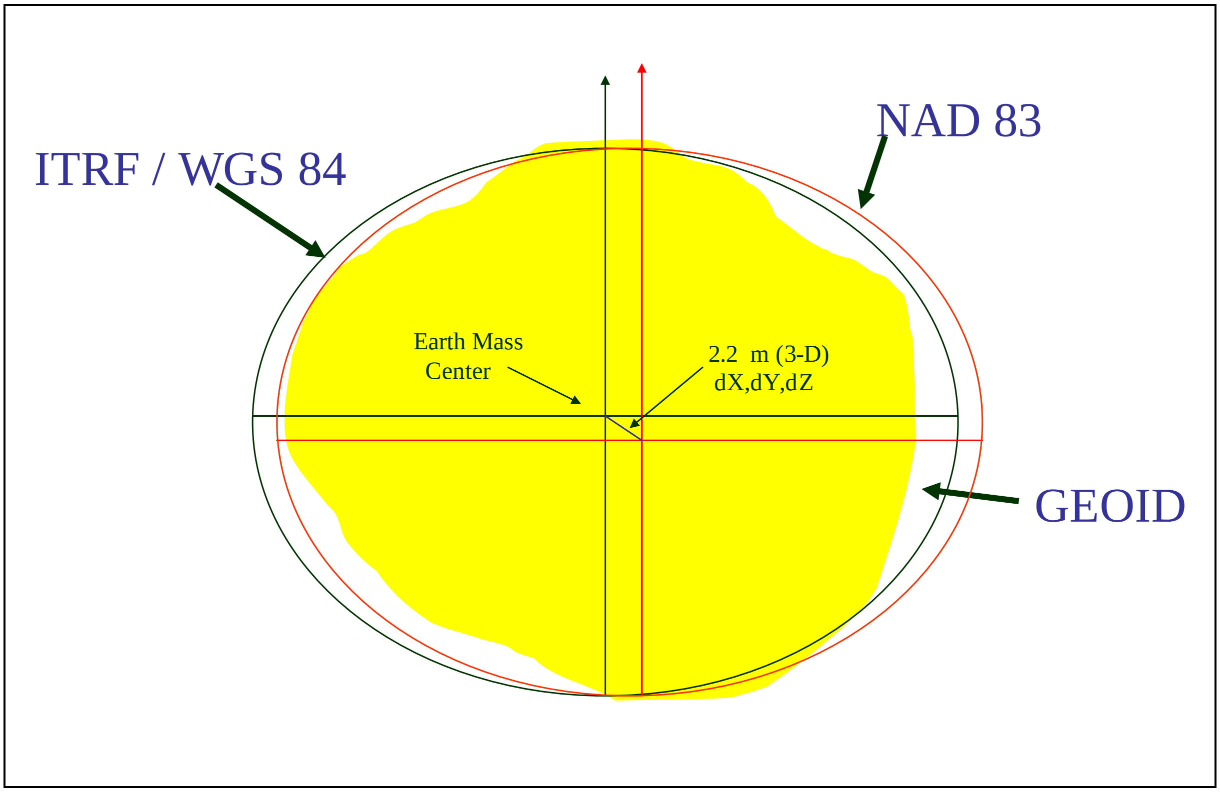

Figure B-5. Relationship between ITRF, NAD83, and the geoid.

B-9. <u>State Plane Coordinate Systems</u>.

a. General. State Plane Coordinate Systems (SPCS) were developed by the NGS to provide plane coordinates over a limited region of the earth's surface. To properly relate geodetic coordinates (ϕ-λ-h) of a point to a 2D plane coordinate representation (Northing, Easting), a conformal mapping projection must be used. Conformal projections have mathematical properties that preserve differentially small shapes and angular relationships to minimize the errors in the transformation from the ellipsoid to the mapping plane. Map projections that are most commonly used for large regions are based on either a conic or a cylindrical mapping surface (Figure B-6). The projection of choice is dependent on the north-south or east-west areal extent of the region. Typically states with limited east-west dimensions and indefinite north-south extent use the Transverse Mercator (TM) type projection while states with limited north-south dimensions and indefinite east-west extent use the Lambert projection. The SPCS is designed to minimize the spatial distortion at a given point to approximately one part in ten thousand (1:10,000). To satisfy this criterion, the SPCS has been divided into zones that have a maximum width or height of approximately one hundred and fifty eight statute miles (158 miles). Therefore, each state may have several zones and/or may employ both the Lambert (conic) and Transverse Mercator (cylindrical) projections. The projection state plane coordinates are referenced to a specific geodetic datum (i.e. the datum that the initial geodetic coordinates are referenced to must be known).

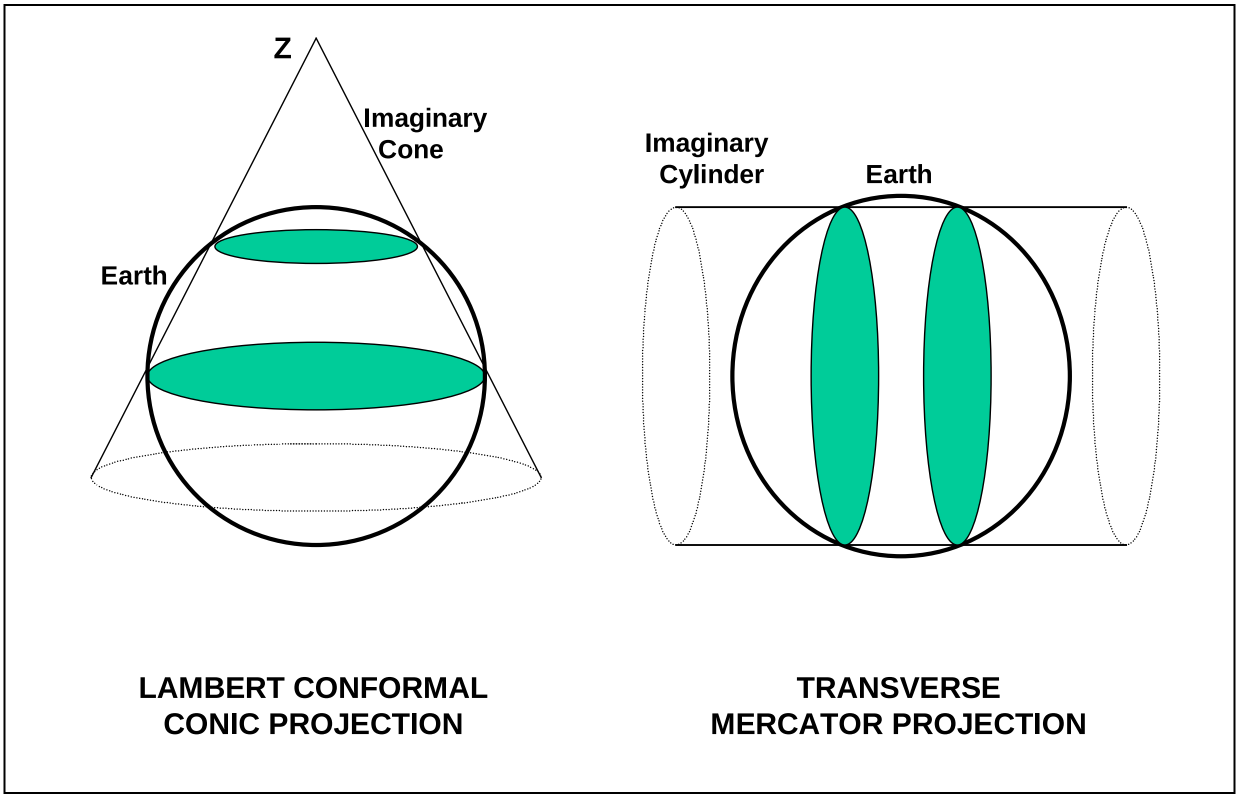

Figure B-6. Common map projections.

b. Transverse Mercator (TM). The Transverse Mercator projection uses a cylindrical surface to cover limited zones on either side of a central reference longitude. Its primary axis is rotated perpendicular to the symmetry axis of the reference ellipsoid. Thus, the TM projection surface intersects the ellipsoid along two lines equidistant from the designated central meridian longitude (Figure B-7). Distortions in the TM projection increase predominantly in the east-west direction. The scale factor for the Transverse Mercator projection is 1.0000 where the cylinder intersects the ellipsoid. The scale factor is less than one between the lines of intersection, and greater than one outside the lines of intersection. The scale factor is the ratio of arc length on the projection to arc length on the ellipsoid. To compute the state plane coordinates of a point, the latitude and longitude of the point and the projection parameters for a particular TM zone or state must be known.

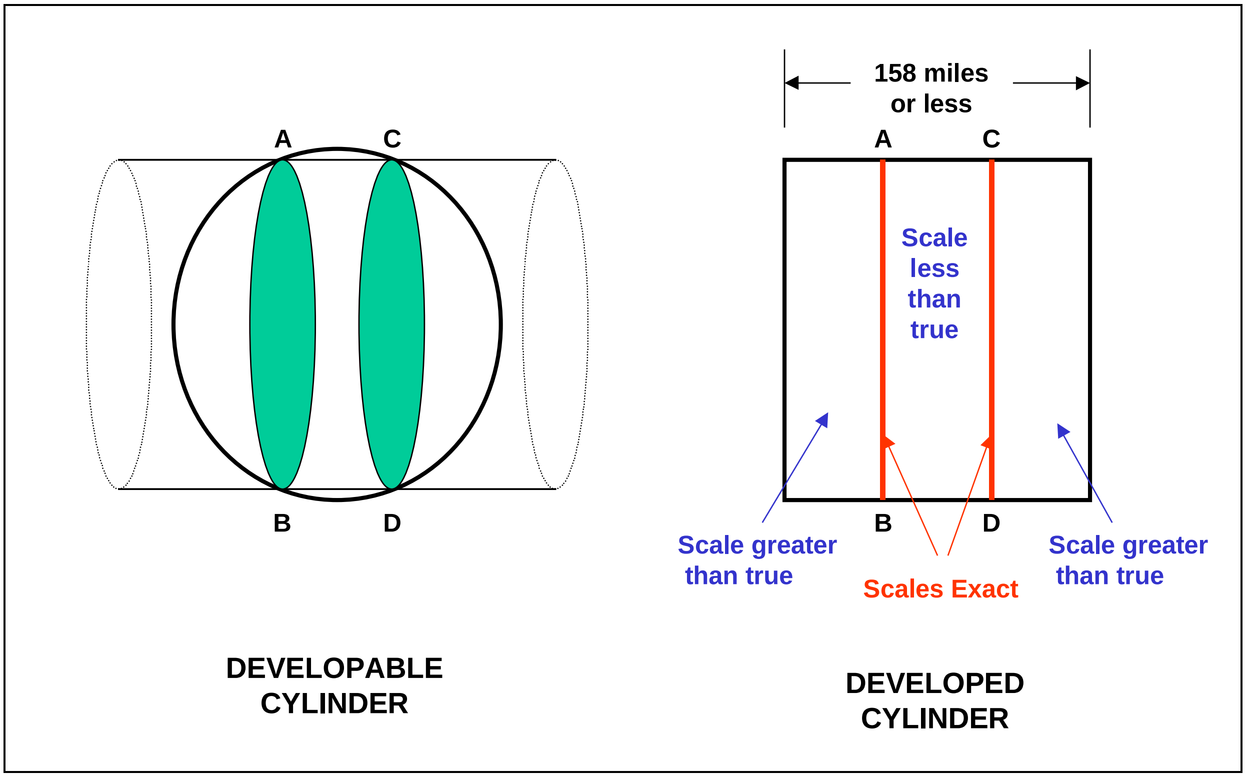

Figure B-7. Transverse Mercator Projection.

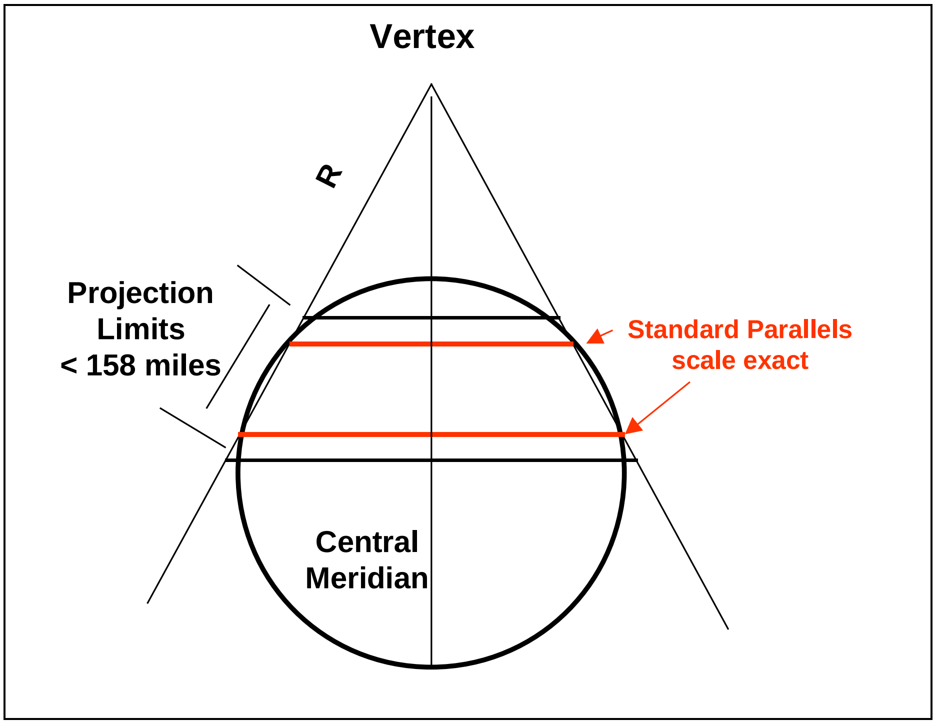

Figure B-8. Lambert Conformal Conic Projection.

c. Lambert Conformal Conic (LCC). The Lambert projection uses a conic surface to cover limited zones of latitude adjacent to two parallels of latitude. Its primary axis is coincident with the symmetry axis of the reference ellipsoid. Thus, the LCC projection intersects the ellipsoid along two standard parallels (Figure B-8). Distortions in the LCC projection increase predominantly in the north-south direction. The scale factor for the Lambert projection is equal to 1.0000 at each standard parallel and is less than one inside, and greater than one outside the standard parallels. The scale factor is the ratio of arc length on the projection to arc length on the ellipsoid and remains constant along the standard parallels.

d. SPCS zones. Figure B-9 depicts the various SPCS zones in the US. The unique state zone number provides a standard reference when using transformation software developed by NGS and USACE. The state zone number remains constant in both NAD27 and NAD83 coordinate systems. There have been some changes in the number of zones in a few of the states, for example, California dropped zone 0407 which is now included in zone 0405 and Montana went from three zones to one.

Figure B-9. SPCS zones identification numbers for the various states.

e. Scale units. State plane coordinates can be expressed in both feet and meters. State plane coordinates defined on the NAD27 datum are published in feet. State plane coordinates defined on the NAD83 datum are published in meters; however, state and federal agencies can request the NGS to provide coordinates in feet. If NAD83 based state plane coordinates are defined in meters and the user intends to convert those values to feet, the proper meter-feet conversion factor (shown below) must be used. Some states use the International Survey Foot rather than the US Survey Foot in the conversion of feet to meters (see Figure B-10).

International Survey Foot:	*1 International Foot = 0.3048 meter (exact)*
US Survey Foot:	*1 US Survey Foot = 1200 / 3937 meter (exact)*

The use of the incorrect conversion factor can lead to significant errors in the resultant coordinates.

Figure B-10. English-metric conversions in the various states (NGS).

B-10. Grid Elevations, Scale Factors, and Convergence. In all planer grid systems, the grid projection only approximates the ellipsoid (or roughly the ground), and "ground-grid" corrections must be made for measured distances or angles (directions). Measured ground distances must be corrected for (1) elevation (sea level factor), and (2) ground to grid plane (scale factor). Figure B-11 illustrates a reduction of a measured distance (D) down to the ellipsoid distance (S). Not shown is the subsequent reduction from the ellipsoid length to a grid system length. Observed directions (or angles) must also be corrected for grid convergence. Also shown on the figure is the relationship between ellipsoid heights (h), geoid heights (N), and orthometric heights (H).

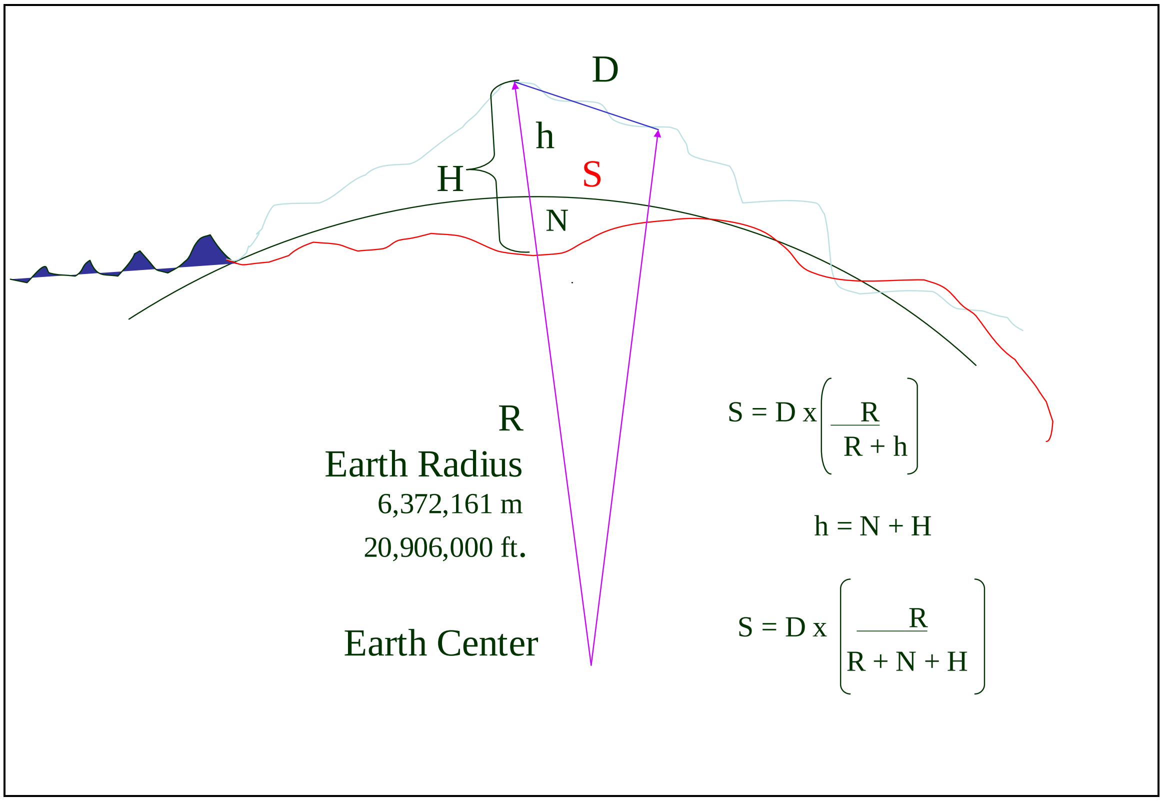

Figure B-11. Reduction of measured slope distance D to ellipsoid distance S (NGS).

a. Grid factor. For most topographic surveys covering a small geographical site, these two factors can be combined into a constant “grid factor” or "combined scale factor."

> *Grid factor = Sea Level Factor x Scale Factor*
>
> then: *Ground Distance = Grid Distance / Grid Factor*
>
> or
>
> *Grid Distance = Ground Distance x Grid Factor*

b. Convergence. Between two fixed points, the geodetic azimuth will differ from the grid azimuth. This difference is known as “convergence” and varies with the distance from the central meridian of the projection. Thus, if a geodetic azimuth is given between two fixed points (inversed from published geographic coordinates, astronomic, or GPS), then it must be corrected for convergence to obtain an equivalent grid azimuth. If lengthy control traverses are being computed on a SPCS or UTM grid, then additional second term corrections to observed angles may be required--e.g., the ”t-T” correction used in older survey manuals.

c. Use of data collectors. The above grid corrections should rarely have to be performed when modern survey data collectors are being used. These total station or RTK data collectors (with full COGO and adjustment capabilities) will automatically perform all the necessary geographic to grid coordinate translations, including sea level reductions and local grid system conversions that are later transformed and adjusted into an established SPCS grid at a true elevation.

d. References. Many DA publications (i.e., Field Manuals) and surveying textbooks contain information, procedures, and examples of these grid transforms.

B-11. Universal Transverse Mercator Coordinate System. Universal Transverse Mercator (UTM) coordinates are used in surveying and mapping when the size of the project extends through several state plane zones or projections. UTM coordinates are also utilized by the DOD for tactical mapping, charting, and geodetic applications. It may also be used to reference site plan engineering surveys if so requested in CONUS or OCONUS installations. The UTM projection differs from the TM projection in the scale at the central meridian, origin, and unit representation. The scale at the central meridian of the UTM projection is 0.9996. In the Northern Hemisphere, the northing coordinate has an origin of zero at the equator. In the Southern Hemisphere, the southing coordinate has an origin of 10,000,000 m. The easting coordinate has an origin of 00,000 m) at the central meridian. The UTM system is divided into 60 longitudinal zones. Each zone is 6 degrees in width extending 3 degrees on each side of the central meridian. UTM coordinates are always expressed in meters. USACE program CORPSCON can be used to transform coordinates between UTM and SPCS systems. Additional details on UTM grids and survey computations thereon may be found in DA publications.

B-12. The US Military Grid-Reference System (FM 3-34.331). The US Military Grid-Reference System (MGRS) is designed for use with UTM grids. For convenience, the earth is generally divided into 6° by 8° geographic areas, each of which is given a unique grid-zone designation. These areas are covered by a pattern of 100,000-meter squares. Two letters (called the 100,000-meter-square letter identification) identify each square. This identification is unique within the area covered by the grid-zone designation.

a. The MGRS is an alphanumeric version of a numerical UTM grid coordinate. Thus, for that portion of the world where the UTM grid is specified (80° south to 84° north), the UTM grid-zone number is the first element of a military grid reference. This number sets the zone longitude limits. The next element is a letter that designates a latitude bond. Beginning at 80° south and proceeding northward, 20 bands are lettered C through X. In the UTM portion of the MGRS, the first three characters designate one of the areas within the zone dimensions.

b. A reference that is keyed to a gridded map (of any scale) is made by giving the 100,000-meter-square letter identification together with the numerical location. Numerical references within the 100,000-meter square are given to the desired accuracy in terms of the easting and northing grid coordinates for the point.

c. The final MGRS position coordinate consists of a group of letters and numbers that include the following elements:

(1) The grid-zone designation.

(2) The 100,000-meter-square letter identification.

(3) The grid coordinates (also referred to as rectangular coordinates) of the numerical portion of the reference, expressed to a desired refinement.

(4) The reference is written as an entity without spaces, parentheses, dashes, or decimal points.

d. Examples of MGRS coordinates are as follows:

18S (locating a point within the grid-zone designation).
18SUU (locating a point within a 100,000-meter square).
18SUU80 (locating a point within a 10,000-meter square).
18SUU8401 (locating a point within a 1,000-meter square).
18SUU836014 (locating a point within a 100-meter square).

e. To satisfy special needs, a reference can be given to a 10-meter square and a 1-meter square, as shown below.

18SUU83630143 (locating a point within a 10-meter square).
18SUU8362601432 (locating a point within a 1-meter square).

f. There is no zone number in the polar regions. A single letter designates the semicircular area and the hemisphere. The letters A, B, Y, and Z are used only in the polar regions. An effort is being made to reduce the complexity of grid reference systems by standardizing a single, worldwide grid reference system.

B-13. US National Grid System. A US National Grid (USNG) system has been developed to improve public safety, commerce, and aid the casual GPS user with an easy to use geoaddress system for identifying and determining location with the help of a USNG gridded map and/or a USNG enabled GPS system. The USNG can provide for whatever level of precision is desired. Many users may prefer to continue using the UTM format for applications requiring precision greater than 1 meter.

a. Grid Zone Designation (GZD). The US geographic area is divided into 6-degree longitudinal zones designated by a number and 8-degree latitudinal bands designated by a letter. Each area is given a unique alphanumeric Grid Zone Designator--e.g., 18S.

b. 100,000-meter square identification. Each GZD 6x8 degree area is covered by a specific scheme of 100,000-meter squares where each square is identified by two unique letters--e.g., 18SUJ identifies a specific 100,000-meter square in the specified GZD.

c. Grid coordinates. A point position within the 100,000-meter square shall be given by the UTM grid coordinates in terms of its Easting (E) and Northing (N). An equal number of digits shall be used for E and N where the number of digits depends on the precision desired in position referencing. In this convention, the reading shall be from left with Easting first and then Northing, for example:

18SUJ20 - Locates a point with a precision of 10 km
18SUJ2306 - Locates a point with a precision of 1 km
18SUJ234064 - Locates a point with a precision of 100 meters
18SUJ23480647 - Locates a point with a precision of 10 meters
18SUJ2348306479 - Locates a point with a precision of 1 meter

The number of digits in Easting and Northing can vary, depending on specific requirements or application.

B-14. Chainage-Offset Coordinate Systems. Most linear engineering and construction projects (roads, railways, canals, navigation channels, levees, floodwalls, beach renourishment, etc.) are locally referenced using the traditional engineering chainage-offset system (Figure B-12). Usually, SPCS coordinates are provided at the PIs, from which, given the alignment between PIs, a SPCS coordinate can then be computed for any given station-offset point. Chainage-offset systems are used for locating cross-sections along even centerline stations. Topographic elevation and feature data is then collected along each section relative to the centerline. Likewise, road, canal, and levee alignments can be staked out relative to station-offset parameters, and internally in a total station or RTK system data collector, these offsets may actually be transformed from a SPCS.

a. Stationing. Alignment stationing (or chainage) zero references are arbitrarily established for a given project or sectional area. For example, stationing on a navigation project usually commences offshore on coastal projects and runs inland or upstream. Stationing follows the channel centerline alignment. Stationing may be accumulated through each PI or zero out at each PI or new channel reach. Separate stationing is established for widener sections, turning basins, levees, floodwalls, etc. Each district may have its own convention. Stationing coordinates use "+" signs to separate the second- and third-place units (XX + XX.XX). Metric chainage often separates the third and fourth places (XXX + XXX.XX) to distinguish the units from English feet; however, some districts use this convention for English stationing units.

b. Offsets. Offset coordinates are distances from the centerline alignment of a road, levee, or navigation channel. Offsets carry plus/minus coordinate values. Normally, offsets are positive to the right (looking toward increasing stationing). Some USACE Districts designate cardinal compass points (east-west or north-south) in lieu of a coordinate sign. On some navigation projects, the offset coordinate is termed a "range," and is defined relative to the project centerline or, in some instances, the channel-slope intersection line (toe). Channel or canal offsets may be defined relative to a fixed baseline on the bank or levee.

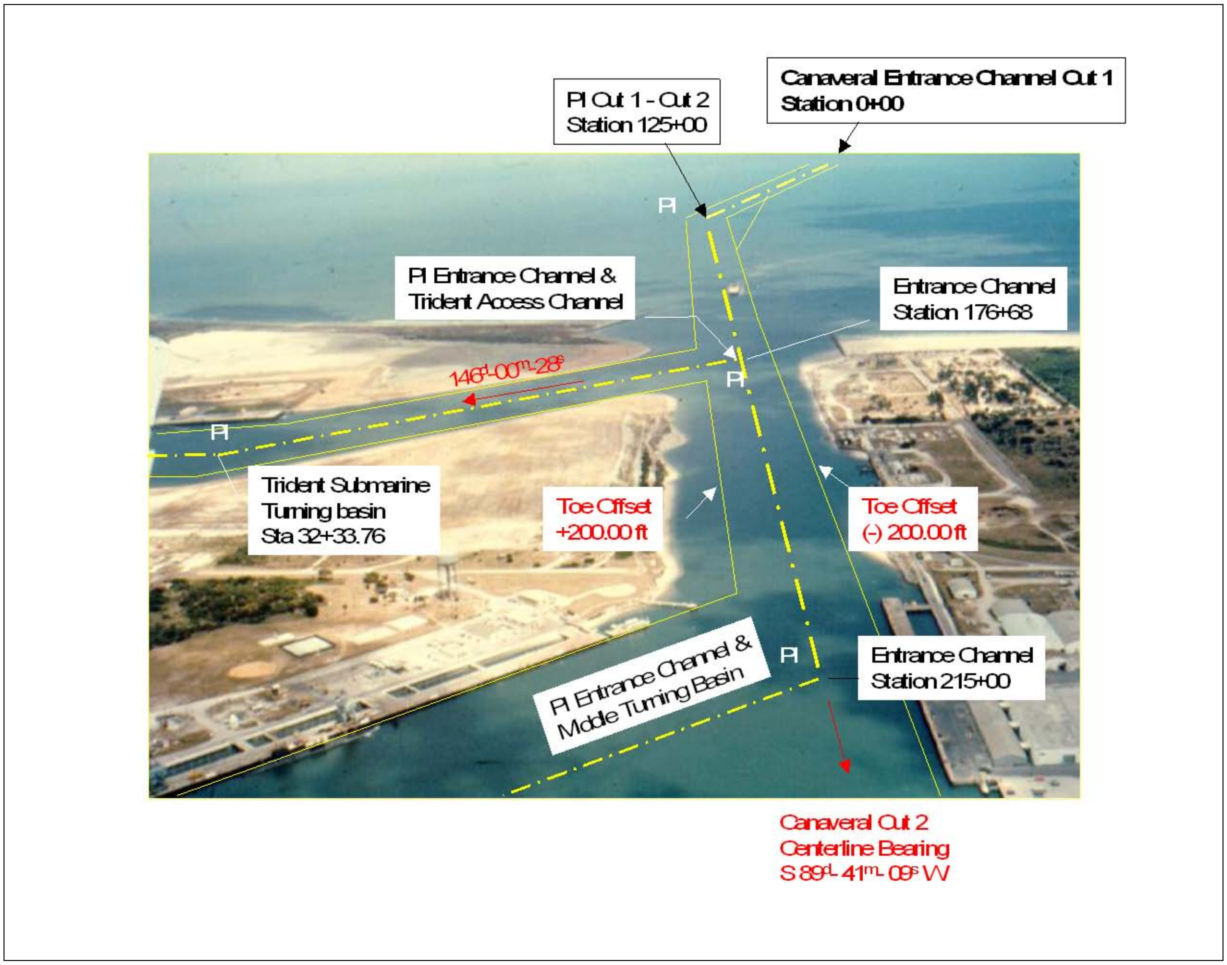

Figure B-12. Chainage-offset project control scheme for a typical deep-draft navigation project--Cape Canaveral, FL. (Jacksonville District)

c. Azimuth. Azimuths are computed relative to the two defining PIs. Either 360-deg azimuth or bearing designations may be used. Azimuths should be shown to the nearest second.

d. Other local alignments. Different station-offset reference grids may be established for individual portions of a project. River sections and coastal beach sections are often aligned perpendicular to the project/coast. Each of these sections is basically a separate local datum with a different reference point and azimuth alignment. Beach sections may also be referenced to an established coastal construction setback line. Circular and transition (spiral) curve alignments are also found in some rivers, canals, and flood risk management projects such as spillways and levees. Surveys will generally be aligned to the chainage and offsets along such curves. Along inland waterways, such as the Mississippi River, stationing is often referenced to either arbitrary or monumented baselines along the bank. In many instances, a reference baseline for a levee is used, and surveys for revetment design and construction are performed from offsets to this line. Separate baselines may exist over the same section of river, often from levees on opposite banks or as the result of revised river flow alignments. Baseline stationing may increase either upstream or downstream. Most often, the mouth of a river is considered the starting point

(Station 0 + 00), or the river reaches are summed to assign a station number at the channel confluence. Stationing may increase consecutively through PIs or reinitialize at channel turns. In addition, supplemental horizontal reference may also be made to a river mile designation system. River mile systems established years ago may no longer be exact if the river course has subsequently realigned itself. River mile designations can be used to specify geographical features and provide navigation reference for users.

B-15. Datum Conversions and Transformation Methods.

a. Topographic site plan surveys of a project can be performed on any coordinate system. Many localized total station topographic surveys are initiated on (or referenced to) an arbitrary coordinate grid system, e.g., X=5,000 ft, Y=5,000 ft, Z=100 ft, and often elevation or scale reductions are ignored. Planimetric and topographic data points collected on this arbitrary grid in a data collector are then later translated, rotated, scaled, and/or "best fit" to some established geographical reference system--e.g., the local SPCS.

b. The process of converting the observed topographic points on the arbitrary grid system to an established geographical reference system (e.g., NSRS/SPCS) is termed a “datum transformation.” In order to perform this transformation, a few points (preferably three or more) in the topographic database must be referenced to the external reference system. These “control” points on a topographic survey have been previously established relative to an installation or project’s primary control network. They normally were established using more accurate “geodetic control” survey procedures, such as differential leveling, static or kinematic DGPS observations, or total station traverse.

c. CORSPCON. Federal Geodetic Control Subcommittee (FGCS) members, which includes USACE, have adopted NAD83 as the standard horizontal datum for surveying and mapping activities performed or funded by the Federal government. To the extent practicable, legally allowable, and feasible, USACE should use NAD83 in its surveying and mapping activities. Transformations between NAD27 coordinates and NAD83 coordinates are generally obtained using the "CORPS Convert" (CORPSCON) software package or other North American Datum Conversion (e.g., NADCON) based programs.

d. Conversion techniques. USACE survey control published in the NGS control point database has been already converted to NAD83 values. However, most USACE survey control was not originally in the NGS database and was not included in the NGS readjustment and redefinition of the national geodetic network. Therefore, USACE will have to convert this control to NAD83. Coordinate conversion methods considered applicable to USACE projects are discussed below.

(1) Resurvey from NAD83 Control. A new survey using NGS published NAD83 control could be performed over the entire project. This could be either a newly authorized project or one undergoing major renovation or maintenance. Resurvey of an existing project must tie into all monumented points. Although this is not a datum transformation technique, and would not normally be economically justified unless major renovation work is being performed, it can be used if existing NAD27 control is of low density or accuracy.

(2) Readjustment of Survey. If the original project control survey was connected to NGS control stations, the survey may be readjusted using the NAD83 coordinates instead of the NAD27 coordinates originally used. This method involves locating the original field notes and observations, and completely readjusting the survey and fixing the published NAD83 control coordinates.

(3) Mathematical Transformations. Since neither of the above methods can be economically justified on most USACE projects, mathematical approximation techniques for transforming project control data to NAD83 have been developed. These methods yield results which are normally within ± 1 ft of the actual values and the distribution of errors are usually consistent within a local project area. Since these coordinate transformation techniques involve approximations, they should be used with caution when real property demarcation points and precise surveying projects are involved. When mathematical transformations are employed they should be adequately noted so that users will be aware of the conversion method.

e. Horizontal datum transformation methods. Coordinate transformations from one geodetic reference system to another can be most practically made either by using a local seven-parameter transformation or by interpolation of datum shift values across a given region.

(1) Seven parameter transformations. For worldwide (OCONUS) and local datum transformations, many surveying textbooks contain additional information, procedures, and examples and may be consulted.

(2) Grid-shift transformations. Current methods for interpolation of datum shift values use the difference between known coordinates of common points from both the NAD27 and NAD83 adjustments to model a best-fit shift in the regions surrounding common points. A grid of approximate datum shift values is established based on the computed shift values at common points in the geodetic network. The datum shift values of an unknown point within a given grid square are interpolated along each axis to compute an approximate shift value between NAD27 and NAD83. Any point that has been converted by such a transformation method should be considered as having only approximate NAD83 coordinates.

(3) NADCON/CORPSCON. NGS developed the transformation program NADCON, which yields consistent NAD27 to NAD83 coordinate transformation results over a regional area. This technique is based on the above grid-shift interpolation approximation. NADCON was reconfigured into a more comprehensive program called CORPSCON. Technical documentation and operating instructions for CORPSCON can be obtained at the AGC web site listed in Chapter 1. This software converts between the following coordinate systems:

NAD27	NAD83	SPCS 27	SPCS 83
UTM 27	UTM 83	NGVD29	NAVD88
GEOID03/09	HARN		

Since the overall CORPSCON datum shift (from point to point) varies throughout North America, the amount of datum shift across a local project is also not constant. The variation can be as much as 0.1 ft per mile. Examples of some 27 to NAD83 based coordinate shift variations that can be expected over a 10,000-ft section of a project are shown below:

Project Area	*SPCS Reference*	*Per 10,000 feet*
Baltimore, MD	*1900*	*0.16 ft*
Los Angeles, CA	*0405*	*0.15 ft*
Mississippi Gulf Coast	*2301*	*0.08 ft*
Mississippi River (IL)	*1202*	*0.12 ft*
New Orleans, LA	*1702*	*0.22 ft*
Norfolk, VA	*4502*	*0.08 ft*
San Francisco, CA	*0402*	*0.12 ft*
Savannah, GA	*1001*	*0.12 ft*
Seattle, WA	*4601*	*0.10 ft*

(4) Scale changes. The above scale changes will cause project alignment data to distort by unequal amounts. Thus, a 10,000-ft tangent on NAD27 project coordinates could end up as 9,999.91 feet after mathematical transformation to NAD83 coordinates. Although such differences may not be appear significant from a lower-order construction survey standpoint, the potential for such errors must be recognized. Therefore, the transformations will not only significantly change absolute coordinates on a project and the datum transformation process will slightly modify the project's design dimensions and/or construction orientation and scale. For example, on a navigation project, an 800.00-ft wide channel could vary from 799.98 to 800.04 feet along its reach. This variation could also affect grid alignment azimuths. Moreover, if the local SPCS 83 grid was further modified, then even larger dimension changes can result. Correcting for distortions may require recomputation of coordinates after conversion to ensure original project dimensions and alignment data remain intact. This is particularly important for property and boundary surveys. A less accurate alternative is to compute a fixed shift to be applied to all data points over a limited area.

(5) Maximum shift limits. Determining the maximum area over which such a fixed shift can be applied is important. Computing a fixed conversion factor with CORPSCON can be made to within ±1 foot. Typically, this fixed conversion would be computed at the center of a sheet or at the center of a project and the conversions in X and Y from NAD27 to NAD83 and from SPCS 27 to SPCS 83 indicated by notes on the sheets or data sets. Since the conversion is not constant over a given area, the fixed conversion amounts must be explained in the note. The magnitude of the conversion factor change across a sheet is a function of location and the drawing scale. Whether the magnitude of the distortion is significant depends on the nature of the project. For example, a 0.5-ft variation on an offshore navigation project may be acceptable for converting depth sounding locations, whereas a 0.1-ft change may be intolerable for construction layout on an installation. In any event, the magnitude of this gradient should be computed by CORPSCON at each end (or corners) of a sheet or project. If the conversion factor variation exceeds the allowable tolerances, then a fixed conversion factor should not be used. Two examples of determining a fixed conversion factor are illustrated below.

Example 1. Assume a 1 inch = 40 ft scale site plan map on existing SPCS 27 (VA South Zone 4502). Using CORPSCON, convert existing SPCS 27 coordinates at the sheet center and corners to SPCS 83 (US Survey Foot), and compare SPCS 83-27 differences.

	SPCS 83	SPCS 27	SPCS 83 - SPCS 27
Center	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
of Sheet	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
NW	N 3,527,595.553	Y 246,700.000	dY = 3,280,895.553
Corner	E 11,920,522.693	X 2,437,525.000	dX = 9,482,997.693
NE	N 3,527,595.556	Y 246,700.000	dY = 3,280,895.556
Corner	E 11,921,522.691	X 2,438,525.000	dX = 9,482,997.691
SE	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
Corner	E 11,921,522.702	X 2,438,525.000	dX = 9,482,997.702
SW	N 3,526,595.535	Y 245,700.000	dY = 3,280,895.535
Corner	E 11,920,522.704	X 2,437,525.000	dX = 9,482,997.704

(6) Since coordinate differences do not exceed 0.03 feet in either the X or Y direction, the computed SPCS 83-27 coordinate differences at the center of the sheet may be used as a fixed conversion factor to be applied to all existing SPCS 27 coordinates on this drawing.

Example 2. Assuming a 1 inch = 1,000 ft base map is prepared of the same general area, a standard drawing will cover some 30,000 feet in an east-west direction. Computing SPCS 83-27 differences along this alignment yields the following:

	SPCS 83	SPCS 27	SPCS 83 - SPCS 27
West	N 3,527,095.554	Y 246,200.000	dY = 3,280,895.554
End	E 11,921,022.711	X 2,438,025.000	dX = 9,482,997.711
East	N 3,527,095.364	Y 246,200.000	dY = 3,280,895.364
End	E 11,951,022.104	X 2,468,025.000	dX = 9,482,997.104

(7) The conversion factor gradient across this sheet is about 0.2 ft in Y and 0.6 ft in X. Such small changes are not significant at the plot scale of 1 inch = 1,000 ft; however, for referencing basic design or construction control, applying a fixed shift across an area of this size is not recommended -- individual points should be transformed separately. If this 30,000-ft distance were a navigation project, then a fixed conversion factor computed at the center of the sheet would suffice for all bathymetric features. Caution should be exercised when converting portions of projects or military installations or projects that are adjacent to other projects that may not be converted. If the same monumented control points are used for several projects or parts of the same project, different datums for the two projects or parts thereof could lead to surveying and mapping errors, misalignment at the junctions and layout problems during construction.

f. Dual grids ticks. Depicting both NAD27 and NAD83 grid ticks and coordinate systems on maps and drawings should be avoided where possible. This is often confusing and can increase the chance for errors during design and construction. However, where use of dual grid ticks and coordinate systems is unavoidable, only secondary grid ticks in the margins leads to less confusion.

g. Field survey methods. If GPS is used to set new control points referenced to higher order control many miles from the project (e.g., CORS networks), inconsistent data may result at the project site. If the new control is near older control points that have been converted to NAD83 using CORPSCON, two slightly different network solutions can result, even though both have NAD83 coordinates. In order to avoid these situations, it is recommended that all project control (old and new) be tied into the same reference system--preferably the NSRS.

h. Local project datums. Local project datums that are not referenced to NAD27 cannot be mathematically converted to NAD83 with CORPSCON. Field surveys connecting them to other stations that are referenced to NAD83 are required.

B-16. Horizontal Transition Plan from NAD27 to NAD83.

a. General. Not all USACE maps, engineering site drawings, documents, and associated products containing coordinate information will require conversion NAD27 to NAD83. To insure an orderly and timely transition to NAD83 is achieved for the appropriate products, the following general guidelines should be followed:

(1) Initial surveys. All initial surveys should be referenced to NAD83.

(2) Active projects. Active projects where maps, site drawings or coordinate information are provided to non-USACE users (e.g., NOAA, USCG, FEMA, and others in the public and private sector) coordinates should be converted to NAD83 the next time the project is surveyed or maps or site drawings are updated for other reasons.

(3) Inactive projects. For inactive projects or active projects where maps, site drawings or coordinate information are not normally provided to non-USACE users, conversion to NAD83 is optional.

b. Levels of effort. For maps and site drawings the conversion process entails one of three levels of effort:

(1) Conversion of coordinates of all mapped details to NAD83, and redrawing the map,

(2) Replace the existing map grid with a NAD83 grid,

(3) Simply adding a datum note.

For surveyed points, control stations, alignment, and other coordinated information, conversion must be made either through a mathematical transformation or through readjustment of survey observations.

APPENDIX C

Requirements and Procedures for Referencing Coastal Navigation Projects to Mean Lower Low Water (MLLW) Datum

C-1. Purpose. This appendix is an edited reprint of USACE technical guidance that was issued in 1993 to implement applicable portions of Section 224 of the Water Resources Development Act of 1992 (WRDA 1992). This guidance was originally issued as an Engineer Technical Letter—i.e., ETL 1110-2-349, which was subsequently rescinded. Much of the guidance in this ETL is still applicable to those Corps projects that have not been fully converted to the latest federal reference datum or tidal epoch. This includes technical considerations and general implementation procedures for referencing coastal navigation projects to a consistent Mean Lower Low Water (MLLW) datum based on tidal characteristics defined and published by the US Department of Commerce. References herein to the "NOS" (the National Ocean Service) now apply to the current NOAA organization responsible for tides and water levels—the "Center for Operational Oceanographic Products and Services" (CO-OPS).

C-2. Applicability. The technical guidance in this ETL [Appendix] applies to commands having responsibilities for design of river and harbor navigation projects on the Atlantic, Gulf, and Pacific coasts, and where such projects are subject to tidal influence.

C-3. References. [*Outdated references in the original ETL were deleted*]

a. Rivers and Harbors Appropriation Act of 1915 (38 Stat. 1053; 33 U.S.C. 562).

b. Water Resources Development Act of 1992 (WRDA 92), Section 224, Channel Depths and Dimensions.

c. The National Tidal Datum Convention of 1980, US Department of Commerce.

C-4. Background.

a. Depths of USACE navigation projects in coastal areas subject to tidal influences are currently referred to a variety of vertical reference planes, or datums. Most project depths are referenced to a local or regional datum based on tidal phase criteria, such as Mean Low Water, Mean Lower Low Water, Mean Low Gulf, Gulf Coast Low Water Datum, etc. Some of these tidal reference planes were originally derived from US Department of Commerce, National Ocean Service (NOS) observations and definitions used for the various coasts. Others were specifically developed for a local project and may be without reference to an established vertical network (e.g., National Geodetic Vertical Datum of 1929) or a tidal reference. Depending on the year of project authorization, tidal epoch, procedures, and the agency that established or connected to the reference datum, the current adequacy of the vertical reference may be uncertain, or in some cases, unknown. In some instances, project tidal reference grades may not have been updated since original construction. In addition, long-term physical effects may have significantly impacted presumed relationships to the NOS MLLW datum.

b. The National Tidal Datum Convention of 1980 established one uniform, continuous tidal datum for all marine waters of the United States, its territories, and Puerto Rico. This convention thereby lowered the reference plane (and tidal definition) of both the Atlantic and Gulf coasts from a mean low water datum to a MLLW datum. In addition, the National Tidal Datum Epoch (NTDE) was updated to the 1960-1978 period and mean higher/high water datums used for legal shoreline delineation were redefined. The latest tidal epoch update is the 1983-2001 period.

c. Since 1989, nautical charts published by NOAA reference depths (or soundings) to the local MLLW reference datum, also termed a "chart datum." US Coast Guard (USCG) Notices to Mariners also refer depths or clearances over obstructions to MLLW. Depths and clearances reported on USACE project/channel condition surveys provided to NOAA, for incorporation into their published charts in plan or tabular format, must be on the same NOS MLLW reference as the local chart of the project site.

d. WRDA 92, Section 224, requires consistency between USACE project datums and NOAA marine charting datums. This act amended Section 5 of the Rivers and Harbors Appropriation Act of 1915 to define project depths of operational projects as being measured relative to a MLLW reference datum for all coastal regions. Only the Pacific coast was previously referenced to MLLW. The amendment states that this reference datum shall be as defined by the Department of Commerce for nautical charts and tidal prediction tables for a given area. This provision requires USACE project reference grades be consistent with NOS MLLW (latest epoch).

C-5. <u>Impact of MLLW Definition on USACE Projects</u>.

a. Corps navigation projects that are referenced to older datums (e.g., Mean Low Water along the Atlantic coast or various Gulf coast low water reference planes) must be converted to and correlated with the local MLLW tidal reference established by the NOS. Changes in project grades due to redefining the datum from mean low water to NOS MLLW will normally be small, and in many cases will be compensated for by offsetting secular sea level or epochal increases occurring over the years. Thus, impacts on dredging due to the redefinition of the datum reference are expected to be small and offsetting in most cases.

b. All Corps project reference datums, including those currently believed to be on MLLW, must be checked to insure that they are properly referred to the latest tidal epoch, and that variations in secular sea level, local reference gage or bench mark subsidence/uplift, and other long-term physical phenomena are properly accounted for. In addition, projects should be reviewed to insure that tidal phase and range characteristics are properly modeled and corrected during dredging, surveying, and other marine construction activity, and that specified project clearances above grade properly compensate for any tidal range variances. Depending on the age and technical adequacy of the existing MLLW reference (relative to NOS MLLW), significant differences could be encountered. Such differences may dictate changes in channels currently maintained. Future NOS tidal epoch revisions after the current 1983-2001 period will also change the project reference planes.

c. Conversion of project datum reference to NOS MLLW may or may not involve field tidal observations. In many projects, existing NOS tidal records can be used to perform the conversion, and short-term simultaneous tidal comparisons will not be required. Tidal observations and/or comparisons will be necessary for projects in areas not monitored by NOS or in cases where no recent or reliable observations are available.

C-6. Implementation Actions. A number of options are available to USACE commands in assessing individual projects for consistency and accuracy of reference datums, and performing the necessary tidal observations and/or computations required to adequately define NOS MLLW project reference grades. Datum establishment or verification may be done using USACE technical personnel, through an outside Architect-Engineer contract, by another Corps district or laboratory having special expertise in tidal work, or through reimbursable agreement with NOS. Regardless of who performs the tidal study, all work should be closely coordinated with both the USC&GS [now NGS] and NOS [CO-OPS] in the Department of Commerce.

a. Technical specifications. The general techniques for evaluating, establishing, and/or transferring a tidal reference plane are fully described in the USACE and Department of Commerce publications referenced in paragraph C-3. These references should be cited in technical specifications used for a tidal study contract or reimbursable agreement with another agency/command.

b. Department of Commerce contacts. Before and during the course of any tidal study, close coordination is required with the NOS.

c. Sources. If in-house forces are not used, the following outside sources may be utilized to perform a tidal study of a project, including any field tidal observations.

(1) Architect-Engineer (A-E) Contract. A number of private firms possess capabilities to perform this work. Either a fixed-scope contract or indefinite delivery contract form may be utilized. In some instances, this type of work may be within the scope of existing contracts. Contact NOS to obtain a typical technical specification which may be used in developing a scope of work. The references in paragraph C-3 of this appendix must be cited in the technical scope of work for the A-E contract.

(2) Reimbursable Support Agreement. Tidal studies and datum determinations may be obtained directly from the NOS, Department of Commerce, via a reimbursable support agreement. A cooperative agreement can be configured to include any number of projects within a district. Funds are provided to NOS by standard inter-agency transfer methods and may be broken down to individual projects. Contact the NOS to coordinate and schedule a study agreement.

d. Scheduling of conversions. Section 224 of WRDA 92 did not specify an implementation schedule for converting existing projects to NOS MLLW (or verifying the adequacy of an existing MLLW datum). It is recommended that a tidal datum study be initiated during a project's next major maintenance cycle.

e. Funding. No centralized account has been established to cover the cost of converting projects to NOS MLLW datum. Project Operations and Maintenance funds will be used to cover the cost of tidal studies and/or conversions on existing projects. For new construction, adequate funding should be programmed during the initial planning and study phases. Budget estimates for performing the work can be obtained from NOS.

f. MLLW relationship to national vertical network. USACE tidal bench marks should be connected to the NSRS--currently NAVD88. Project condition surveys, maps, reports, studies, etc. shall clearly depict the local relationship between MLLW datum and the NSRS vertical network.

g. Changes in dredging. It is not expected that the datum conversion will significantly impact dredging requirements. USACE commands should request HQUSACE guidance should a datum conversion cause a significant change in a channel's maintained depth.

APPENDIX D

Tampa Harbor Navigation Project: Evaluation of the Project Datum and Implementation of a VDatum Model (Jacksonville District)

D-1. Purpose. This appendix contains excerpts from Jacksonville District reports that illustrate the evaluation of the adequacy of a project datum for a typical deep-draft navigation project. It outlines the procedures for updating the reference tidal datum along with procedures for implementing use of VDatum for dredging and construction surveys.

a. Section 1. Section 1 in this appendix contains excerpts from a 2007 Comprehensive Evaluation of Project Datums (CEPD) report on Tampa Harbor. This report was prepared by HQUSACE directive. This CEPD report evaluated the current condition of the project's datums and recommended corrective actions to bring the project into compliance with Corps policy.

b. Section 2. Section 2 outlines excerpts from a 2009 internal Jacksonville District channel framework report on subsequent actions proposed to correct the deficiencies identified in the Section 1 CEPD report. It also illustrates recommended VDatum site calibration requirements for a project with full VDatum and partial RTN coverage. (Portions of this report were revised and edited since it was based on a superseded version of VDatum and the latest [2010] release of the VDatum model for Tampa Bay has not yet been field calibrated).

D-2. Project Description. The total project consists of a channel from the Gulf of Mexico to ports in Tampa Bay—see Figure D-1. Project features include the entrance channel from the Gulf of Mexico to Hillsborough Bay. At Hillsborough Bay, the channel splits into two legs, with one continuing west to Port Tampa and the other east to Gadsden Point. The west channel continues to Port Tampa and ends in a turning basin. The west channel to Gadsden Point includes the Alafia River, Port Sutton, East Bay, and Seddon Channels. The project depth varies from 45 feet in the entrance channel at the Egmont Bar Channel to 30 feet in the Alafia River. Length of the project is about 67 miles including 3.6 miles in the Alafia River. The Port of Tampa has more cargo tonnage than all other Florida ports combined.

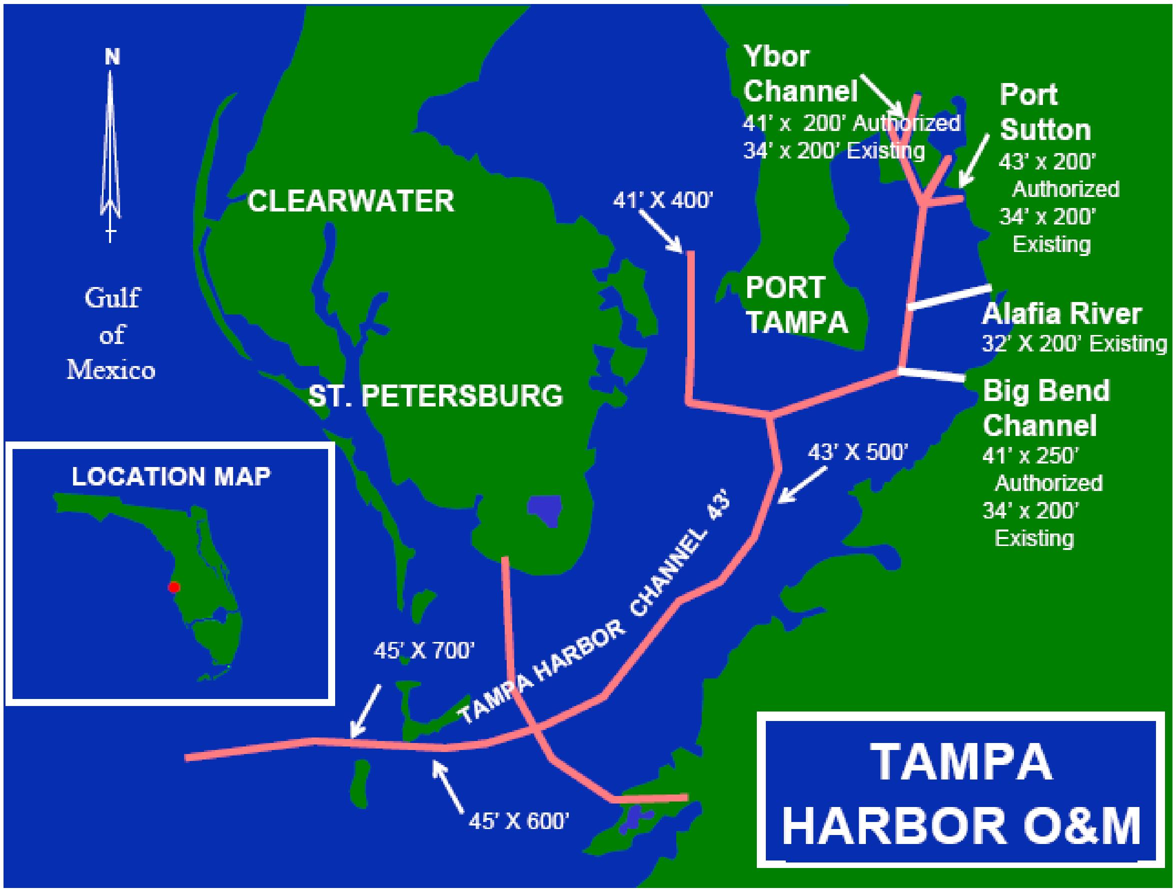

Figure D-1. Tampa Harbor Deep-Draft navigation project.

D-3. Section 1—Tampa Harbor CEPD Project Datum Evaluation Report (Jacksonville District).

US ARMY ENGINEER DISTRICT, JACKSONVILLE
Comprehensive Evaluation of Project Datums

PROJECT DATUM EVALUATION REPORT

Tampa Harbor, Florida (30 to 45-Foot Projects)
Hillsborough River (9 & 12-Foot Project)
Alafia River (30-Foot Project)

9 September 2007 (Revised 15 Oct 07)

Synopsis of Overall Project Assessment

This report assesses the adequacy and accuracy of reference datums for the Tampa Harbor Project, including all related shore protection control structures, and/or upland/offshore disposal sites associated with this project, as described in the project authorization documents. This evaluation is performed in compliance with the Commanding General's 4 December 2006 directive memorandum, subject, "Implementation of Findings from the Interagency Performance Evaluation Task Force for Evaluating Vertical Datums and Subsidence/Sea Level Rise Impacts on Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects." The findings in this report are summarized below.

1. The project is NOT compliant with the standards and guidance in EC 1110-2-6065 [1]*.*

2. The current tidal MLLW reference datum model for this project is of uncertain origin, not fully documented, and appears not to have been updated to the latest 1981-2001 sea level epoch in accordance with WRDA 92. NOAA CSDL has developed a VDatum hydrodynamic model of the MLLW gradient throughout the area. This model is not being used in USACE surveys.

3. Currently, water surface elevation corrections for dredging measurement & payment are based on extrapolated staff gage readings set from benchmarks of uncertain origin, that are not referenced to the NSRS, and/or are referenced to the superseded NGVD29 datum. Use of NOAA PORTS gage readings may be resulting in mixed tidal epochs. Recent RTK surveys have originated at NOAA tidal benchmark sites; however, survey and dredging reference are still on the superseded 1960-1978 tidal epoch. Project framework and control documents do not clearly define references or relationships between these benchmarks and NOAA tidal gages or tidal benchmarks.

4. Given VDatum coverage, no significant corrective actions will be required to hydrodynamically model the tidal regime, or model the geoid. Corrective actions will be *required to establish a fully calibrated RTK horizontal and vertical positioning network throughout the project, and update project framework documents. Recommended actions are outlined in this report.*

5. The estimated cost to effect corrective actions is $76,000.

6. Corrective actions should be budgeted and programmed for completion in FY 2008.

7. Estimated cost avoidance savings potentially realized in effecting corrective actions is $7,500,000.

Hydrological and Hydraulic Modeling Requirements

1. Figure CEPD-1 shows the typical tidal correction locations currently used on a portion of this project. The reference datum is referred to MLLW as required under WRDA92. However, the tidal epoch has not been updated from 1960-1978 to 1983-2001. Water surface elevations are extrapolated from shore-based gages.

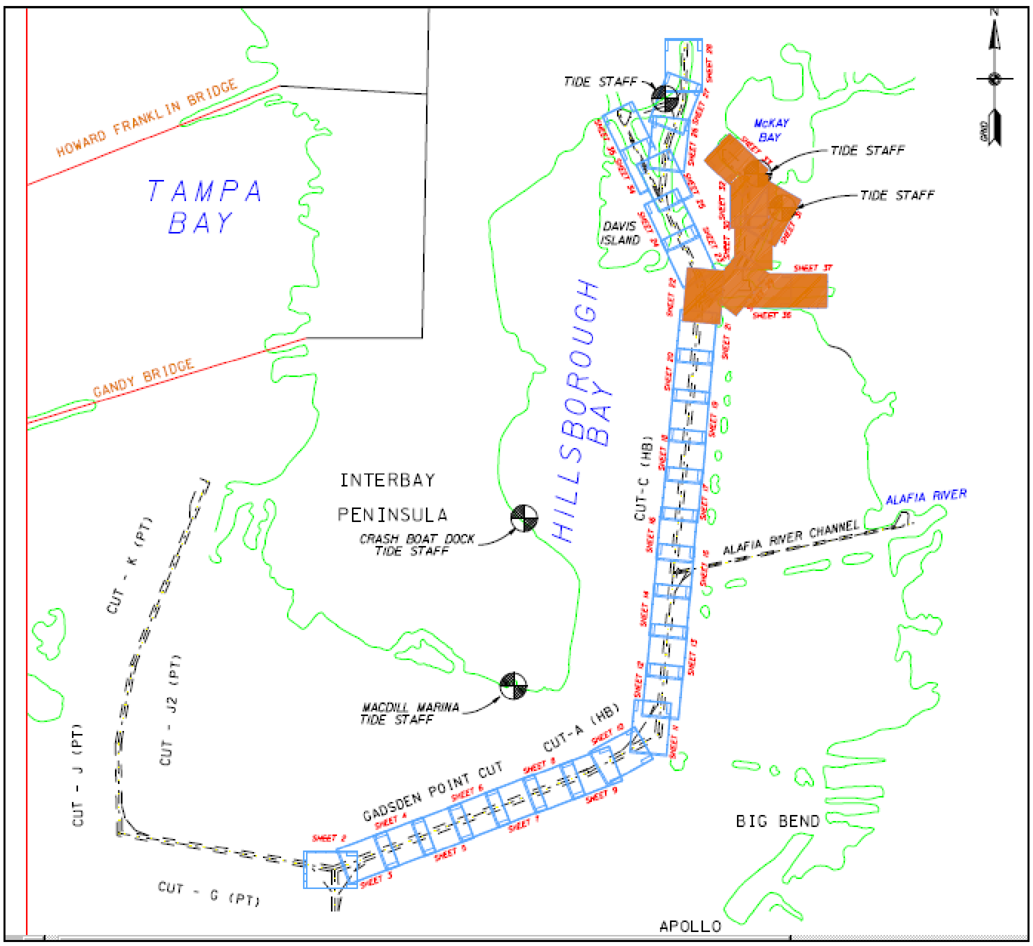

Figure CEPD-1. Typical tidal gage locations currently used in northern portion of project (07-076)—reference NGVD29 & 1960-78 epoch.

2. Figure CEPD-2 depicts the existing NOAA CSDL VDatum model that can be used to develop MLLW-NAVD88-geoid relationships throughout this project. Note that the VDatum model is currently on the 1960-1978 tidal epoch.

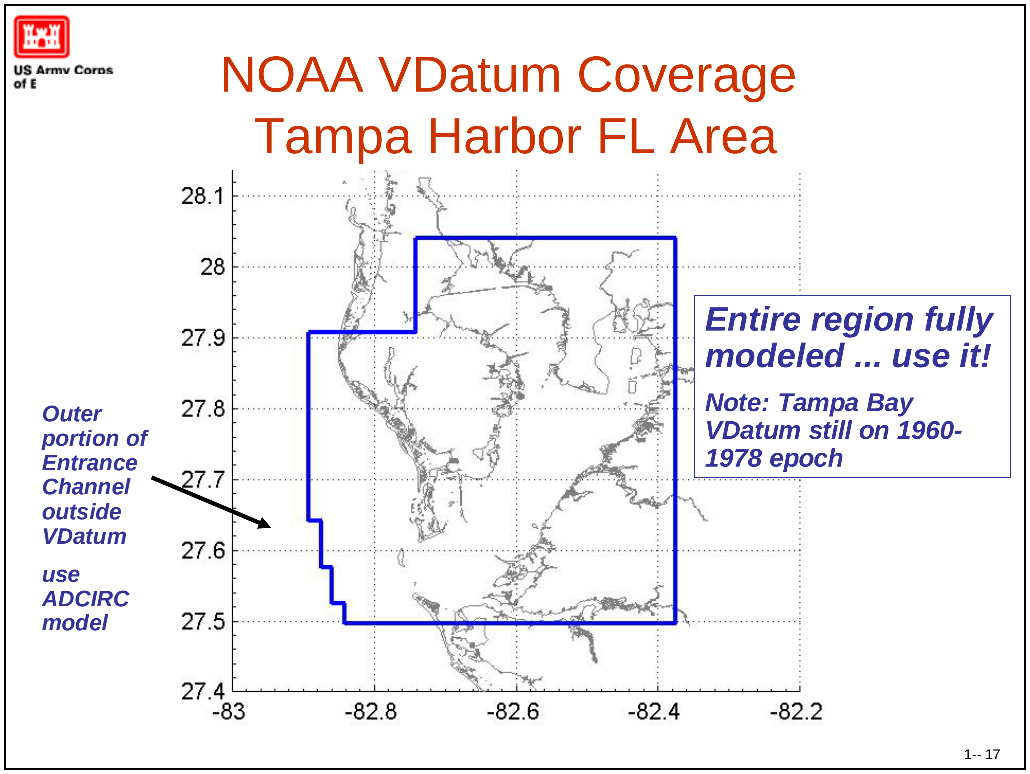

Figure CEPD-2. NOAA CSDL VDatum model coverage.

3. Figure CEPD-3 depicts tidal bench mark gage sites maintained in the NOAA CO-OPS database. It is presumed that these gages were used to develop the VDatum model—this should be verified with NOAA CSDL. Not all these gages have been updated to the 1983-2001 epoch.

Figure CEPD-3. NOAA CO-OPS tidal bench mark sites in Tampa Harbor area.

4. The outermost portion of Egmont Cut 1 does not have VDatum coverage. ADCIRC data may be used to estimate the tidal range gradient in this area if it is significant (i.e., gradient and/or maintenance).

5. No requirement for additional gages is anticipated on this project.

6. The 10-mile Hillsborough River shallow draft portion of the project may not warrant detailed modeling or RTK coverage. Ascertain if this area was picked up within the resolution/coverage of the VDatum model. Determine effort, if any, based on past survey/maintenance activity. (This portion of project not researched during CEPD assessment).

7. Actions.

(1) Obtain updated tidal epoch data from NOAA CO-OPS on tide stations not yet updated to 1983-2001.

(2) Request CSDL Update VDatum model to 1983-2001 epoch.

(3) From VDatum model, generate a 3D digital gridded (100 ft x 100 ft) tidal model for the entire project area depicting the relationship between MLLW (1983-2001), NAVD88, and LMSL.

(4) RTK network calibration verification. The RTK network corrections derived from the updated model needs to be verified at NOAA tide gages in the area. Any variations need to be resolved and the model corrected accordingly.

8. Geoid Model Update Requirements

(1) Action. None—geoid heights are included in the VDatum model.

9. NOAA Tidal Gage RTK Network Calibration and NSRS Connection Requirement.s

(1) Sufficient NSRS high-order accuracy vertical control exists in this project area to provide NAVD88 reference for navigation measurement & payment surveys, topographic surveys of upland disposal sites, or construction surveys of coastal protection structures—see Figure CEPD-4. These NSRS control points will suffice as "Primary Project Control" PBMs in accordance with the requirements of EC 1110-2-6065 [1]. All supplemental or local project control PBMs, RTK calibration PBMs, and existing USACE PBMs deemed suitable for future use, must be directly connected to these NSRS "Primary Control" PBMs using either GPS or differential leveling methods. Reference EC 1110-2-6065 [1] for detailed survey specifications, metadata reporting, documentation requirements, and requirements for NSRS input. (Note that supplemental or local PBMs, although tied to the NSRS, do not need to be input into the NSRS; however, there may be exceptions at NOAA or Corps gage sites).

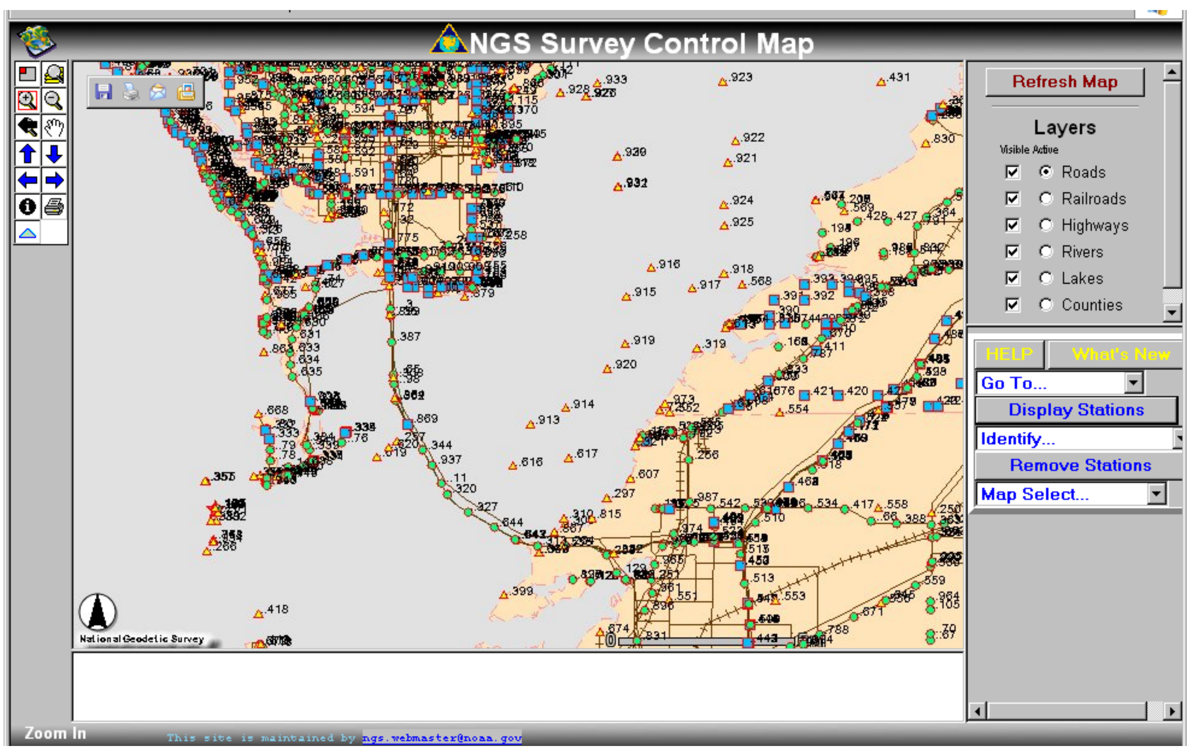

Figure CEPD-4. NGS/NSRS control data in southern region of Tampa Bay.
(Squares-H+V, Circle-V)

(2) Sufficient NOAA tide gages and benchmarks (see Figures CEPD-3 and CEPD-4) exist throughout the project area to facilitate calibration of the combined tidal-geoid model used with a RTK elevation measurement system. Tide staffs should be set at NOAA gage sites on MLLW (1983-2001) to calibrate the model and for use as a QC on periodic RTK measurement & payment surveys.

10. Actions.

(1) Recover tidal benchmarks and set RTK calibration staffs at approximately eight to ten NOAA tide gage sites along the project reach; as needed to afford RTK coverage to the work sites. Follow EC 1110-2-6065 [1] *bench mark recovery and documentation requirements.*

(2) As required, perform full OPUS DB/PROJECT observations at tidal benchmarks at each of the above tidal gage sites, per EC 1110-2-6065 [1] *specifications. No GPS observations are required if the site has NSRS NAVD88 control, or can be leveled to.*

(3) Tidal benchmarks at existing NOAA CO-OPS sites are recommended as temporary RTK base stations for local dredge operations and measurement & payment surveys.

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[1] Above references to "EC 1110-2-6065" in this 2007 CEPD report refer to an interim guidance document in effect at that time. This manual replaces the interim circular]

D-4. Section 2—Tampa Harbor Channel Framework Report (Jacksonville District).

Tampa Harbor Navigation Project

Master Channel Control Framework Report

21 August 2009

REFERENCES:

CESAJ-EN-DW "Procedures & Standards for Developing & Maintaining Master Channel Control Framework Documents" (7 August 2009)

CEPD PROJECT DATUM EVALUATION REPORT—Tampa Harbor, dated 9 Sep (15 Oct) 2007

2009 Project Condition Survey: (Reference File 09-083)

1. Overview

a. This report summarizes actions in developing a Master Channel Control Framework for the subject project. The scope of this report includes all channels from Egmont Cut in the Gulf into Tampa Bay and Hillsborough Bay; including the Alafia River and port areas. It does not include the 9- &12-ft Hillsborough River project.

b. The master channel control framework version was developed using a composite of the most recent PCS Survey (2009)--referenced above—and updated channel framework dgn files. The composite Master Channel Control Framework dgn file was developed using these various sources. The project has been transformed to NAD83 and the MLLW vertical reference grade updated to the current tidal epoch (1983-2001).

2. Horizontal Datum Transform

Existing project framework drawings were converted to NAD83 at some point prior to the 2007 CEPD Report—no documentation exists on this effort. It is assumed that the process outlined in EM 1110-2-1005 (and prior guidance documents) was followed and the current channel PI framework coordinates adequately define the geospatial channel alignment.

3. Vertical Datum Modeling (VDatum)

This project area is covered by a VDatum model referenced to the 1960-1978 epoch—refer to the 2007 CEPD Report. In 2009, CESAJ-EN-DG updated the current VDatum model for Tampa Bay to the latest tidal epoch (1983-2001)—a 0.20 ft average change in MLLW was assumed constant throughout the project. A Kinematic Tidal Datum (KTD) file was then created for Tampa Bay. This updated model (and KTD file) should be used until NOAA releases a new version of VDatum in FY10 which will be on the 1983-2001 epoch. A "preliminary" site calibration of the epoch-corrected VDatum model was performed at various NOAA gage sites around the perimeter of Tampa Bay. NOAA gage MLLW elevations were compared with the VDatum MLLW from RTK/RTN observations. Maximum deviations between RTK tide and gage tide observations were not more than 0.2 ft. Once the new FY10 release of VDatum is available, a final site calibration should be performed for the channel reaches, as outlined below.

4. Geoid Model

The current NGS geoid model shall be used to correct for undulations over the project. The extrapolated geoid heights shall be considered as absolute for correcting observed ellipsoid heights.

5. Construction Survey Positioning Criteria

a. Horizontal Vessel Positioning. A regional RTN network based on NSRS CORS covers most of the Tampa Bay project. Therefore, the RTN indirectly represents the PPCP(s) for this project, subject to local site calibration at a NSRS or NWLON point.

b. The following reaches are outside the RTN coverage and require an RTK base station.

Channel Reach	*RTK Base*	*PID*
Egmont Cut 1	*Desoto C*	*AG 0489*

c. Vertical Control. Vertical RTK/MLLW calibrations on the most recent surveys have been "checked" by comparisons with real-time PORTS values. When the FY10 update of VDatum is released, a complete RTK/RTN vertical calibration of "RTK Tides" against gage tide readings should be performed relative to a staff gage set to MLLW from tidal bench marks at the following NOAA gage stations.

NOAA Gage	*Station ID*	*"RTK Tide" Calibration Check Results*
Egmont Key	*872 6347 or*	_________
Port Manatee	*872 6384*	_________
St. Petersburg	*872 6520*	_________
Gadsden Point	*872 6573*	_________

Long Shoal-MacDill	*872 6604*	_________
Davis Island	*872 6657*	_________
Port Tampa	*872 6607*	_________

d. RTN/RTK observed tide levels above MLLW should ideally agree with the staff gage observations to around 0.2 ft. If these differences at a gage are consistent, then these gage-channel-zoned RTK/RTN site calibration values should be applied by all users.

6. Master Channel Control Framework Drawing Notes

The following notes shall be placed on the master channel control framework drawing.

HORIZONTAL REFERENCE SYSTEM:
THE HORIZONTAL REFERENCE DATUM FOR THIS PROJECT IS NAD83, BASED ON THE CURRENT VERSION OF THE NOAA NATIONAL SPATIAL REFERENCE SYSTEM (NSRS). GRID COORDINATES ARE SHOWN IN THE FLORIDA STATE PLANE COORDINATE SYSTEM (SPCS)—WEST ZONE (0902). CHANNEL STATIONING AND OFFSET COORDINATES ARE RELATIVE TO THE INDICATED CHANNEL BASELINE FOR EACH CHANNEL REACH. CHANNEL ALIGNMENTS ARE GRID BEARINGS REFERENCED TO THE SPCS GRID. UNLESS OTHERWISE INDICATED, CHANNEL WIDTHS AND LIMITS CONFORM TO THE AUTHORIZED PROJECT DIMENSIONS.

VERTICAL REFERENCE SYSTEM:
THE TIDAL REFERENCE GRADE FOR THIS PROJECT IS MEAN LOWER LOW WATER (MLLW), BASED ON THE NOAA 1983-2001 NATIONAL TIDAL DATUM EPOCH. THE NAVD88-MLLW RELATIONSHIP ON THIS PROJECT HAS BEEN HYDRODYNAMICALLY MODELED USING NOAA VDATUM--REFERENCE “TAMPA HARBOR FRAMEWORK REPORT.” THE ESTIMATED LOCAL (RELATIVE) ACCURACY OF THIS TIDAL MODEL IS ± 0.1 FT.

CONSTRUCTION SURVEY POSITIONING CRITERIA:
HORIZONTAL POSITIONING AND WATER SURFACE ELEVATION MEASUREMENTS (INCLUDING CALIBRATIONS) SHALL BE PERFORMED UTILIZING REAL-TIME KINEMATIC (RTK) OR RTN GPS OBSERVATIONS FROM (OR CALIBRATED TO) THE FOLLOWING PRIMARY REFERENCE PBMS. PBM COORDINATES AND TIDAL PBM MLLW ELEVATION DATA SHALL BE OBTAINED FROM THE CURRENT NOAA NSRS AND NWLON DATABASES.

MLLW CALIBRATION GAGES (NOTE THAT SOME GAGES MAY HAVE SITE CALIBRATION ADJUSTMENTS):

Channel Reach	*NOAA Gage*	*Station ID*
Egmont Cuts	*Egmont Key*	*872 6347 or*
Mullet Key Cut	*Mullet Key*	*872 6364*
Cut A, Cut B, and Cut C	*Port Manatee*	*872 6384*
Cut D, Cut E, and Cut F	*St. Petersburg*	*872 6520*
Gadsden Point Cut to PI Cut A & C (HB) and Cut G (PT)	*Gadsden Point*	*872 6573*
Cut C (HB) and Alafia River Channel	*Long Shoal-MacDill*	*872 6604*
Davis Island, Seddon Island	*Ballast Point*	*872 6639 or*
Port Sutton, & McKay Bay Channels	*Hooker Point*	*872 6668 or*
	Davis Island	*872 6657*
Cut J (PT) & Cut K (PT)	*Port Tampa*	*872 6607*

RTK BASE STATIONS OUTSIDE RTN COVERAGE:

Channel Reach	*RTK Base*	*PID*
Egmont Cut 1	*Desoto C*	*AG 0489*

THE SPATIALLY MODELED NAVD88-MLLW RELATIONSHIPS FOR EACH CHANNEL REACH HAVE BEEN INCORPORATED INTO A KINEMATIC TIDAL DATUM MODEL FOR TAMPA HARBOR. THIS KTD FILE INCORPORATES A 0.20 FT EPOCH CORRECTION INTO THE NOAA PUBLISHED VDATUM MODEL ON THE 1960-1978 EPOCH. THIS CESAJ-OD-H KTD MODEL SHALL BE USED TO CORRECT MEASURED RTK/RTN ELLIPSOID HEIGHTS FOR NAVD88-MLLW DATUM AND GEOID HEIGHT VARIATIONS; FOR SURVEY OPERATIONS PERFORMED ON THIS PROJECT. [AN UPDATED NOAA VDATUM MODEL IS EXPECTED IN 2010].

REFERENCES:

"TAMPA HARBOR FRAMEWORK REPORT," VERSION DATED 21 AUG 09.
MASTER CHANNEL FRAMEWORK FILE [TampaHbrV-SPmccf.dgn] VERSION __ ___ 09 AUTHORIZATION DATA: (REFER TO TAMPA HARBOR FRAMEWORK REPORT)
LOCATION OF REFERENCES AND KTD FILE: PROJECTWISE\CONTROL DATA\NAVIGATION PROJECTS\TAMPA HARBOR

A sample portion of the KTD file generated for Tampa Bay is shown below. The values in the file represent the differences between NAVD88 and MLLW (1983-2001), at the grid points in the file. NAVD88-MLLW differences throughout the entire Tampa Bay region vary some 0.3 ft. This KTD file would be provided to all users, along with any site calibration gage constants.

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0 0 0 0 0 +1.73 +1.74 +1.75 +1.75 +1.75 +1.75 0 0 0 0 0 0 0 0 0 0
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APPENDIX E

Tidal Modeling Procedures for Coastal Navigation Projects

E-1. Purpose. This appendix provides supplemental guidance to Chapter 4. It contains technical guidance and examples of interpolated tidal modeling procedures for USACE coastal navigation projects. It is primarily applicable in those regions where NOAA VDatum model coverage is not complete (as of 2009) or does not exist, such as in intracoastal waterways and sounds.

E-2. Requirements for Accurately Modeled Tidal Reference Datums. Tidal reference datums vary both spatially and temporally. Thus, the water surface elevation at a shore-based gage is adequate only for that specific location and time. The height of the water level will be significantly different between two points around an inlet, due to varying times and weather conditions—see Figure E-1. Likewise the MLLW datum will vary with the tidal range variations, which are modified by the topography of an inlet or coastal region. The MLLW datum elevation at a reference gage should not be extrapolated to another location without some modeled correction. It is also subject to long-term variation due to sea level change, subsidence, or other factors. This requires periodic updating of tidal datums based on NOAA's latest NTDE, which is currently 1983-2001 for most areas.

a. Tidal datum models. Lack of accurately modeled tidal datums can have significant impacts on navigation project construction and maintenance costs. Navigation projects that are not adequately referenced to an established tide gage and modeled MLLW datum plane, or have not been updated for sea level change, can result in overdredging or underdredging, along with increased construction disputes and claims. The primary factors that need to be considered in evaluating the adequacy of depth grade determination on a navigation project include the following:

(1) Tidal range variations over the project reach. Geospatial variation of MLLW dredging datum relative to the orthometric and ellipsoid datums used to reference acoustic depth measurements.

(2) Tidal phase variations over the project reach. Real-time survey techniques used to measure the elevation of the water surface at the construction site that corrects tidal phase variations observed at a reference tide gage.

(3) Tidal epoch adjustments for sea level or land subsidence changes. Involves monitoring NOAA tide gage records for changes to tidal epochs, tidal PBM adjustments, etc.

(4) Quality of reference tidal datum determinations. Awareness of the uncertainties in tidal gage datums and any models derived therefrom—i.e., VDatum.

b. Tide gage extrapolations. The long-established practice for dredging and related payment surveys of navigation projects involves extrapolation of a water (tide) level gage to the offshore construction site—i.e., extrapolating the tide level at the gage to the offshore point. This assumes both the water surface level and reference datum range are constant over the extrapolated distance—i.e., assumes no tidal phase or range variations exist. This distance may range from a few hundred feet to over 10 miles. These assumptions of linearity in water surface levels and datum degrade with distance from the reference gage. At low tidal ranges, longer extrapolations may be possible. At higher ranges (> 2 ft), extrapolations greater than a half-mile to 1 mile may be invalid and inaccurate. In addition, local weather conditions may further degrade the distance that a tide reading can be reliably extrapolated from a gage. Sea surface setup due to strong winds can significantly alter the surface model. Approximate methods for estimating tide phase differences ("tidal zoning") are used in some Districts, with mixed accuracy results as these methods do not account for local weather conditions.

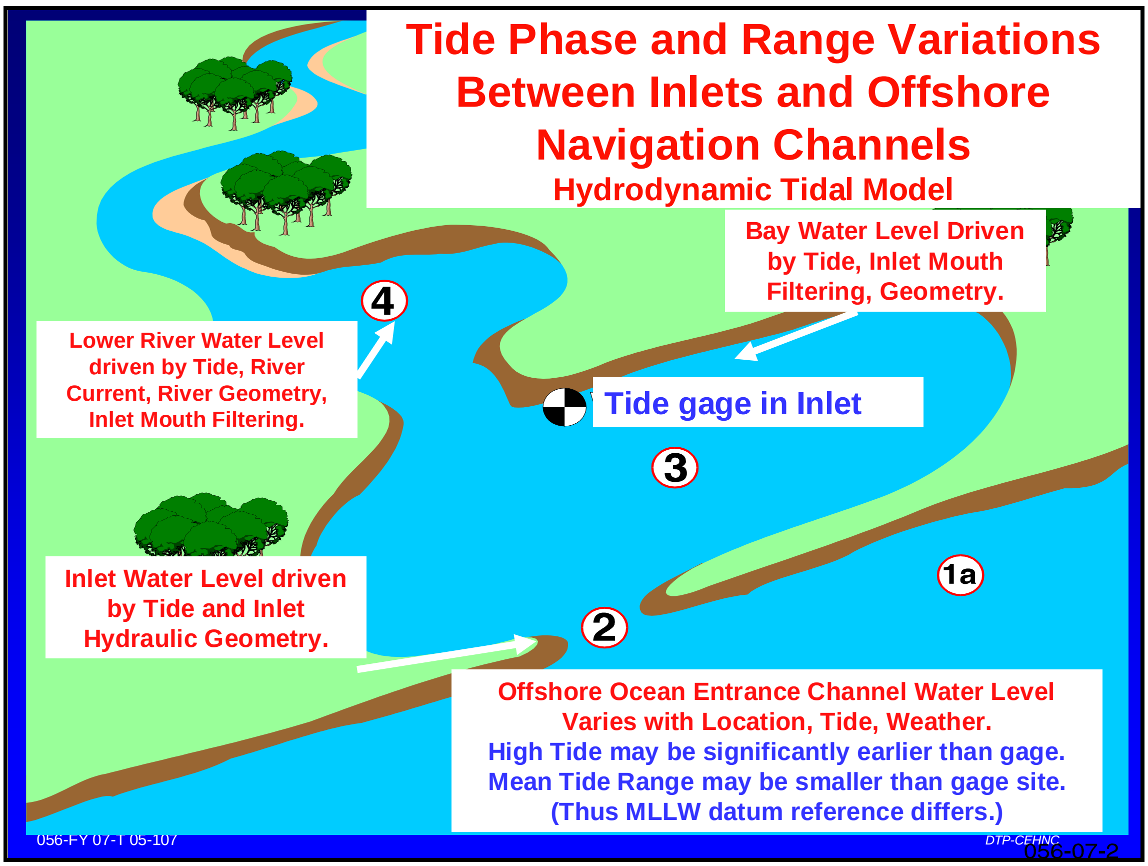

Figure E-1. Tide phase and range variations at an inlet.

c. Hydrodynamic conditions at tidal inlets. The overall effect of adverse conditions encountered at typical USACE coastal inlets is best summarized in the following excerpt from Part II of EM 1110-2-1100 (*Coastal Engineering Manual*).

“Hydrodynamic conditions at tidal inlets can vary from a relatively simple ebb-and-flood tidal system to a very complex one in which tide, wind stress, freshwater influx, and wind

waves (4- to 25-sec periods) have significant forcing effects on the system … Flow enters the bay (or lagoon) through a constricted entrance, which is a relatively deep notch (usually 4 to 20 m at the deepest point). Entrance occurs after flow has traversed over a shallow shoal region where the flow pattern may be very complex due to the combined interaction of the tidal-generated current, currents due to waves breaking on the shallow shoal areas, wind-stress currents, and currents approaching the inlet due to wave breaking on adjacent beaches …. Particularly during stormy conditions with strong winds, flow patterns may be highly complex. Also, the complicated two-dimensional flow pattern is further confounded because currents transverse to the coast tend to influence the propagation of waves, in some cases blocking them and causing them to break … Final complications are structures such as jetties, which cause wave diffraction patterns and reflections. In inlets with large open bays and small tidal amplitudes, flows can be dominated by wind stress. In such cases, ebb conditions can last for days when winds pile up water near the bay side of the inlet, or long floods can occur when winds force bay water away from the inlet. Most inlet bays, however, are small and some are highly vegetated, so wind stress is not a dominant feature, except under storm conditions ... Although many bays do not receive much fresh water relative to the volume of tidal flow, substantial freshwater input due to river flow can sometimes create vertically stratified flows through a tidal inlet. Typically, however, well-mixed conditions exist for most inlets."

E-3. Modeling the MLLW Dredging Datum on USACE Navigation Projects. Most often, linear or surface interpolations between gages will be used. On projects with larger tide ranges where the uncertainty of a linear model between gages increases beyond the allowable tolerance, a more sophisticated hydrodynamic model may be required to best define the MLLW datum. This presumes adequate gage records exist from which to calibrate the tidal model in an area. On some projects, a single gage may be adequate. Others may require additional gages to define and verify the model. If these additional gages do not exist, then a gaging program will have to be programmed. In addition, topographic and bathymetric models of the project may have to be generated if they do not exist. A firm connection to the orthometric datum (NAVD88) may also be required. Thus, a number of project-specific technical factors will govern the overall effort required to model the MLLW datum plane of a project. This will also include the experience of those assessing the tidal model relative to the required relative accuracy of the tidal model.

a. Tidal error budget. One must not lose sight of the overall error budget in evaluating the effort required to model the MLLW datum on a project. Relative to removing large phase and wind setup errors with RTK measurements, these MLLW datum modeling errors are often insignificant. Thus, before embarking on any extensive and costly gaging program, the significance or sensitivity of these added gage observations on the overall tidal model must be substantiated. Likewise, the difference between a simple linear interpolation and a hydrodynamically modeled interpolation must be evaluated for significance relative to the intended tolerance. In addition, there is no point in performing elaborate MLLW datum tidal modeling unless RTK surface elevation measurements are mandated for the completed project. Having a MLLW tidal model accurate to ±0.1 ft with a ±1 ft phase error due to extrapolated gage readings five miles offshore would obviously be an inconsistent use of resources.

b. Tidal model accuracy. An evaluation of the errors in a tidal model is necessary to evaluate uncertainties between tide stations. The primary factors that need to be considered in modeling a reference tidal datum around a coastal protection structure include the following:

(1) Tidal phase variations over the project reach.

(2) Tidal range variations over the project reach.

(3) Tidal epoch adjustments for sea level or land subsidence changes.

(4) Quality of reference tidal gage datum determinations

(5) Seasonal variations in LMSL (i.e., biased sea level rise during hurricane season).

(6) Need for short-term (i.e., 5-year) tidal epochs in high subsidence or uplift areas.

c. Modeling tide range or MLLW variations over a navigation project. Variations in tidal range (i.e., undulations in MLLW datum relative to MSL or to the local geodetic/orthometric datum) within a project must be accounted for. This requires developing some model of the tidal hydrodynamic characteristics throughout the project. Figure E-2 illustrates this MLLW variation over a Jacksonville District deep-draft coastal inlet project (St Johns River—Ocean to Jacksonville, FL). The MLLW datum relative to MSL varies from the ocean through the entrance jetties and up river some 20 miles to the termination of the deep-draft project past the gage at Longbranch, and further upstream in the shallow-draft project to Palatka, FL. The MSL reference plane also varies relative to NAVD88, generally rising upstream. Figure E-2 also depicts that NGVD29 and NAVD88 are not parallel datums. The MSL-MLLW datum variation may also be impacted by fresh water flow into the tidal area.

(1) Modeling the MLLW datum through a navigation project requires an adequate density of tide gages from which the model can be calibrated, and intermediate datum variations between the gages can be modeled. In Figure E-2, the roughly 5.6 ft tide range at the ocean narrows down to 1.6 ft over a 25-mile navigation project. Although the gages shown in Figure E-2 are spaced at about every 5 to 10 miles, they should be of sufficient density to calibrate a hydrodynamic tidal model for this project. The interpolations between the gages shown on Figure E-2 represent only a crude tidal model of the MLLW reference plane—a full hydrodynamic tidal model such as VDatum would be represented by a smooth curve. In many cases with small tidal range variations, or with a dense gage network, a linearly interpolated model may prove adequate. That may be the case for portions of the above project where the variation between gages is not large.

(2) Figure E-3 illustrates the tidal range variation over seven miles of a Norfolk District shallow draft project on the Atlantic east coast in Virginia. There would appear to be a sufficient density of gage data to model the MLLW datum plane for this project—updating the older MLW datum. The NGVD29 orthometric datum reference on this project needs to be updated to NAVD88 along with a NAD83/GRS80 ellipsoid reference. Note that the relationship to the legacy datum (Corps of Engineers Low Water— C.E.L.W.) is shown on the figure.

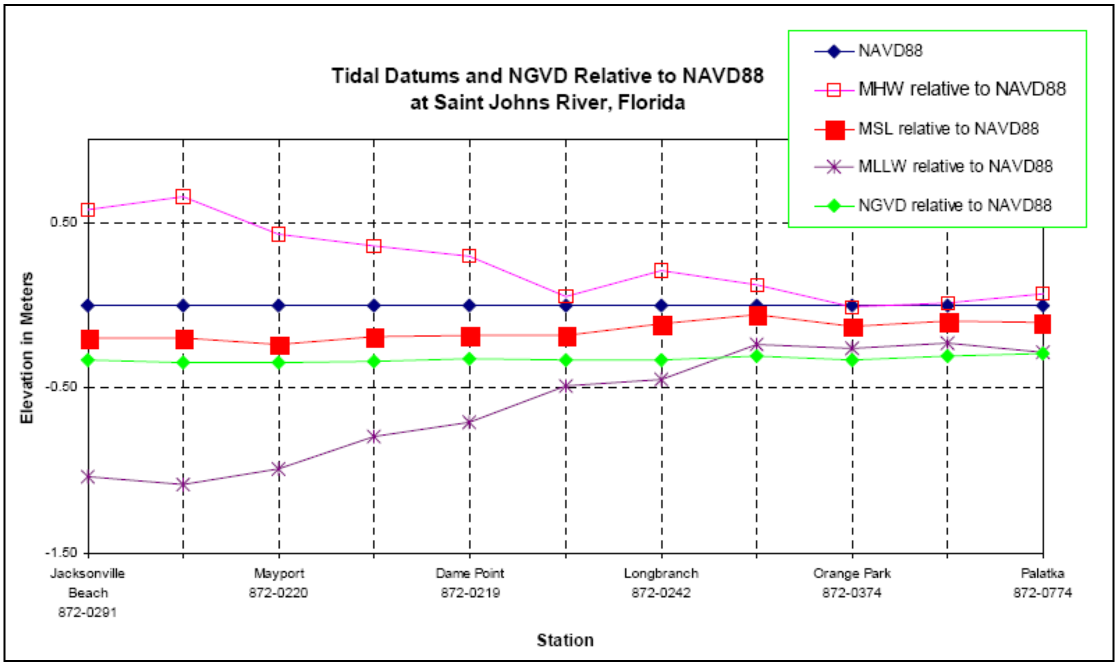

Figure E-2. Tidal range variation at a coastal inlet.

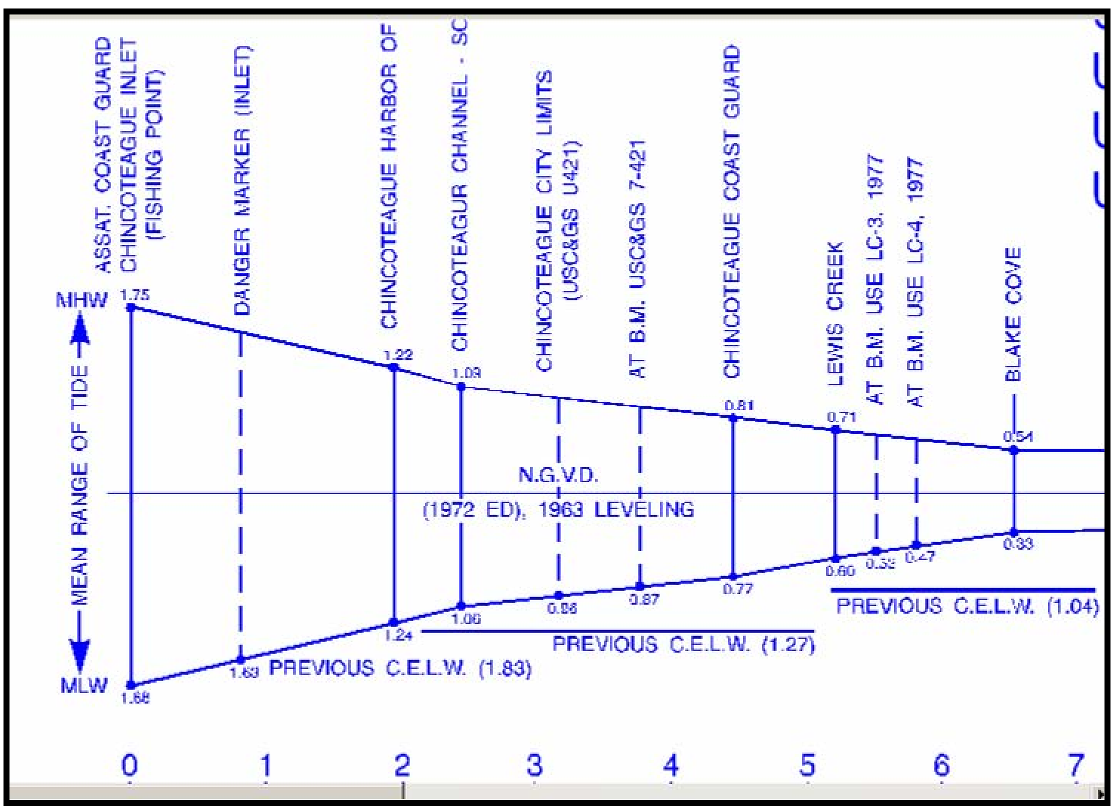

Figure E-3. Tidal range variation at Chincoteague Inlet, VA.

E-4. Procedures for Estimating Navigation Project MLLW Datum Models using Spatial Interpolation Techniques.

a. Tidal model updating actions. A number of options exist to update a tidal model for coastal navigation projects that require upgrading to the current NOAA tidal reference. Updating the tidal model requires the following basic actions: (1) ensure tidal datum reference planes (MLLW) are defined relative to published NOAA gages and tidal bench marks, (2) ensure the latest NTDE adjusted by NOAA is used, (3) model the MLLW reference plane relative to NAVD88 throughout the length of the project, (4) publish and disseminate the NAVD88-MLLW model for users, (5) optionally develop the NAVD88-MLLW datum relationship at tidal bench marks if these marks are used as RTK base stations, and (6) submit any USACE hydrodynamic tidal modeling data to NOAA for their use in expanding the nationwide VDatum.

b. Tidal gaging options. Items (1) and (2) above are easily achieved as long as an existing or historical gage exists at the navigation project. This will likely be the case for the majority of the Corps' deep-draft navigation projects. If not, then a standard gaging program will have to be developed in order to establish a tidal datum at a project—see "*Computational Techniques for Tidal Datums Handbook*" (NOAA 2003). Any such effort must be coordinated with NOAA in order to ensure the project becomes included in NOAA's NWLON inventory. Time and cost estimates for performing the gaging can be obtained from NOAA. Project modeling—Items (3) through (6) above—will require close coordination with District H&H elements, ERDC/CHL, and/or NOAA. In small tide ranges either between gages or in the overall area, linear interpolation of the MLLW model will often be sufficiently accurate and economically developed. These models may already have been developed for some projects, and may currently need only to be adjusted for tidal epoch updates and geoid models.

c. Example of model vs. interpolation decisions--Miami Harbor (Jacksonville District). Figure E-4 depicts a navigation project (Miami Harbor) where a simple straight-line interpolation of the tidal datum might be warranted in lieu of performing a full hydrodynamic model study. Initial estimates of changes in time and range of tide for any survey area can be obtained from a review of the NOAA tide prediction "Table 2" information found on the NOAA CO-OPS web site. (This web site also provides links to NGS bench mark datasheets). The NOAA tide table values should be used with caution as the data summaries are from observations of varying lengths and various time periods and may be out of date and no longer reflective of current conditions. The tide tables list mean ranges of tide (MHW – MLW), Spring Ranges of Tide (Range of tide at New and Full moons), and the elevation of Mean Tide Level (MTL) above Chart Datum (MLLW). NOAA tide prediction data for the Miami Harbor area is shown below (in feet).

	Lat	*Long*	*Mn Rge*	*Spg Rge*	*MTL*
Miami Harbor Entrance	*25° 46.1'*	*80° 07.9'*	*2.46*	*2.93*	*1.39*
Government Cut,					
Miami Harbor					
Entrance	*25° 45.8'*	*80° 07.8'*	*2.32*	*2.83*	*1.32*
Biscayne Bay					
San Marino Island	*25° 47.6'*	*80° 09.8'*	*2.14*	*2.57*	*1.21*
Miami, Marina	*25° 46.7'*	*80° 11.1'*	*2.18*	*2.59*	*1.22*
Dodge Island,					
Fishermans Channel	*25° 46.2'*	*80° 10.1'*	*2.10*	*2.52*	*1.19*
Dinner Key Marina	*25° 43.6'*	*80° 14.2'*	*1.94*	*2.33*	*1.10*

This navigation project has an adequate density of NOAA tide data and has a relatively small tidal range—around 2.5 ft at the ocean entrance. The mean range of tide decreases by 0.16 ft between the Miami Beach Government Cut and inside near the Port of Miami turning basin. Similarly, the 0.14 ft range decrease is small between outside on Miami Beach and Miami Beach Government Cut. The regionally modeled tidal range at a point 3 miles offshore in open ocean could be compared with the range at the Miami Beach pier to see if there is a significant difference. The slope of MLLW can be estimated by looking at the changes in the elevation of MTL relative to MLLW. On the outside, the MTL-MLLW difference is approximately 1.4 ft and decreases to approximate 1.2 ft. inside at the Miami Marina (see Figure E-4). Given the small tide range, and the relatively small tidal range variations between outside and inside channels, the complexity of the variations is not sufficient to warrant a development of a hydrodynamic model. Thus, a straight-line interpolation of the model between observation locations would be acceptable. A regional ocean tidal model (e.g., the ADCIRC 2001 East Coast Model) would be considered in assigning a range value to the model for the outer offshore end of the entrance channel.

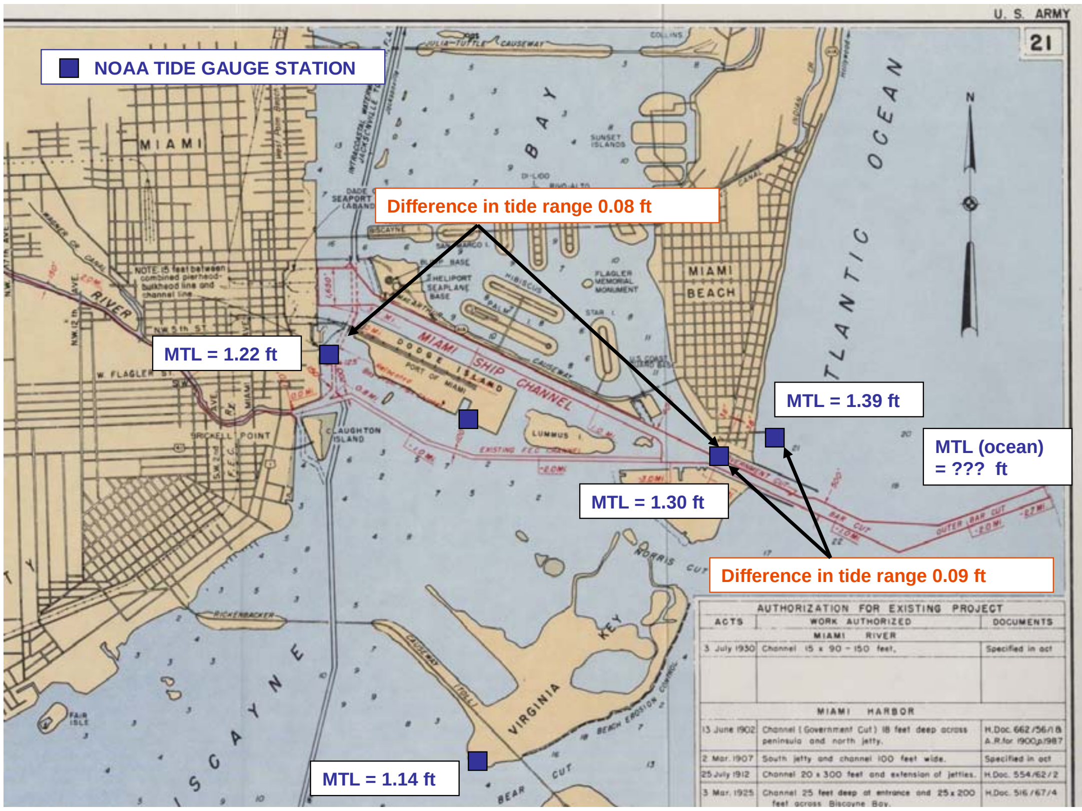

Figure E-4. Tidal Model calibrations at Miami Harbor.

d. Example: Yaquina River (Portland District). A similar analysis can be made for a West Coast project with a larger tide range—Yaquina River, OR (Portland District). The authorized depth varies from 40-ft at the bar, to 18 ft at Yaquina, then 10-ft to Toledo. The estimated mean range of tide and the MTL-MLLW elevation differences from the NOAA tide tables are shown below (in feet):

Yaquina Bay and River	*Lat*	*Long*	*Mn Rge*	*Spg Rge*	*MTL*
Bar at entrance	*44° 37'*	*124° 05'*	*5.9*	*7.9*	*4.2*
Newport	*44° 38'*	*124° 03'*	*6.0*	*8.0*	*4.3*
Southbeach	*44° 37.5'*	*124° 02.6'*	*6.37*	*8.34*	*4.51*
Yaquina	*44° 36'*	*124° 01'*	*6.2*	*8.2*	*4.4*
Winant	*44° 35'*	*124° 00'*	*6.3*	*8.2*	*4.3*
Toledo	*44° 37'*	*123° 56'*	*6.3*	*8.1*	*4.2*

However, a check of the latest NOAA tide station published bench mark information shows that the tide table values are out-of-date and should not be used. In general, if the latitude/longitude files have values only to the nearest degree, as opposed to a tenth of a degree, then the data are from pre-1960 observations. Using the latest information collected in the 1980's by CO-OPS, the table becomes (in feet):

Yaquina Bay and River	*Lat*	*Lon*	*Mn Rge*	*MTL*
Bar at entrance	*44 37*	*124 05*	*5.9*	*4.2*
Newport	*44 36.6*	*124 03.3*	*6.21*	*4.49*
Southbeach	*44 37.5*	*124 02.6*	*6.26*	*4.51*
Weiser Point	*44 35.6*	*124 00.5*	*6.46*	*4.57*
Toledo	*44 37.0*	*123 56.2*	*6.87*	*4.71*

Thus the older results show much less variability in the tide range than the updated, more recent data. The table and Figure E-5 shows that the range of tide increases by almost 1.0 ft. from outside to upriver at Toledo, and there is a 0.50 ft. slope in MLLW relative to MTL. This may be an area where a hydrodynamic model may prove useful to account for the non-linear changes in the tide going upriver.

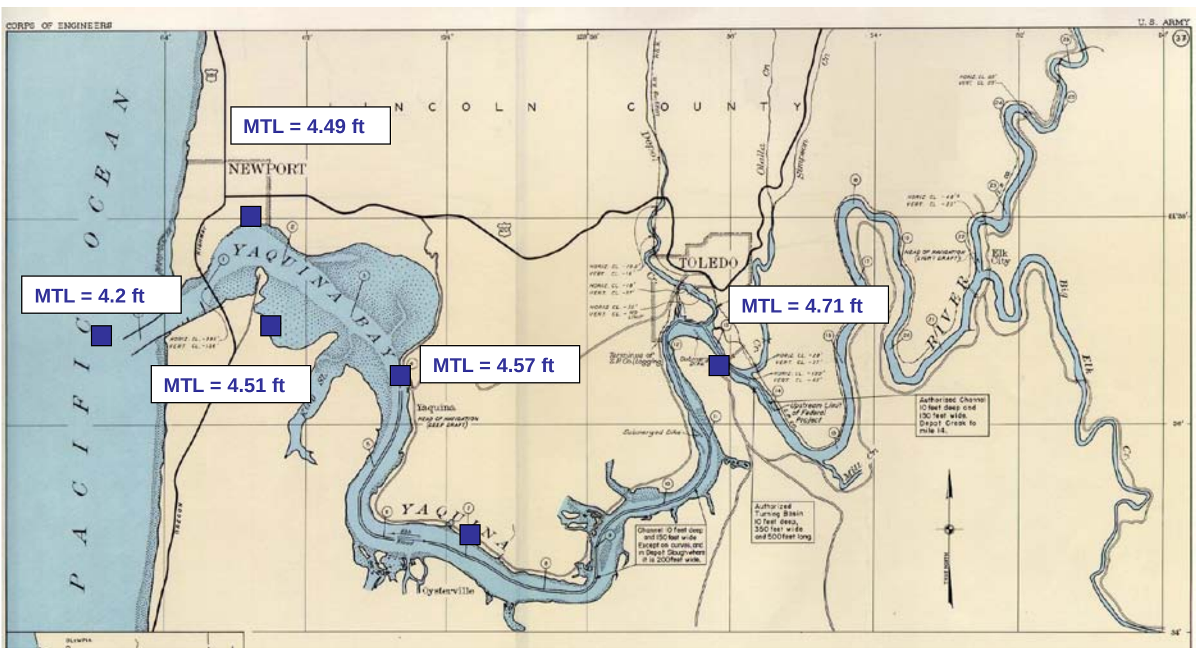

Figure E-5. Tidal Model calibrations at Yaquina River, OR.

e. Example: Portsmouth, NH (New England District). The following New England District project (Portsmouth, NH) is typical of a large tidal range variance—approximately 8 ft. MTL variations at various points are shown in Figure E-6. Predicted tide ranges are shown below.

Portsmouth Harbour	*Lat*	*Long*	*Mn Rge*	*Spg Rge*	*MTL*
Jaffrey Point	*43° 03.4'*	*70° 43.9'*	*8.7*	*10.0*	*4.7*
Gerrish Island	*43° 04.0'*	*70° 41.7'*	*8.7*	*10.0*	*4.7*
Fort Point	*43° 04.3'*	*70° 42.7'*	*8.6*	*9.9*	*4.6*
Kittery Point	*43° 04.9'*	*70° 42.2'*	*8.7*	*10.0*	*4.7*
Seavey Island	*43° 05'*	*70° 45'*	*8.1*	*9.4*	*4.4*
Portsmouth	*43° 04.7'*	*70° 45.1'*	*7.8*	*9.0*	*4.2*

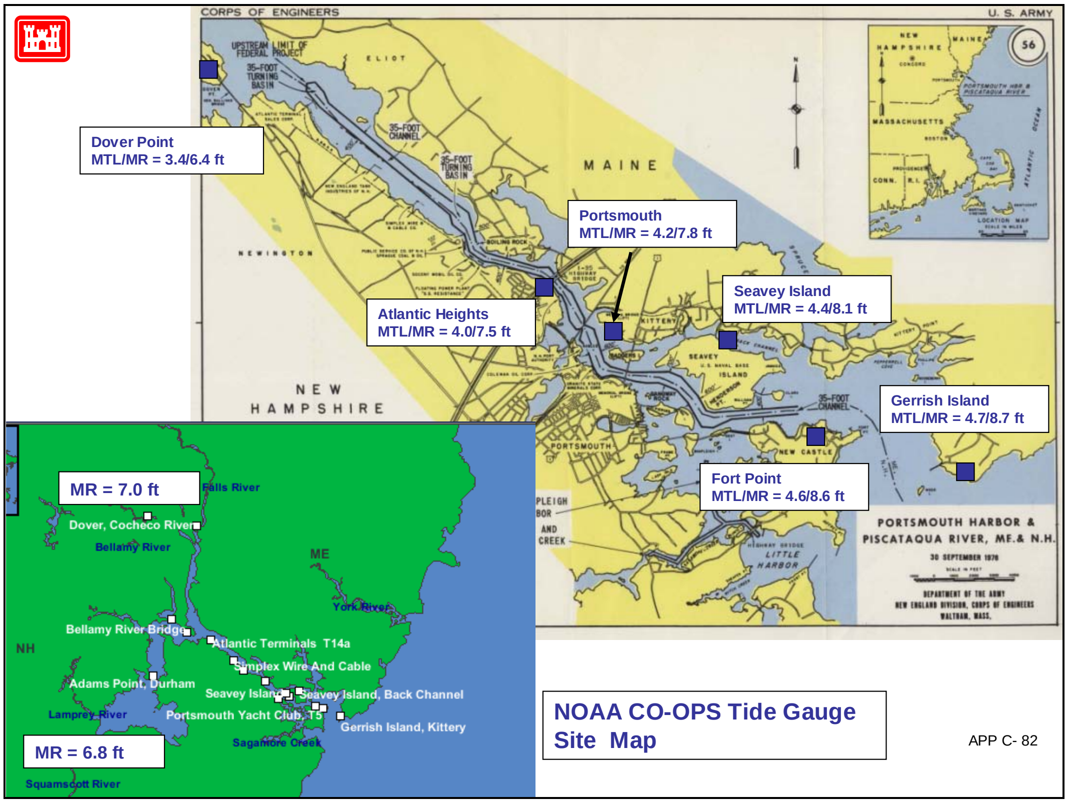

Figure E-6. Tidal Model calibrations at Portsmouth, NH.

E-5. Tidal Zoning Models. Discrete tidal zones are constructed based on knowledge of the tide at shore-based historical stations and estimated positions of co-tidal lines for range and time of tide. For most NOAA applications the resolution of the zoning has been to construct a zone polygon for every 0.2-foot change in range and every 0.3-hour change in time of tide. For many tidally complex areas (such as around Key West for instance) tide zones with higher resolution are used. Tidal zoning errors are considered random errors although they have a certain periodic nature and not a normal statistical distribution. Zoning errors also are characterized by two components: a time correction and a range ratio correction to observations from a nearby tide station. Maximum zoning errors for each project are estimated by simultaneously comparing tide curves constructed from time and range corrections to historical tide station observations. Statistics of the residuals are then analyzed to estimate the error in the zoning for the entire project.

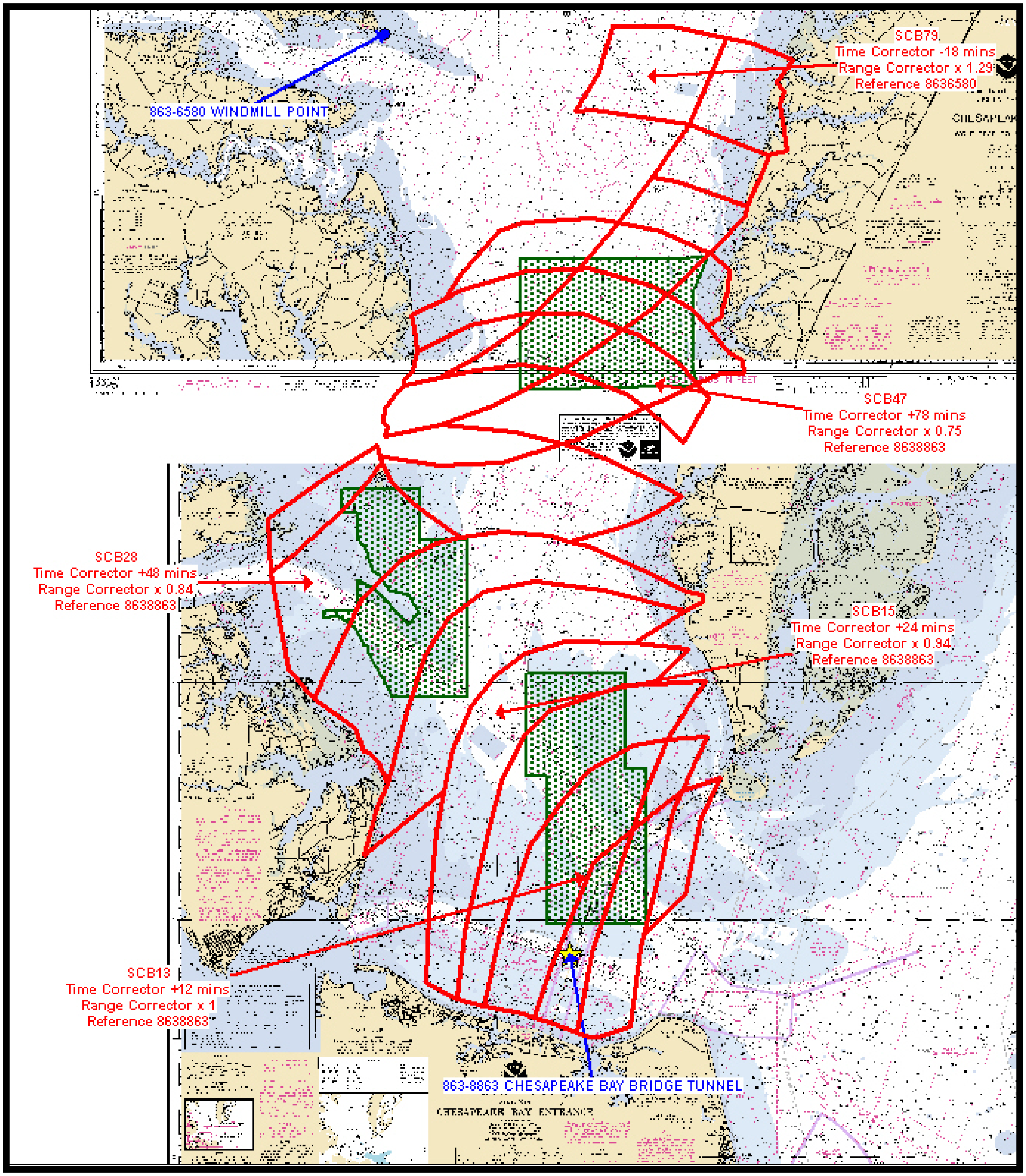

Figure E-7. The discrete tidal zones constructed from the co-tidal lines and the survey areas in lower Chesapeake Bay.

a. Tidal zoning errors. There are inherent errors in the application of discrete tidal zoning: 1) discontinuities at the edge of the zones; 2) resolution in areas of complex tidal characteristics, where the location and number of zones is not adequate to describe the changes in the tide over the survey area; 3) where large time corrections and large range ratios are required; and 4) the fact that placement of the zones becomes subjective when the co-tidal lines are based upon inconsistent or inadequate source data. Figure E-7 illustrates an application for tidal zoning in Chesapeake Bay—in particular for areas in the middle of the bay where no RTK or RTN coverage is available. Where RTK/RTN coverage is available only the co-range model would have application.

b. Discussion of applications. The major contributors to the tides error budget are the datum error, which contributes as a systematic bias, and the tidal zoning error, which contributes as a random error. In practice the datum error is reduced with longer data series. Errors can be very significant if less than 30-days of data are observed. Substantial reductions in error from those of a 30-day series are not realized until one-year of data are collected. For tidal modeling purposes, NOAA gage datums, (or acceptable datums from another agency's long-term gages) will be assumed as absolute—no effort will be considered in improving the accuracy of existing datums by extending gage periods. The tidal zoning error can be reduced by lessening the amount of time and range correction needed by establishing more tide stations for use in direct control of the survey.

c. TCARI. Use of NOAA's "Tidal Constituent and Residual Interpolation" (TCARI) procedures can also reduce tidal zoning errors. Project planning an implementation are focused on finding the practical balance between the number of tide stations required and the amount of tidal zoning required. This in turn depends upon the complexities of the tidal characteristics in the area along with the resources and logistics required to establish and maintain tide stations. Calibrated tide gages that are configured and installed to minimize dynamic errors result in the measurement errors usually being minor contributors to the tides error budget. The estimated total tides error can then be root-summed-squared with all of the other hydrographic survey error sources to estimate the total survey error budget.

d. Example tidal zoning project. Even in these larger tidal ranges the gage density appears sufficient to model the MLLW datum variation by interpolation throughout the deep draft portion of the project. Figure E-8 is a graphic showing the CO-OPS discrete tidal zoning scheme for Porthmouth, NH. If RTK procedures were not employed at this project site, time and range correctors for each zone would be applied to an appropriate tide station installed in the harbor to account for time and range changes in the project area. The closest NOAA operating NWLON stations are Boston, MA and Portland, ME.

E-6. <u>Hydrodynamic Tidal Modeling of Navigation Projects</u>.

a. From the above, it would appear that many USACE deep-draft navigations will have a sufficient density of NOAA CO-OPS tidal data such that spatially interpolated models will be adequate. Interpolated models can be:

(1) a linear interpolation of elevation relationships over relatively short distances.

(2) a discrete tidal zoning interpolation based on changes in cotidal lines over the survey area.

(3) a continuous tidal zoning interpolation model such as TCARI.

Where this is not the case, then a hydrodynamic tidal model may have to be generated to define the MLLW datum plane throughout a project.

b. The technical process of developing a hydrodynamic tidal model of a typical coastal inlet, and calibrating that model to one or more fixed gages, is relatively straightforward and models for performing this are well documented in part II of EM 1110-2-1100 (*Coastal Engineering Manual*) and other sources. Many USACE navigation projects have been extensively studied over the years and existing numerical models may be readily utilized to assess the tidal datum relationships—e.g., activities studied under the ERDC/CHL "Diagnostic Modeling System."

c. Projects requiring hydrodynamic tidal modeling to define the MLLW datum can be accomplished by any number of organizations. Some of these include:

(1) District Hydrology & Hydraulics (H&H) sections.

(2) Coastal Engineering A-E firms.

(3) NOAA CSDL VDatum Team.

(4) ERDC/Coastal and Hydraulics Laboratory (CHL).

d. Each of the above options will have different approaches, costs, and turn-around response. Cost estimates for this modeling effort can be obtained from any of these organizations. These costs may include gaging programs which will have to be obtained from NOAA. Actual gage installation can be accomplished via an A-E contract with a coastal engineering firm or through NOAA.

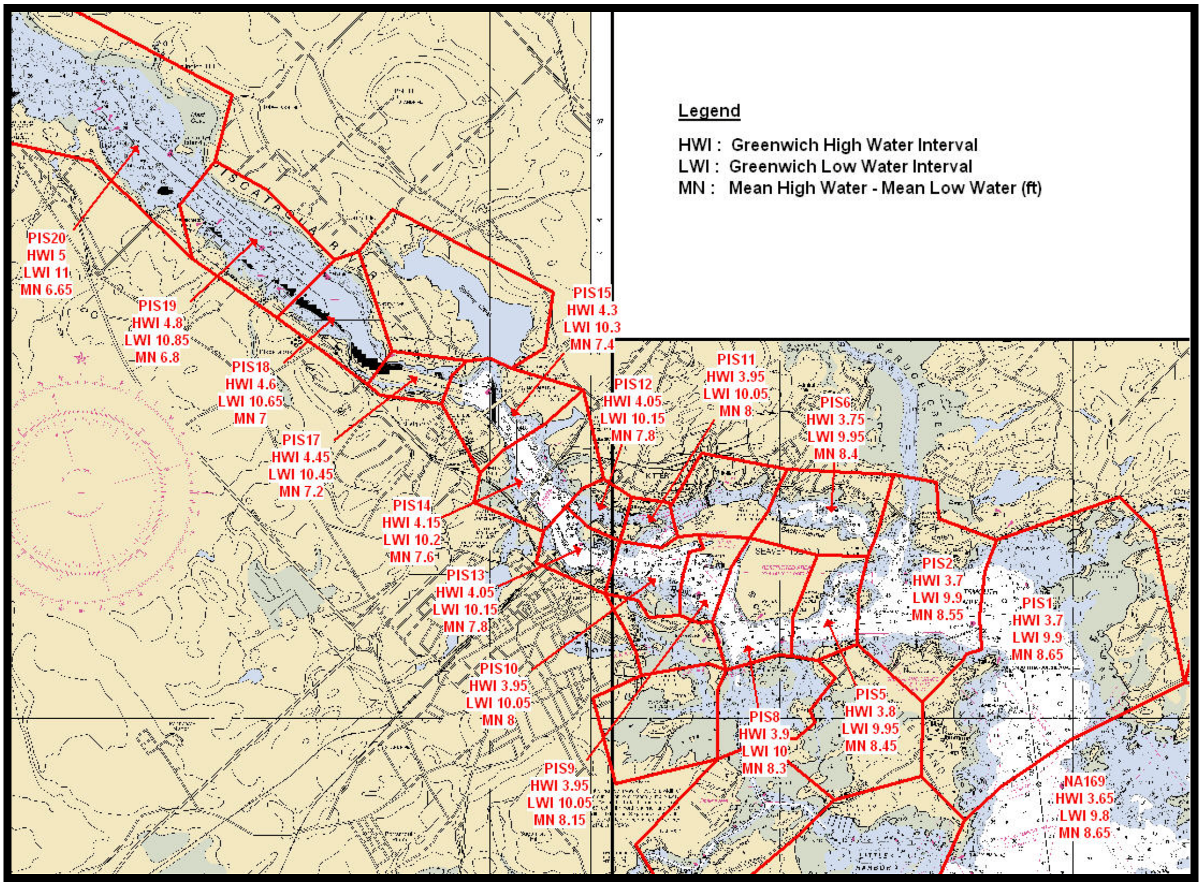

Figure E-8. NOAA discrete tidal zoning scheme for Portsmouth, New Hampshire.

APPENDIX F

Canaveral Harbor, FL— Establishing a PPCP and Tidal Datum Reference when Adequate NOAA Gage Data Exists (Jacksonville District)

F-1. <u>Purpose and Background</u>. This appendix illustrates the establishment of references to the NSRS and the NWLON for a typical navigation project. Figure F-1 depicts a Jacksonville District deep draft project that has been adequately referenced to a NOAA tidal datum and the NSRS. This project supports US Navy, commercial, and cruise ship interests. Underkeel clearances on the US Navy Trident portion of this project are considered critical. A shallow-draft barge canal exists west of the lock at the end of the deep-draft project. That portion of the project illustrates an application where the tidal range is small to negligible and a MSL reference datum is used.

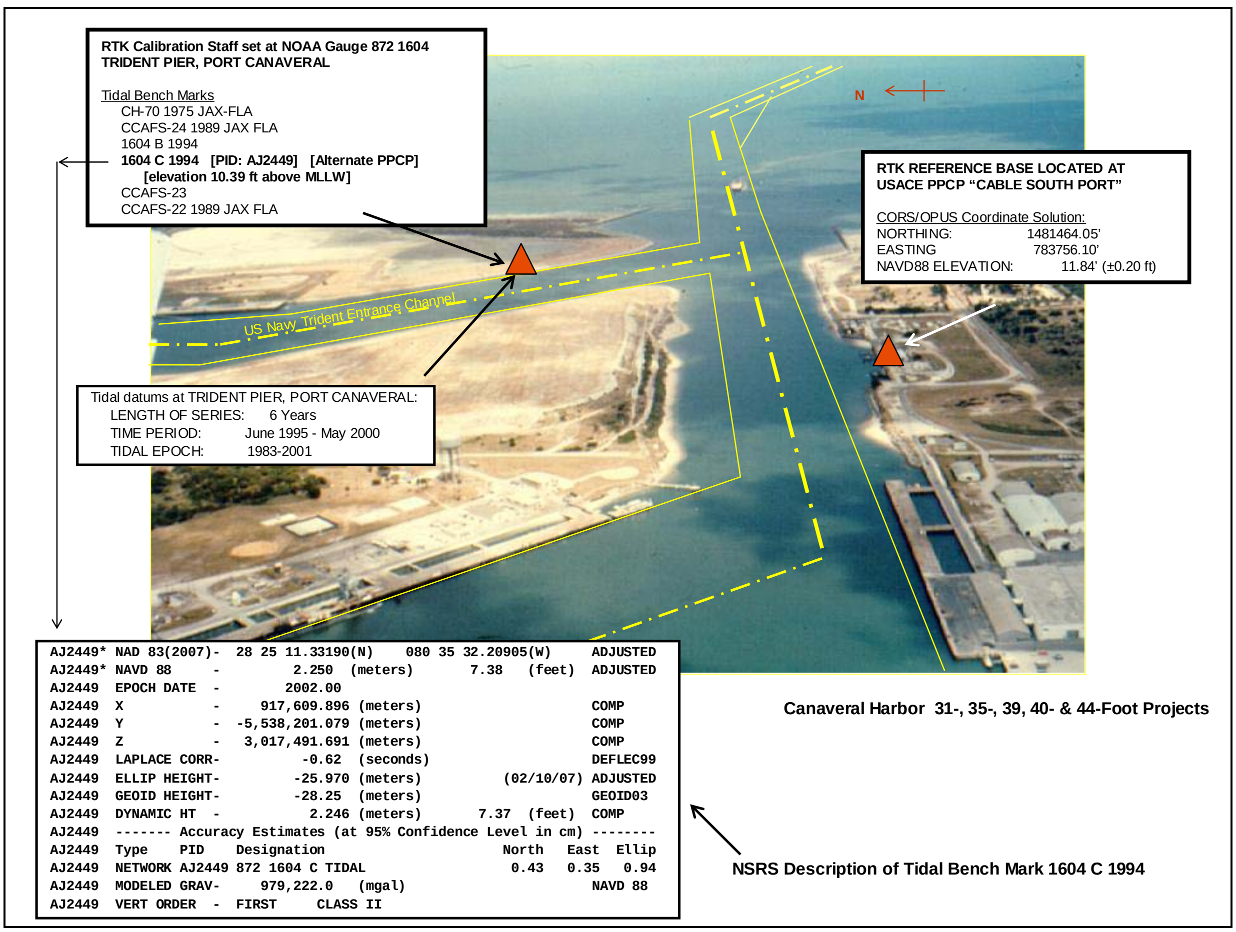

Figure F-1. Tidal PBM and RTK PPCPs established at Canaveral Harbor, FL. (Jacksonville District)

This site contains an active NOAA gage (TRIDENT PIER, PORT CANAVERAL) on the north side of the entrance at the Trident Entrance Channel. This gage has six reference PBMs, one of which (1604 C 1994) is listed in the NSRS with adjusted First-Order NAVD88 vertical control. An existing USACE bench mark (CABLE SOUTH PORT) is located on the south side of the entrance channel. This bench mark had a NGVD29 elevation of uncertain origin and was previously used for tidal corrections at the project. A RTK base station situated on either side of the entrance will provide survey and dredge position coverage to the outer end of the project in the ocean.

F-2. Canaveral Harbor: Deep-Draft Tidal Project.

a. Project description. (See Figures F-2 and F-3). Maintenance of an entrance channel 41 feet deep and 400 feet wide; an inner channel 40 feet and 400 feet wide; a 1200 foot diameter turning basin 39 feet deep; a channel 39 feet deep and 400 feet wide for an 1800 foot length; enlargement of barge channel to 12 feet deep and 125 feet wide to the Intracoastal Waterway; a channel extension 31 feet by 300 feet by 1,500 feet dredged west of turning basin; and a barge lock 90 feet wide and 600 feet long west of the harbor dike; and two entrance jetties to the 12-foot contour. Length of project is about 11.5 miles. The entrance channel and part of the inner channel have been deepened to 44 feet for the Navy's TRIDENT Project.

Figure F-2. Canaveral Harbor project: Trident basin, cruise ship basin, and barge canal lock.

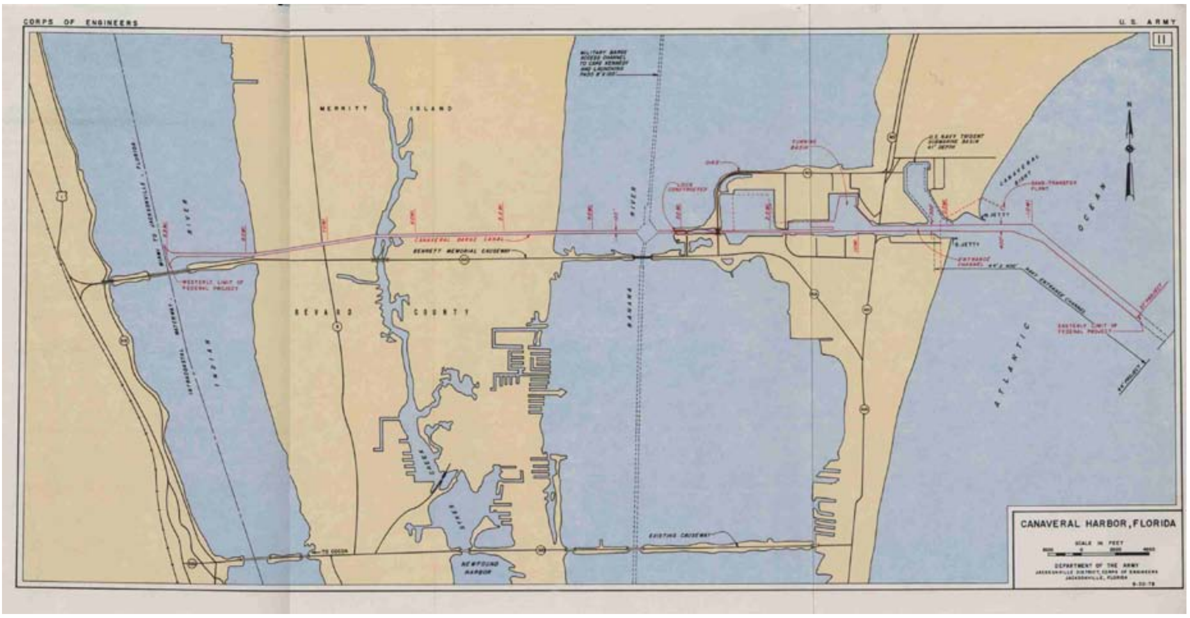

Figure F-3. Canaveral Harbor project map.

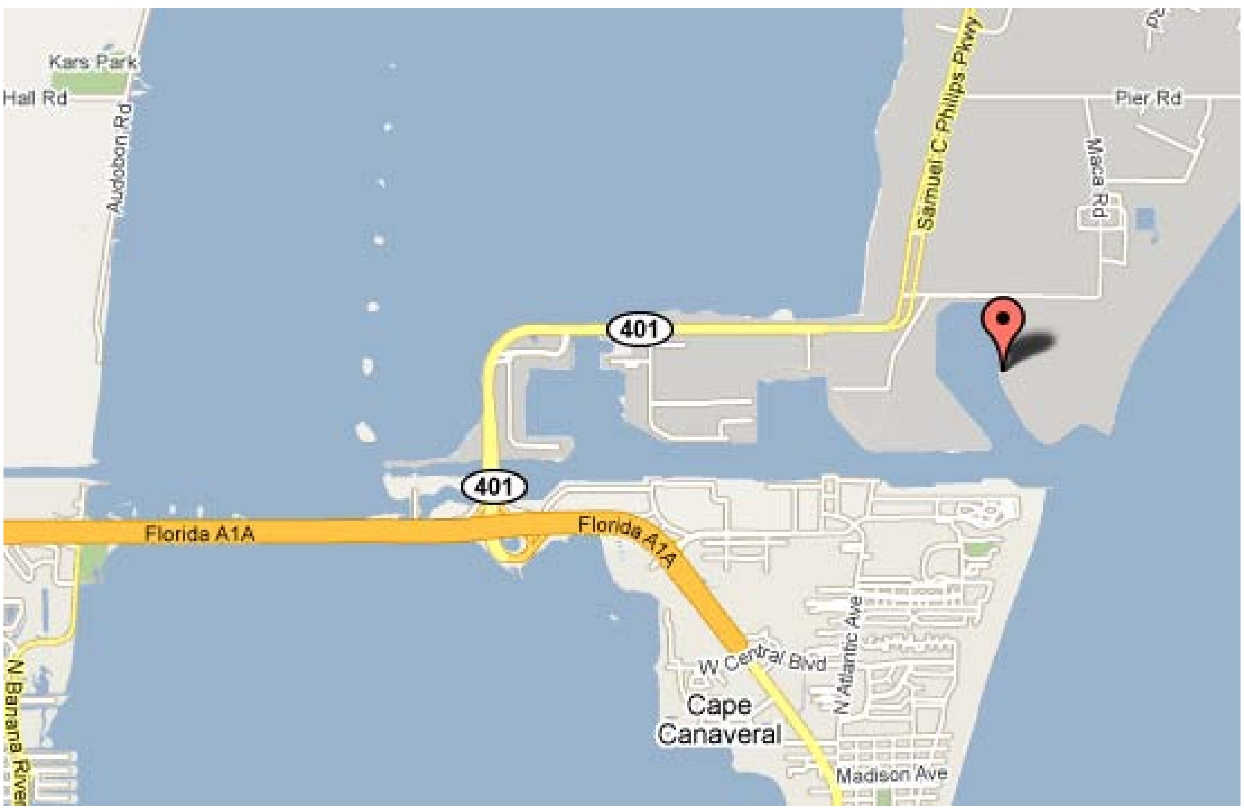

Figure F-4. NOAA gage "TRIDENT PIER" in Trident Basin.

b. Tidal datum reference. NOAA CO-OPS data for TRIDENT PIER gage (Figure F-4) indicates it is relatively current with a 6-year recording series from 1995 to 2000. No significant deepening or entrance modifications have been made since that period. Thus this NOAA gage was deemed as acceptable for the primary datum reference on this project. (Had an existing, or unsuitable, tide station not been located at this project site, then a short-term gage would have had to be installed to determine a reference datum following NOAA CO-OPS standards in "*Computational Techniques for Tidal Datums Handbook*" (NOAA 2003).

(1) Tidal reference bench marks. Three of the published tidal bench marks at the TRIDENT PIER gage were recovered and Third-Order level runs between these marks indicated they were stable internally to within 0.02 ft. Recovery notes on these tidal bench marks were transmitted to NOAA CO-OPS. (As above, had no reference tidal bench marks been recovered, then for all practical purposes, the gage is lost and new tidal observations would be required).

(2) Calibration tide staff. A tide staff was set at the TRIDENT PIER site relative to published MLLW datum on the tidal PBMs. The staff zero was set at MLLW so visual readings were direct elevations of the water surface above MLLW.

c. Primary "PPCP " tidal reference mark. Tidal bench mark "1604 C 1994" at the TRIDENT PIER gage site has a solid First-Order (II) orthometric elevation and observed ellipsoid height observations—see the NSRS extract in Figure F-1. The estimated 95% confidence in the observed ellipsoid height is less than 1 cm. This tidal PBM, with published geodetic, ellipsoidal, and tidal reference elevations, is the obvious choice as the designated PPCP for this project. No additional field survey observations would be needed at this project if this point were used as an RTK base station. Since GPS observations were once made at this mark (per NSRS description), its use as an RTK base is presumed adequate. However, due to site access restrictions at this military site, an alternate RTK base station PBM was established on the south side of the channel at USACE PBM "CABLE SOUTH PORT."

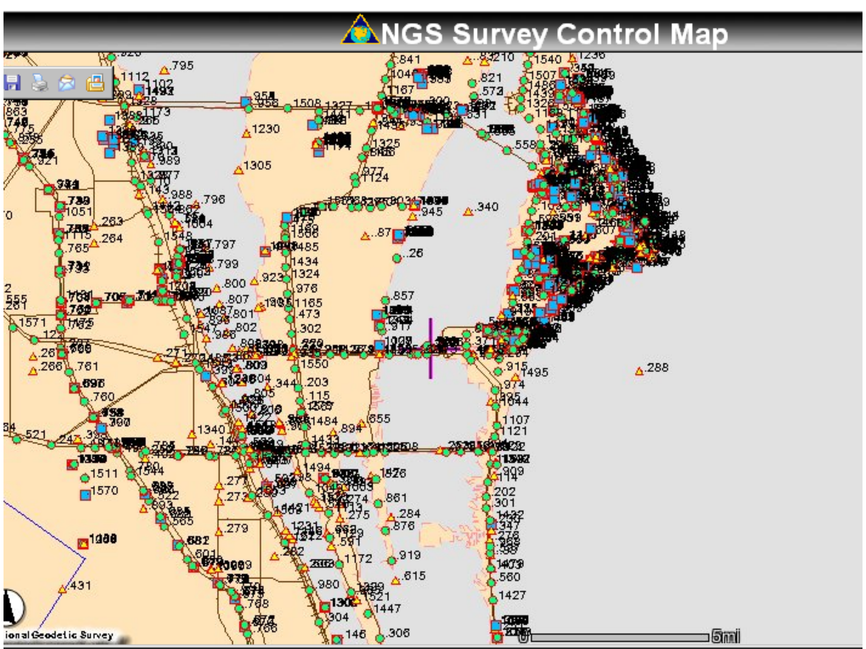

Figure F-5. NGS control network in Cape Canaveral area.

d. GPS surveys to position a primary RTK base station. Obtaining a PPCP for this project was not an issue at this site, given the dense NSRS network in the region—see Figure F-5. USACE PBM " CABLE SOUTH PORT" on the south side of the channel was positioned using CORS/OPUS techniques. An 8-hour session of CORS observations were recorded and transmitted to OPUS for reduction. The results along with descriptive data were transmitted through OPUS-DB for published input to the NSRS. A static baseline was simultaneously observed from tidal PBM 1604 C 1994 as a check on the CORS/OPUS solution. These observations were performed concurrently with hydrographic survey observations so no additional field effort was required to perform these control surveys. PBM "CABLE SOUTH PORT" is thus the designated PPCP for this project.

e. MLLW tidal model. Since VDatum coverage did not exist at this project site in 2009, an estimated tidal model was required. Based on comparisons in diurnal tide ranges between TRIDENT PIER and other NOAA gages in the surrounding offshore region (i.e., ocean pier gages north and south of Canaveral—see Figure F-6), there did not appear to be any significant tidal MLLW datum gradient between the ocean and the interior channels up to the Trident Basin. This may be due to the relatively wide entrance. A constant NAVD88-MLLW difference of 3.01 ft was therefore assumed constant throughout the project. This difference was computed using data at Tidal bench mark "1604 C 1994"—i.e., 10.30 ft NAVD88 - 7.38 ft MLLW. Future VDatum coverage is not expected to significantly modify this constant model since these NOAA gages would likely be used to develop the VDatum model.

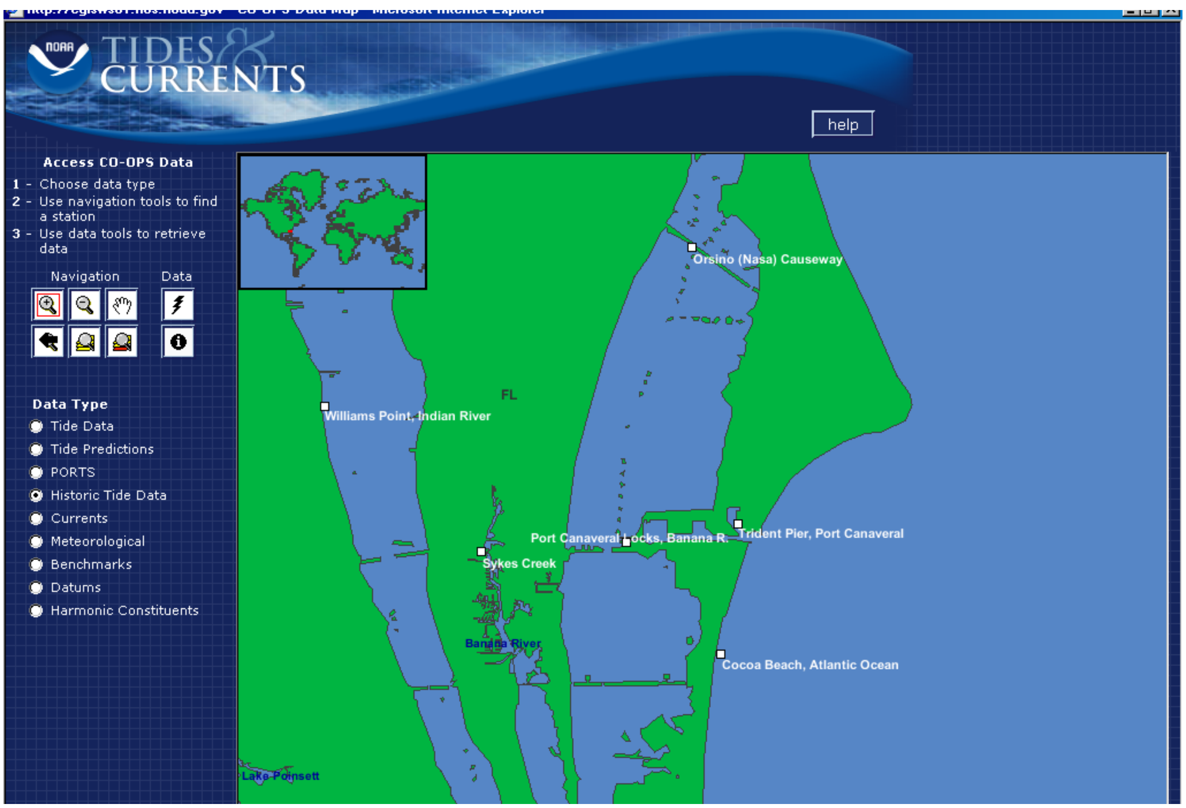

Figure F-6. NOAA tide gage data vicinity of Cape Canaveral.

f. Measurement & payment survey procedures. The RTK base station is initialized at PBM "CABLE SOUTH PORT" using the newly NSRS published coordinates based on CORS/OPUS solutions. RTK observations are adjusted in the vessel positioning software to correct for the NAVD88-MLLW datum difference—i.e., the "K" term referenced Chapter 4. Likewise geoid height variations ("N") over the project are automatically adjusted in the GPS acquisition or processing software.

F-3. Canaveral Lock and Barge Canal to Banana River and Indian River: Non-Tidal. This section illustrates procedures for referencing construction datums in areas with little or imperceptible tide ranges. Tidal influences are small in the areas (Banana River and Indian River) west of the Canaveral Lock, given the nearest inlets north and south are over 20 miles distant. Figure F-7 depicts available tidal information from the Florida Department of Environmental Protection, Land Boundary Information System (LABINS). Survey reaches outside (west of) the Canaveral Lock are depicted as non-tidal, based on LABINS and NOAA gage data. It was decided that dredging elevations within these reaches shall be referred to Mean Sea Level (MSL) as the reference construction datum. Previously, dredging datums in this area were related to MLLW relative to NGVD29, as outlined in the 31 August 2007 CEPD Report for this project:

"Water surface elevation measurement tidal and geoid undulation corrections have not been hydrodynamically modeled or calibrated throughout the [Canaveral] project area. Portions of the project area may have not been converted from MLW to MLLW datum. Portions are still on NAD27 horizontal datum—Barge Canal. Currently, water surface elevation corrections for dredging measurement & payment are based on extrapolated staff gage readings set from unmodeled benchmarks that are set from benchmarks of uncertain origin, are not referenced to the NSRS, and are referenced to the superseded NGVD29 datum. Project framework and control documents do not define references or relationships between these benchmarks and NOAA tidal gages or tidal benchmarks."

"NOAA tidal PBM G 215 is used for extrapolating water surfaces in the Barge Canal, however the USACE 8.42 ft elevation differs from NGS published 9.92 ft (NGVD29). An unmodeled constant MLLW datum surface (0.5 ft below NGVD29) is assumed throughout the Barge Canal project area. The source of this corrector is uncertain. The tidal range in this project is small—a constant difference may prove to be valid once a model is developed."

a. The 2007 CEPD report made the following recommendation regarding the Canaveral Barge Canal:

"Tidal range gradients west of the Canaveral Lock should also be assessed and developed if significant, per [HQUSACE] guidance. NOAA gage data needs to be obtained up to the IWW (J—M) to determine the low water datum and whether any significant tidal gradient exists."

b. This section illustrates corrective actions taken based on the 2007 CEPD recommendation.

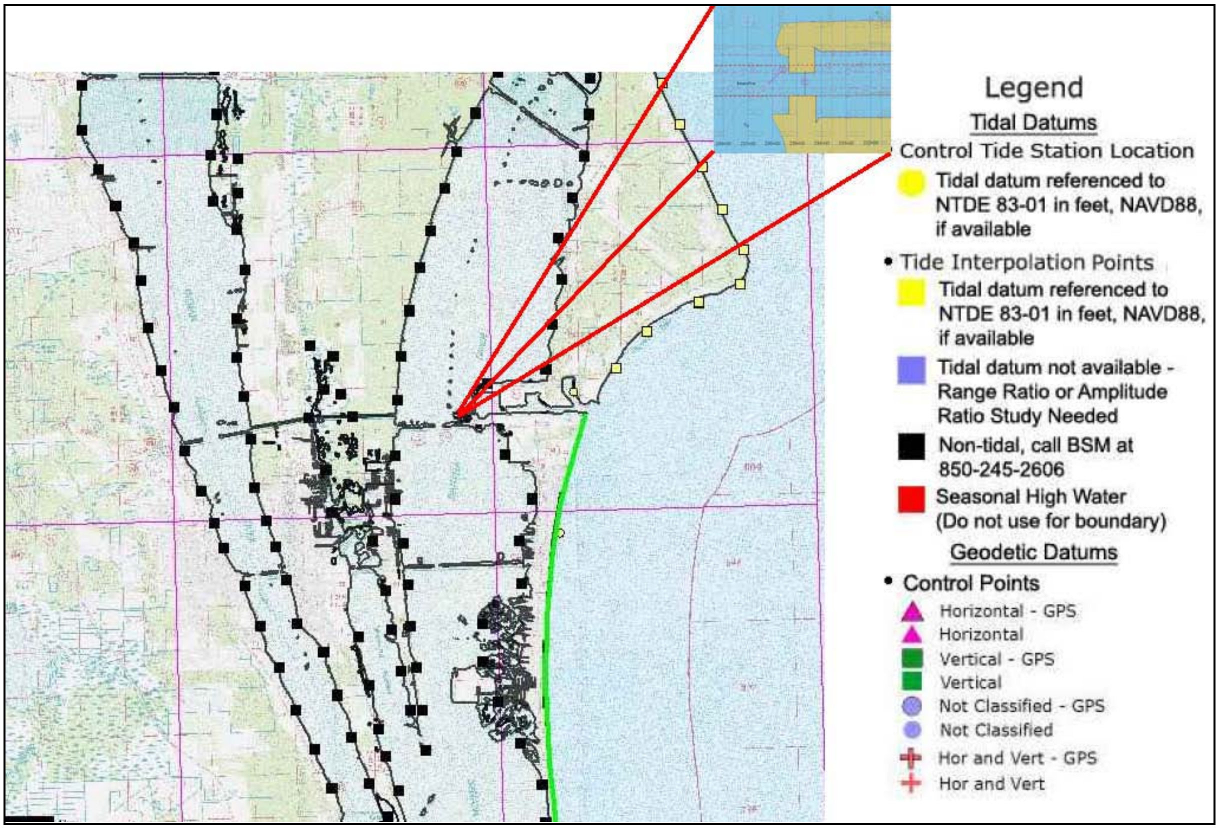

Figure F-7. Canaveral Barge Canal Datum Determination: Non-Tidal gages in Banana River and Indian River Region (Florida Department of Environmental Protection, Land Boundary Information System (LABINS)).

c. The NAVD88-MSL relationship was determined to be 0.70 ft within survey reaches west of the Canaveral Lock chamber (Station 235+00 of the Canaveral Barge Canal). This relationship was determined by performing a distance weighted interpolation from NOAA gages 872-1533 (Orsino Causeway), 872-1456 (Titusville, Indian River), and 872-1456 (Pineda, Indian River)—see Figures F-8 and F-9. Published NOAA gage information at these locations depicts the NAVD88 to MSL relationship—see NOAA datasheets in Figures F-10 and F-11 at the end of this section.

CANAVERAL BARGE CANAL DATUM

NOAA GAGE	NAVD88-MSL ft (x)	DISTANCE MILES (d)	WEIGHT (1/d)	(x) * (1/d)	Weighted Varience	w * (χ - x)^2
8721533	0.690	7.414	0.093067	0.064216		0.000028
8721749	0.730	13.717	0.053219	0.038850		0.000027
8721456	0.702	17.41	0.040322	0.028306		0.000001
MEAN (χ)	0.707	SUM	0.186607	SUM 0.131372	SUM	0.000056
		weighted mean	**0.704**		**weighted std dev**	**0.017**

Figure F-8. Computation of NAVD88-MSL difference west of Canaveral Lock.

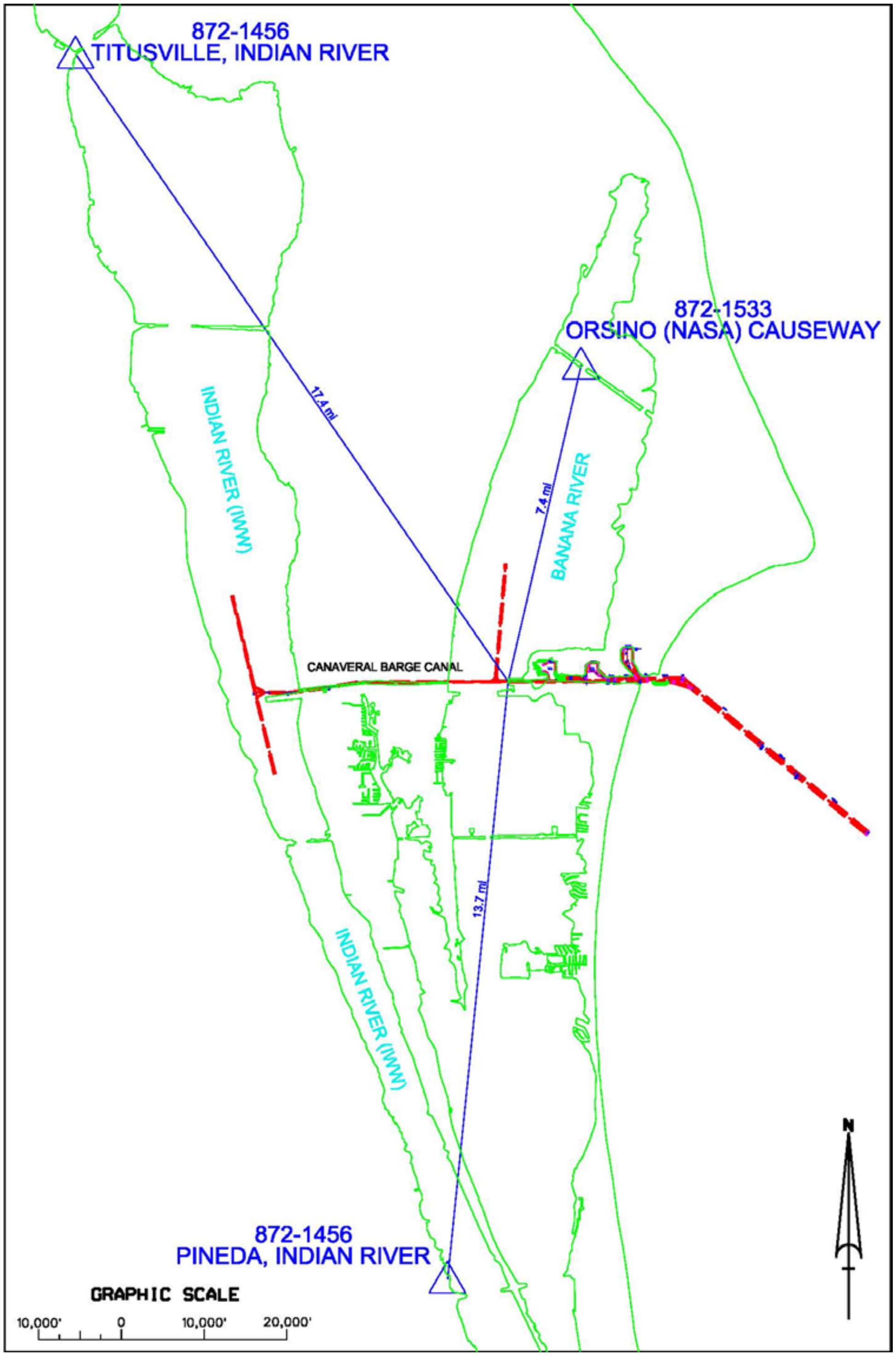

Figure F-9. Spatial interpolation of NAVD88-MSL relationship west of Canaveral Lock.

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Mar 25 2010 13:17        ELEVATIONS ON STATION DATUM
                        National Ocean Service (NOAA)
Station:  8721456                                              T.M.:        75 W
Name:     TITUSVILLE, INDIAN RIVER, FL                         Units:       Feet
Status:   Accepted                                             Epoch:  1983-2001
          Datum         Value  Description
          ---------  --------  --------------------------------------
          MHHW                 Mean Higher-High Water
          MHW                  Mean High Water
          DTL                  Mean Diurnal Tide Level
          MTL                  Mean Tide Level
          MSL            4.01  Mean Sea Level
          MLW                  Mean Low Water
          MLLW                 Mean Lower-Low Water
          GT                   Great Diurnal Range
          MN                   Mean Range of Tide
          DHQ                  Mean Diurnal High Water Inequality
          DLQ                  Mean Diurnal Low  Water Inequality
          HWI                  Greenwich High Water Interval (in Hours)
          LWI                  Greenwich Low  Water Interval (in Hours)
          NAVD           4.71  North American Vertical Datum
          Maximum        5.08  Highest Water Level on Station Datum
          Max Date   19701020  Date Of Highest Water Level
          Max Time      00:00  Time Of Highest Water Level
          Minimum        2.41  Lowest  Water Level on Station Datum
          Min Date   19780204  Date Of Lowest Water Level
          Min Time      00:00  Time Of Lowest Water Level
```

```
Mar 25 2010 13:17        ELEVATIONS ON STATION DATUM
                        National Ocean Service (NOAA)
Station:  8721533                                              T.M.:        75 W
Name:     ORSINO (NASA) CAUSEWAY, FL                           Units:       Feet
Status:   Accepted                                             Epoch:  1983-2001
          Datum         Value  Description
          ---------  --------  --------------------------------------
          MHHW                 Mean Higher-High Water
          MHW                  Mean High Water
          DTL                  Mean Diurnal Tide Level
          MTL                  Mean Tide Level
          MSL            3.20  Mean Sea Level
          MLW                  Mean Low Water
          MLLW                 Mean Lower-Low Water
          GT                   Great Diurnal Range
          MN                   Mean Range of Tide
          DHQ                  Mean Diurnal High Water Inequality
          DLQ                  Mean Diurnal Low  Water Inequality
          HWI                  Greenwich High Water Interval (in Hours)
          LWI                  Greenwich Low  Water Interval (in Hours)
          NAVD           3.88  North American Vertical Datum
          Maximum              Highest Water Level on Station Datum
          Max Date             Date Of Highest Water Level
          Max Time             Time Of Highest Water Level
          Minimum              Lowest  Water Level on Station Datum
          Min Date             Date Of Lowest Water Level
          Min Time             Time Of Lowest Water Level
```

Figure F-10. NOAA datasheets for tide gages 872 1456 and 872 1533.

Mar 25 2010 13:18 ELEVATIONS ON STATION DATUM
National Ocean Service (NOAA)

Station: 8721749 T.M.: 75 W
Name: **PINEDA, INDIAN RIVER, FL** Units: Feet
Status: Accepted Epoch: 1983-2001

Datum	Value	Description
MHHW		Mean Higher-High Water
MHW		Mean High Water
DTL		Mean Diurnal Tide Level
MTL		Mean Tide Level
MSL	2.71	Mean Sea Level
MLW		Mean Low Water
MLLW		Mean Lower-Low Water
GT		Great Diurnal Range
MN		Mean Range of Tide
DHQ		Mean Diurnal High Water Inequality
DLQ		Mean Diurnal Low Water Inequality
HWI		Greenwich High Water Interval (in Hours)
LWI		Greenwich Low Water Interval (in Hours)
NAVD	3.44	North American Vertical Datum
Maximum		Highest Water Level on Station Datum
Max Date		Date Of Highest Water Level
Max Time		Time Of Highest Water Level
Minimum		Lowest Water Level on Station Datum
Min Date		Date Of Lowest Water Level
Min Time		Time Of Lowest Water Level

To refer Water Level Heights to a Tidal Datum, apply the desired Datum Value.

Click HERE for further station information including New Epoch products.

To refer Water Level Heights to either
NGVD (National Geodetic Vertical Datum of 1929) or
NAVD (North American Vertical Datum of 1988), apply the values located at:
National Geodetic Survey

Figure F-11. NOAA datasheet for tide gage 872 1749.

F-4. RTK Coverage. Figure F-12 depicts proposed RTK coverage for this projects. Base stations would be located at CABLE SOUTH PORT and CBC-101. The published NSRS datasheet for CABLE SOUTH PORT is shown on Figure F-13.

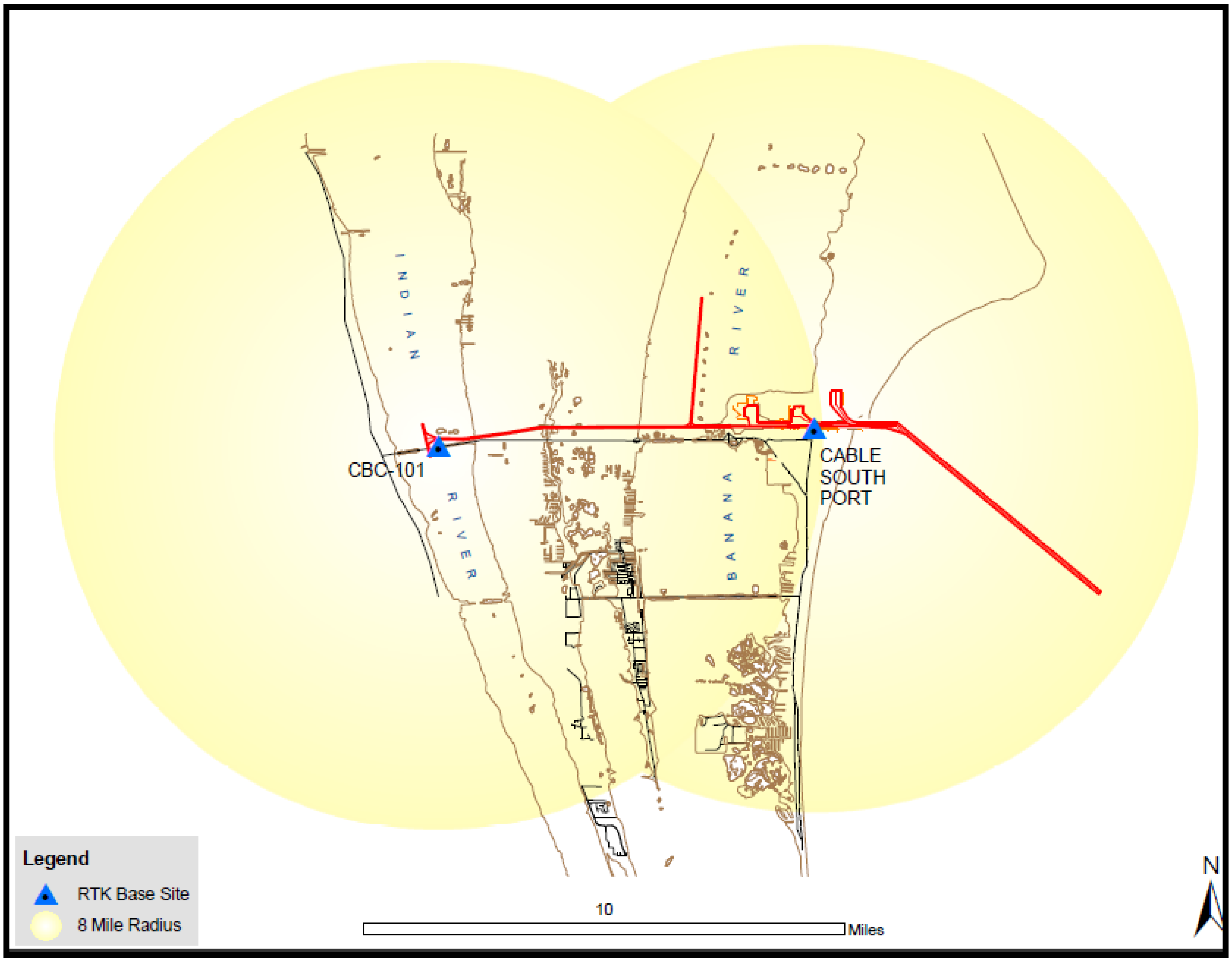


Figure F-12. RTK scheme for Canaveral Harbor.

SURVEY DATASHEET (Version 1.0)

PID: BBBV18
Designation: CABLE SOUTH PORT
Stamping: CABLE SOUTH PORT 1988
Stability: Monument will probably hold position well
Setting: Retaining wall or concrete ledge
Description: From the intersection of I-95 and FL-528, proceed East on FL-528 for 12.6 miles to Astronaut BLVD/A1A S. Proceed South on A1A for 0.6 miles. Take Port Canaveral exit and drive 0.9 miles to N. Atlantic BLVD. Turn left into Port Entrance. Proceed through Port security and take first right onto Herring St. Drive 0.2 miles and turn right onto Glen Cheek Dr. Proceed approximately 0.2 miles to mark on left. Mark is located on a seawall approximately 65 feet east of the centerline of Jetty Park Rd.
Observed: 2010-03-30T16:35:00Z
Source: OPUS - page5 0909.08

Close-up View

REF_FRAME:	EPOCH:	SOURCE:	UNITS:	SET PROFILE	DETAILS
NAD_83(CORS96)	2002.0000	NAVD88 (Computed using GEOID09)	m		

LAT:	28° 24' 30.91334"	± 0.018 m
LON:	-80° 36' 11.16570"	± 0.020 m
ELL HT:	-24.601	± 0.043 m
X:	916660.763	± 0.019 m
Y:	-5538959.611	± 0.033 m
Z:	3016397.958	± 0.035 m
ORTHO HT:	3.639	± 0.046 m

	UTM 17	**SPC** 901(FL E)
NORTHING:	3142529.061m	451551.147m
EASTING:	538876.040m	238889.309m
CONVERGENCE:	0.18882919°	0.18882919°
POINT SCALE:	0.99961865	0.99995983
COMBINED FACTOR:	0.99962251	0.99996370

Figure F-13. Portion of NSRS Datasheet for USACE PPCP "CABLE SOUTH PORT."

APPENDIX G

Fort Fisher Shore Protection and Beach Stabilization Project (Wilmington District)

G-1. Purpose. This appendix contains an example of a Wilmington District project that has been adequately referenced to the current NSRS orthometric datum and to the local tidal datum. The project consists of a 3,000 ft stone revetment sector that has an established reference baseline with local control on a legacy NGVD29 vertical datum. Beach monitoring surveys are performed in an area some two miles to the south of the revetment area, as shown on Figure G-1. The mean range of tide is 4.2 ft and the maximum known storm tide is 10.7 feet above MSL. Average annual shoreline retreat rate is approximately 15 ft in this area.

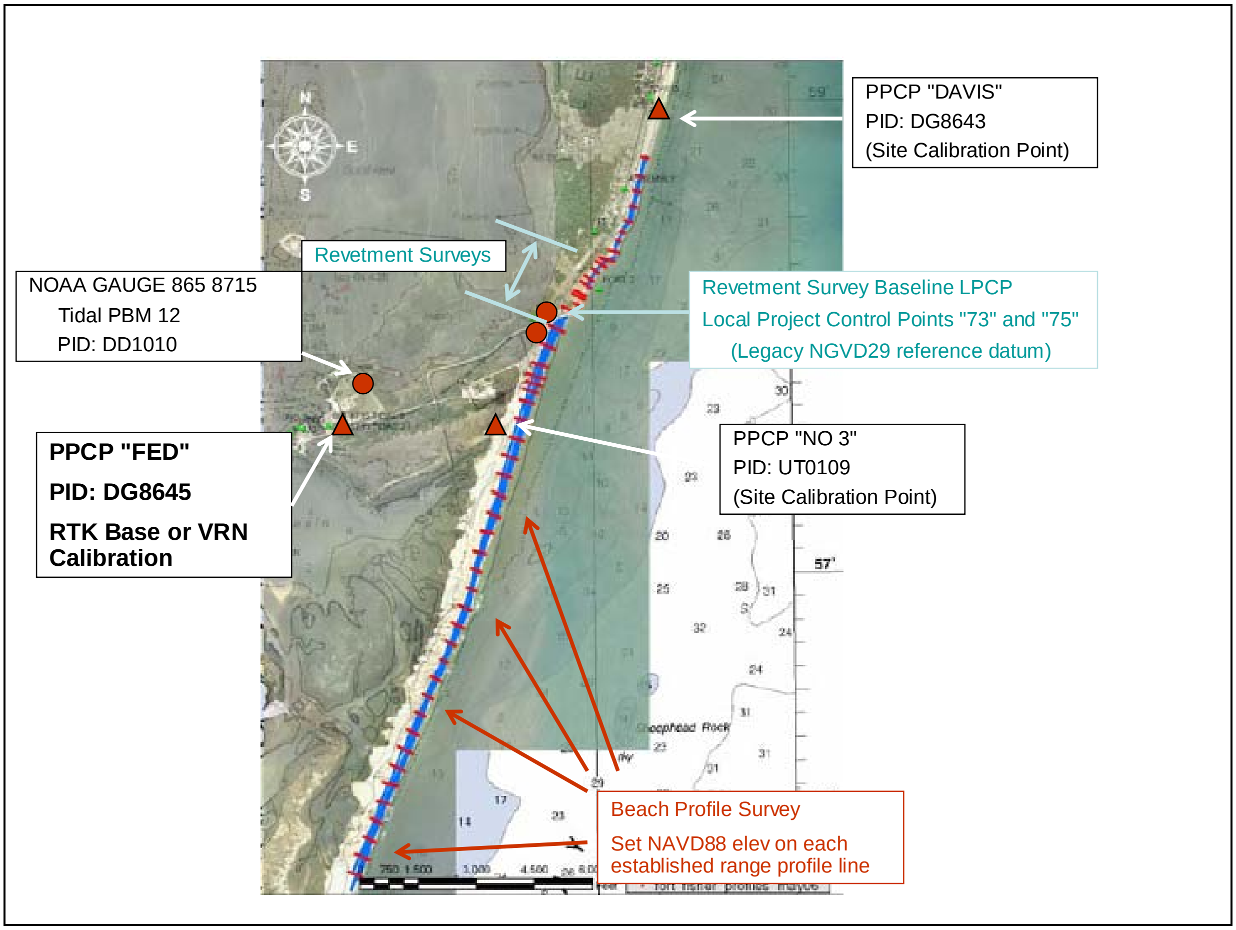


Figure G-1. Fort Fisher revetment and beach monitoring survey scheme.

G-2. Connections to NSRS and Tidal Datum References. The PPCP established for the project is PBM "FED," as shown on Figure G-1. This point is published in the NSRS and has observed NAD83/GRS80 ellipsoid height observations. Excerpts taken from the NOAA NGS published datasheet are shown in Figure G-2.

```
DG8645 ***********************************************************************
DG8645  DESIGNATION -  FED
DG8645  PID         -  DG8645
DG8645  STATE/COUNTY-  NC/NEW HANOVER
DG8645  USGS QUAD   -  KURE BEACH (1979)
DG8645
DG8645                        *CURRENT SURVEY CONTROL
DG8645  ___________________________________________________________________
DG8645* NAD 83(2007)-  33 57 37.24132(N)    077 56 28.77768(W)     ADJUSTED
DG8645* NAVD 88     -         5.992  (meters)      19.66   (feet)  ADJUSTED
DG8645  ___________________________________________________________________
DG8645  EPOCH DATE  -        2002.00
DG8645  X           -   1,106,339.885 (meters)                     COMP
DG8645  Y           -  -5,178,836.890 (meters)                     COMP
DG8645  Z           -   3,542,781.497 (meters)                     COMP
DG8645  LAPLACE CORR-          -4.76  (seconds)                    DEFLEC99
DG8645  ELLIP HEIGHT-         -31.476 (meters)          (02/10/07) ADJUSTED
DG8645  GEOID HEIGHT-         -37.37  (meters)                     GEOID03
DG8645  DYNAMIC HT  -           5.986 (meters)      19.64  (feet)  COMP
DG8645
DG8645  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
DG8645  Type    PID    Designation                      North   East  Ellip
DG8645  -------------------------------------------------------------------
DG8645  NETWORK DG8645 FED                                0.57   0.51   1.53
DG8645  -------------------------------------------------------------------
DG8645  MODELED GRAV-     979,620.0   (mgal)                       NAVD 88
DG8645
DG8645  VERT ORDER  -  SECOND    CLASS II
```

Figure G-2. Portion of NGS datasheet for bench mark FED.

The 19.66 ft NAVD88 elevation of FED is based on adjusted leveling observations. The estimated 95% confidence of the ellipsoid height is less than 2 cm. Thus, this is an excellent point for use as an RTK base station from which all supplemental surveys can be referenced.

a. PPCP verification. The coordinates of PPCP "FED" were site calibrated against three nearby published bench marks in the NSRS—DAVIS, NO 3, and 865 8715 TIDAL 12. (DAVIS for horizontal only). These site calibration checks were based on RTK observations with the base station at FED. (Subsequent surveys used the North Carolina RTN rather than a RTK base at FED). Figure G-3 shows a typical field site calibration at PBM "NO 3." The 0.1 ft horizontal difference and 0.01 ft elevation difference are within acceptable measurement tolerances.

geodynamics
COMPLEX COASTAL CHANGE MADE CLEAR

RTK-GPS Pre-Survey Site Calibration

General

Date	5/3/2006
Project	USACE Fort Fisher Beach Profiles
Surveyor(s)	Freeman / Bernstein
Equipment	Trimble 5700 Basestation, Trimmark III 25 watt RTK Radio, Maxrad 5dB gain Antenna, Zepher Geodetic base antenna, Trimble 5700 RTK rover, Zepher antenna
Weather	Sunny, Few Clouds, 83 F, WNW Wind 15-20 kts.
Units	Meters
Notes	Tidal No 2- destroyed, FF 1-not found, Assembly - not found, FF3 found bubt not good for GPS Observations at the time
Coordinate System	NC State Plane, NAD83 (horiz), NAVD88 (vert)

Basestation Information

Designation	FED
PID	DG8645
Agency	US Coast Guard
Horiz Order	1
Vert Order	2
N	23857.459
E	707461.906
Z	5.992

Datasheet WWW Link

Benchmark Checks

Designation	NO 3
PID	UT0109
Agency	US BM 1911
Horiz Order	2
Vert Order	2

	Recorded	Published	Difference
N	23869.183	23869.135	-0.048
E	707419.36	707419.549	0.189
Z	1.793	1.804	0.011

Notes

Figure G-3. Site calibration RTK observations at PBMs "FED" and "NO 3."
(Checks to other site calibration points were made but are not shown in this figure)

b. Local PBM and TBM control. Figure G-4 lists the local reference PBMs and TBMs that were connected to the PPCP for the project using RTK positioning methods and differential leveling. The legacy NGVD29 elevation was retained for the two reference points used for the stone revetment monitoring survey area (USACE "73" and USACE "75"). The beach monitoring TBMs for each profile range were positioned using total station and RTK methods, and set relative to previously established monitoring range locations. Differential levels were also run through these points and the leveled elevations were held over the RTK elevations. All elevation measurements were relative to PPCP "FED" on the NAVD88 reference datum.

NGS Mon "FED", Stamped "FED 1993"
NAD '83 (2007)
N: 78, 271.26'
E: 2,321,065.58'
Elevation: 19.66' (adjusted) NAVD"88

NGS data sheet attached, below

Rock Revetment Survey based on (NGVD'29):

Point	Baseline Station	Northing	Easting	Elevation	Description
73	568+91.19, -225	81,832.0151	2,328,078.783	15.16'	USACOE Monument
75	566+38.80, -200	82,051.6633	2,328,251.723	21.40'	USACOE Monument

Beach Profile Temporary Survey Control, NAVD'88

Point	Northing	Easting	Elevation	Description
1005	83,131.9915'	2,329,171.5304'	8.93'	18" WOOD STAKE
1006	83,637.4330'	2,329,405.8200'	12.82'	18" WOOD STAKE
1007	84,023.9970'	2,329,509.8780'	13.14'	18" WOOD STAKE
1008	84,417.9320'	2,329,587.6850'	14.33'	18" WOOD STAKE
1009	84,807.9160'	2,329,677.6160'	14.91'	18" WOOD STAKE
1010	85,292.8280'	2,329,800.0250'	15.17'	18" WOOD STAKE
1011	85,378.4740'	2,329,879.1460'	15.36'	18" WOOD STAKE
1014	81,278.5780'	2,328,047.0970'	9.50'	18" WOOD STAKE
1019	79,480.3100'	2,326,830.2700'	9.04'	18" WOOD STAKE
1023	82,644.5270'	2,328,815.5440'	10.86'	18" WOOD STAKE
1024	82,014.4670'	2,328,323.9140'	22.59'	40D NAIL
1025	80,118.1730'	2,327,103.5670'	19.01'	18" WOOD STAKE
4010	82,016.5763'	2,328,322.6362'	23.52'	18" WOOD STAKE
4011	81,297.8452'	2,328,013.0360'	9.60'	18" WOOD STAKE
4105	76,622.5126'	2,326,039.5377'	16.82'	18" WOOD STAKE
4109	77,659.4700'	2,326,346.0500'	12.45'	18" WOOD STAKE
4121	75,134.2843'	2,325,579.0767'	13.44'	18" WOOD STAKE
4125	74,195.2775'	2,325,347.2231'	11.84'	18" WOOD STAKE
4130	73,268.5720'	2,324,986.0883'	15.12'	18" WOOD STAKE
4134	72,341.6740'	2,324,640.1954'	13.26'	18" WOOD STAKE
4143	70,966.8913'	2,324,028.6202'	10.84'	18" WOOD STAKE
4148	70,059.4353'	2,323,607.9422'	12.62'	18" WOOD STAKE
4442	70,805.7718'	2,323,964.7684'	7.52'	18" WOOD STAKE
4450	78,271.1098'	2,321,065.5552'	19.44'	18" WOOD STAKE
4459	70,805.6693'	2,323,964.8716'	7.47'	18" WOOD STAKE
4616	70,805.7390'	2,323,964.8300'	7.37'	18" WOOD STAKE
8097	82,729.9630'	2,329,006.9098'	10.44'	18" WOOD STAKE
8331	79,534.2030'	2,326,908.7685'	14.60'	18" WOOD STAKE
25018	83,377.3410'	2,329,272.9640'	8.47'	18" WOOD STAKE

Figure G-4. Primary, local, and baseline range control for Fort Fisher project.

c. Conversion from NAVD88 to NGVD29. The survey specifications required that range profile data be referenced to NGVD29 in order to compare prior surveys on that legacy datum. All topographic and hydrographic surveys of each profile range observed on NAVD88 were converted to NGVD29 based on an average VERTCON difference of (-) 0.956 ft (NAVD88 - NGVD29). This conversion assumed prior surveys referenced to NGVD29 were based on published NSRS connections. (It would have been preferable to establish NAVD88 on an existing mark with a published agency elevation to derive the conversion value.)

d. Tidal reference datum. The tidal relationship to NAVD88 was estimated using the nearest NOAA gage to the project site. Figure G-5 shows a number of NOAA gages in the region; however, only the Wilmington Beach gage on the coast is most representative of the project site to the south. The Southport gage to the south has similar mean and diurnal tide ranges as the Wilmington Beach gage. Therefore, the interpolated tidal characteristics at the project site are likely representative of these gages.

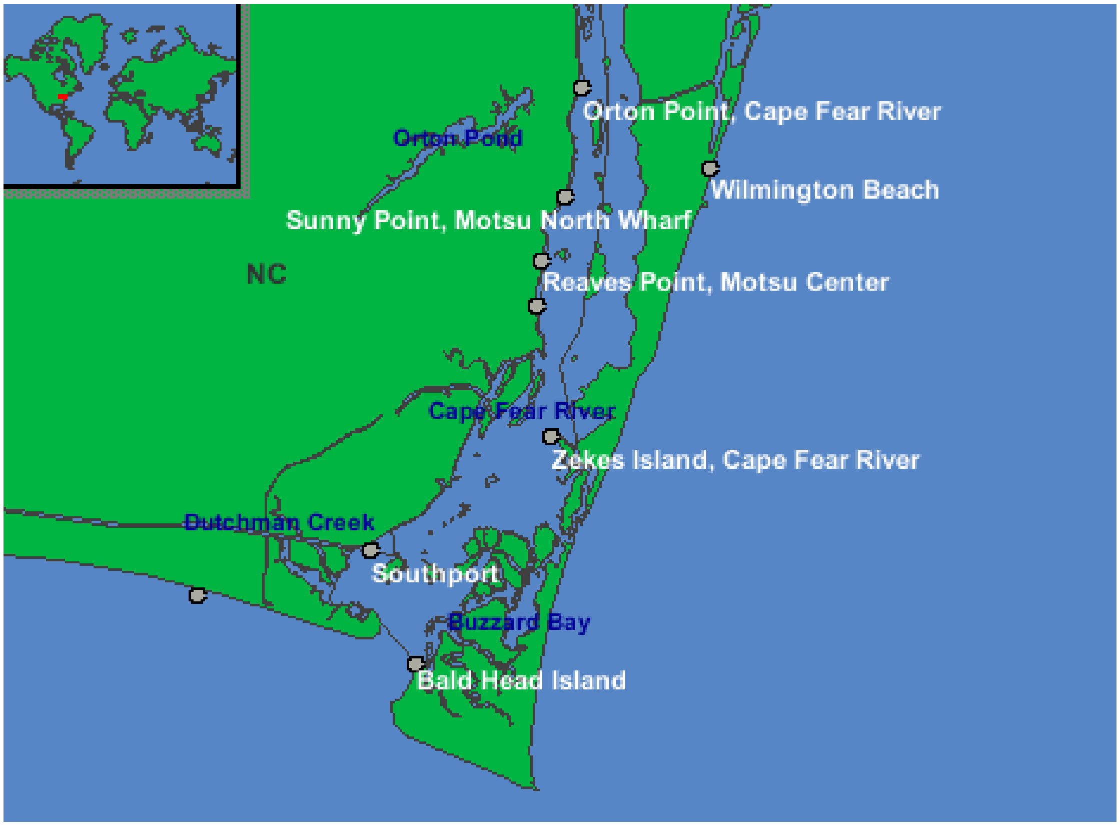


Figure G-5. Published NOAA/CO-OPS gage data in vicinity of Fort Fisher, NC.

(1) The NOAA CO-OPS station tabulation for this gage is shown in Figure G-6.

```
Tidal datums at WILMINGTON BEACH [865 8559] based on:

     LENGTH OF SERIES:      4 MONTHS
     TIME PERIOD:           March 1977 - June 1977
     TIDAL EPOCH:           1983-2001
     CONTROL TIDE STATION:  8658120 WILMINGTON, CAPE FEAR RIVER


Elevations of tidal datums referred to Mean Lower Low Water (MLLW), in METERS:

     MEAN HIGHER HIGH WATER (MHHW)                 =  1.433   [4.70 ft]
     MEAN HIGH WATER (MHW)                         =  1.329
     NORTH AMERICAN VERTICAL DATUM-1988 (NAVD)     =  0.902   [2.96 ft]
     MEAN TIDE LEVEL (MTL)                         =  0.688
     MEAN SEA LEVEL (MSL)                          =  0.686   [2.25 ft]
     MEAN LOW WATER (MLW)                          =  0.047
     MEAN LOWER LOW WATER (MLLW)                   =  0.000
```

Figure G-6. NOAA tide gage Wilmington Beach.

(2) Based on the data in Figure G-6, NAVD88 is 0.71 ft above MSL (0.902 m – 0.686 m). The difference between NGVD29 and MSL is then 0.25 ft (0.96 ft – 0.71 ft). The difference between NAVD88 and MSL is assumed the same at the project site to the south. (A VDatum model may show slight variations).

(3) The gage at NOAA historical tide station 865 8715 is no longer published by NOAA CO-OPS. This gage would not be relevant to this project given its interior location.

(4) Based on the above data, the geodetic and tidal relationships at PPCP "FED" could be tabulated as shown in Table G-1.

Table G-1. Elevations at PPCP "FED."

Datum [1]	Elevation	Referenced From	Estimated Uncertainty	Relative to
MLLW	22.62 ft	NOAA gage 865 8559	±0.2 ft	NWLON
NGVD29	20.62 ft	VERTCON transform	±0.3 ft	NSRS
MSL	20.37 ft	NOAA gage 865 8559	±0.2 ft	NWLON
NAVD88	19.66 ft	NSRS	±0.1 ft [2]	NSRS
MHW	17.92 ft	NOAA gage 865 8559	±0.2 ft	NWLON
Ellipsoid	-103.27 ft	GPS observations	±0.05 ft	NAD83/GRS80

[1] Tidal datum elevations are estimated based on NOAA gage 865 8559 (Wilmington Beach).

[2] Uncertainty relative to NSRS not factored in to supplemental surveys.

APPENDIX H

East Branch Clarion River Dam and Spillway Control Surveys (Pittsburgh District)

H-1. Introduction. This appendix is an example of establishing NSRS control on a Pittsburgh District dam and reservoir on the East Branch of the Clarion River (Figure H-1). A single primary PBM was connected by a combination of GPS and CORS observations from surrounding NSRS bench marks. Secondary deformation monitoring points were controlled from the primary PBM and dam and spillway profile surveys were run. Elevation data sets developed from LIDAR collected in 2006 was then used for comparative analysis against the profile surveys.

a. This appendix was compiled from survey reports by two Pittsburgh District Contractors: TerraSurv, Inc. and Photo Science, Inc. TerraSurv performed the initial NSRS network connections in May 2008 and Photo Science performed comparative data mapping analysis in 2009.

Figure H-1. East Branch Dam and Reservoir (Elk County, PA).

b. The Pittsburgh District issued task orders to TerraSurv and Photo Science to establish a positional relationship/correlation between the hydraulic, geodetic, and engineering design datum at the East Branch Clarion River Dam and Spillway located in Elk County, PA. This work supports the District's adherence to ER 1110-2-8160 (*Policies for Referencing Project Elevation Grades to Nationwide Vertical Datums*), namely to address the "… need to firmly establish the relationships between hydraulic and geodetic datums…"

H-2. Project Location. Authorized by the Flood Control Act of 1944, East Branch Clarion River Lake is one of 16 projects in the USACE, Pittsburgh District. An important link in a system of flood risk management projects, East Branch provides flood protection for the Clarion River Valley as well as the lower Allegheny and upper Ohio Rivers. Completed in 1952, East Branch Lake has the capability to store the equivalent run-off of 21.84 inches of precipitation from its 72.4 square mile drainage area.

H-3. Scope of Work. The initial requirements outlined for the project were as follows:

a. Establish a primary NSRS bench mark and two secondary bench marks at each project location. This is intended to conform to the criteria in ER 1110-2-8160 that "...the designed, constructed, and maintained elevation grades of projects shall be reliably and accurately referenced to a consistent nationwide framework, or vertical datum—i.e., NSRS..."

b. Determine validity of dam/spillway design grade to current as-built surveys and other sources (LIDAR mapping).

c. Establish relationship/correlation between hydraulic and geodetic datums at each project.

H-4. CEPD Assessment.

a. As part of the 2007 Corps-wide CEPD review, the following actions were taken to accomplish the above objectives.

(1) Perform reconnaissance surveys at the dam site to verify existing local control.

(2) Develop recommendations for Corrective Actions.

(3) Establish NSRS Project Control, e.g., Primary bench mark and two secondary PBMs.

(4) Survey gage reference points.

(a) Pool.

(b) Outflow.

(5) Establish pool elevation relative to NAVD88.

(6) Profile dam and spillway.

(7) Reference deformation monitoring points to NAVD88 and PPCP.

(8) Perform alignment measurements.

b. A CEPD research of existing control data at the East Branch Dam site indicated that geodetic control was referenced to legacy datums of dated origins. The CEPD review is summarized below.

"East Branch Dam horizontal positions are controlled by traverses tied to USGS Stations TT3K, TT6K, and TT7K, datum uncertain, and are computed on Pa. North-Zone System of Coordinates. Elevations are based on those same USGS Stations. Topography was compiled by plane table in 1946 and traced on Map Sheet 038b-U1-16/1 through 10. Additional topography was compiled from aerial photographs exposed November 1979 and consists of Map Sheets 038b-U1-101/1 thru 4, scale 1:2,400, control based on N.A.D. 1927 and N.G.V.D. 1929."

H-5. Options Considered for Corrective Action Field Surveys.

a. The following methods were considered for connecting the Primary Project Control Point (PPCP) to the NSRS.

(1) Differential Leveling (Orthometric Height Accuracies ~ 0.5-2 cm).

(2) GPS Network-"Blue Booking" (Orthometric Height Accuracies ~ 2-3 cm).

(3) OPUS DB-Using CORS Network (Orthometric Height Accuracies ~ 5-10 cm).

b. Differential leveling options.

(1) Labor Intensive.

(2) High cost.

(3) Projects are in isolated areas, some are quite distant from existing level lines.

(4) Horizontal positions not determined.

(5) Determined to be not economically feasible.

c. Blue Book option.

(1) Create a GPS network, format and submit to NGS.

(2) Advantages:

(a) Ties to adjacent points and bench marks.

(b) Multiple occupations.

(c) Homogenous network.

(3) Disadvantages:

(a) More complex to implement.

(b) Requires multiple receivers and planned GPS campaign.

(c) Costly data processing.

d. OPUS-DB option.

(1) Advantage: Relatively simple to implement (Single GPS receiver).

(2) Disadvantages:

(a) No ties to bench marks or adjacent points.

(b) Single occupation.

(c) Requires minimum of two 4-hour occupations.

e. The Blue Book method was selected for the following reasons.

(1) Provides high quality control at each project, as requested by Project Managers.

(2) Utilizes ties to bench marks/HARN/CORS.

(3) Takes advantage of GPS data collected at each site 2005-2008 (i.e. less than 4 hour sessions, not acceptable to OPUS-DB).

H-6. Recommended Primary Control Bench Mark at Project Site. It was recommended that USACE mark "1-500" be used as the primary project bench mark, and M1 (right bank, on dam axis) and M2 (left bank, dam axis) be used as secondary bench marks. Yearly ties are made between these three marks during the alignment survey. Levels to the water gage reference marks could be run from any of the aforementioned marks. A precise level tie has been recently run to PBM 1-500 from an NSRS mark. This data could be used to submit a vertical blue book project to the NSRS. This will require that the raw data file be retrieved, and that a differential level tie be made from Z 337 to another mark on the same line (two mark tie). It is believed that an adjacent mark within a reasonable distance should not be hard to find. The initial evaluation assumed that PBM 1-500 would be obstructed and not suitable for GPS. Two options for GPS derived elevations were considered in the initial evaluation. One option was to simultaneously occupy a secondary project bench mark, M2, and two nearby NSRS bench marks. The primary PBM 1-500 is partially obstructed and not suitable for GPS. Levels are run yearly between M1, M2, and 1-500 so there would be sufficient data available to provide an accurate tie to PBM 1-

500. Alternatively, 1-500 or M2 could be occupied for two sessions of at least 240 minutes (4 hours) and submitted to OPUS-DB.

H-7. Primary NSRS Control Network. Existing USACE survey disk (PBM) "1-500" is the designated primary control point for this project. This disk is located atop the upstream parapet wall at the land abutment for the concrete bridge leading to the intake tower on the right bank of the reservoir. This abutment is not rigidly connected to the bridge; rather the bridge sits on the abutment seat. This mark is shown in Figure H-2.

Figure H-2. Primary Project Control Point 1-500 on bridge leading to intake tower.

a. Several options were available for bringing control in from the National Spatial Reference System (NSRS) to the project. A search was made of the NSRS database. There is a level line running north-south along a railroad located approximately 1.6 air miles west of the dam. Bench mark Z 337 (MA0592), located on a bridge abutment, was recovered on this line, and determined to be suitable for GPS observations. Research revealed that a survey crew from the Corps of Engineers had run a line of differential levels from this bench mark to PBM 1-500 and back in 2003 using a Zeiss DiNi 12 digital level and 2 m invar rods. However, only field notes were found, the raw data (which could be Blue Booked) was not found. A search was then made of the NSRS for bench marks located on stable structures that also have HARN horizontal positions published on NAD83 (NSRS2007). This search returned two marks listed Table H-1.

Table H-1. HARN Bench Marks.

Name of Mark	PID	Horizontal Accuracy	Vertical Order	Location	Setting
TTS 64 K	MA0735	0.3 cm	II-Class 0	17.9 mi west	Bedrock
V 25	MA0095	0.3 cm	II-class 0	19.9 mi east	Bridge abutment

b. All three NSRS marks are located on stable structures, and are shown on the map in Figure H-3. The data from these three marks to the on site primary control point 1-500 was formatted for Blue Book submittal to the NSRS. This resulted in the inclusion of PBM 1-500 in the NSRS database. The horizontal datum is NAD83 (CORS1996), obtained via an OPUS solution. The vertical datum is the NAVD88, obtained via direct GPS ties to the three bench marks described above. The three marks located at the dam, 1-500, M1, and M2, were connected via differential levels, static GPS, and EDM/angle measurements. This data was not Blue Booked.

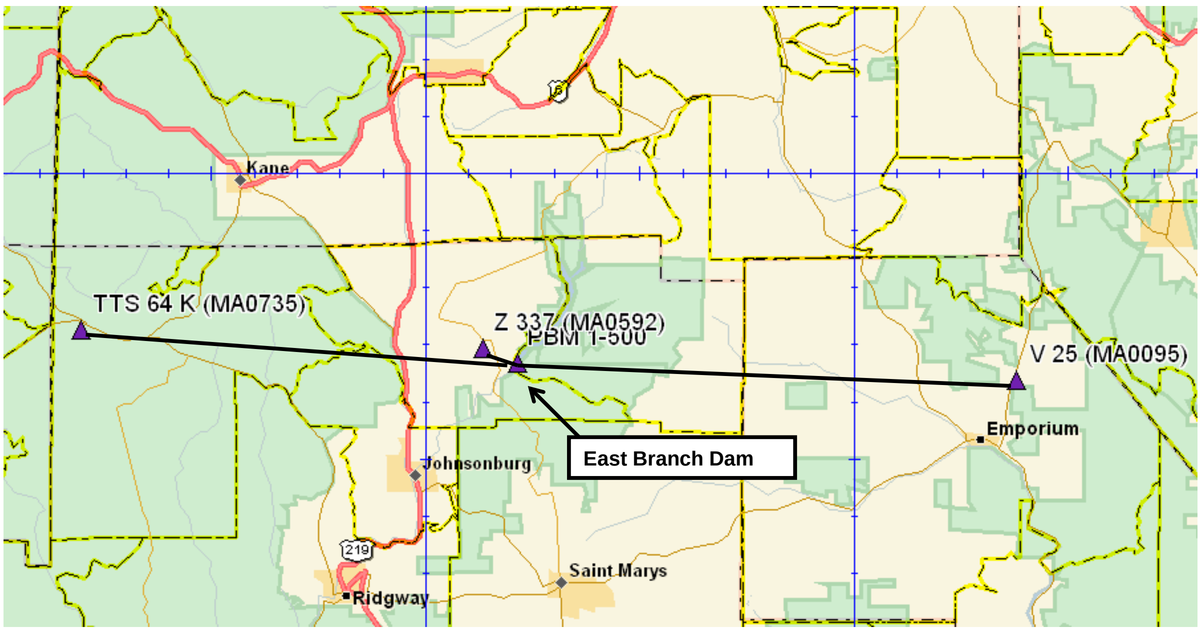


Figure H-3. NSRS control scheme for establishing elevation on PBM 1-500.

H-8. <u>GPS Survey Procedures to Connect PBM 1-500</u>. Three Trimble dual frequency receivers (a 5700 and two R8 GNSS receivers) were used on days 133 and 134 of 2008. Fixed height tripods were utilized for the occupations of the NSRS bench marks. A standard survey tripod with tribrach was used at PBM 1-500 due to the difficulty of using a fixed height tripod at that location. Each point had two independent occupations. The Trimble 5700 receiver with a Zephyr antenna was setup on both days on PBM 1-500, and collected data during the entire day. The two HARN/ bench marks (V 25 and TTS 64 K) were each occupied twice, once on day 133 and once on day 134, each time with a different operator/tripod/receiver. The nearby bench mark, Z 337, was occupied twice on day 134, at different times of the day. The data collected at PBM 1-500 (GPS# 08042A) was submitted to the Online Positioning User Service (OPUS). The OPUS online processor selected three nearby CORS and determined the position of the submitted point. The results are shown in Table H-2.

Table H-2. OPUS Solutions.

Day	Duration Minutes	CORS USED	Overall RMS	% OBSERVATIONS USED	% Ambiguity Fixed
133	337	UPTC NYSM NYFS	0.014 m	93%	97%
134	752	UPTC NYSM NYFS	0.015 m	92%	98%

a. The average of the OPUS derived positions was used as the horizontal position and ellipsoidal height of PBM 1-500. The data was downloaded to a PC and processed using the Weighted Ambiguity and Vector Estimator (WAVE) processor in Trimble Geomatics Office, V1.63. The single baseline method was used, with the precise (IGS Rapid) ephemeris. All of the baselines were integer bias fixed solutions. Table H-3 shows the results of the baseline processing:

Table H-3. WAVE Baseline Results.

From	To	UTC Start	Duration Minutes	Length Meters	Ratio	Variance	RMS
MA0095	08042A	5/12/08 19:33	45	32378	21.60	1.5	0.012
MA0095	MA0735	5/12/08 19:33	43	60774	26.48	1.1	0.010
MA0735	08042A	5/12/08 19:24	52	28411	20.69	1.4	0.012
MA0095	08042A	5/13/08 11:30	46	32378	15.82	1.7	0.016
MA0095	MA0735	5/13/08 11:42	34	60774	10.74	0.8	0.013
08042A	MA0592	5/13/08 21:47	30	2435	36.41	12.1	0.011
08042A	MA0592	5/13/08 13:01	30	2435	30.94	2.9	0.005
MA0735	08042A	5/13/08 11:42	46	28411	14.20	1.7	0.015

b. Each of the baselines was measured twice in independent sessions. The processed vector components were transformed to a local horizon system (north, east, & up) for analysis—see Table H-4.

Table H-4. Baseline Residuals (in meters).

From	To	Delta N	Delta E	Resultant	Delta U	Length
08042A	MA0095	-0.004	-0.003	0.005	-0.001	32379
MA0095	MA0735	0.001	0.008	0.008	0.009	60774
08042A	MA0735	0.001	0.003	0.003	0.013	28412
08042A	MA0592	-0.005	-0.006	0.008	-0.007	2435

H-9. Least Squares Adjustments. The GPS data was adjusted using ADJUST, a least squares adjustment program from the NGS. The processed baselines were parsed to form an input file in the G-file format. The results from the two OPUS solutions were also included. No scaling of the a priori baseline statistics was done. Station errors (HI and centering) of 0.005 m were also included. Geoid separations for each station were interpolated using the GEOID03 model. The first adjustment constrained the CORS UPTC ARP to the published NAD83 (epoch 2002.0) position (latitude, longitude, and ellipsoidal height). The standard deviation of unit weight was

2.42. This value was then used to scale the G file using the "modgee" program. The subsequent adjustment, utilizing the scaled G file, had a standard deviation of unit weight of 1.004. The misclosures at the three NSRS stations and the other two CORS used are shown in Table H-5.

Table H-5. Station Misclosures.

Station	Azimuth	Distance	Δ Ortho H	Δ Ellip H
MA0592 (Z 337)			+0.005 m	
MA0095 (V 25)	217°	0.002 m	-0.004 m	-0.001 m
MA0735 (TTS 64 K)	243°	0.008 m	-0.022 m	-0.008 m
NYSM ARP	285°	0.025 m		+0.006 m
NYFS ARP	280°	0.025 m		-0.002 m

a. The straight line distance between the two HARN bench marks is 60.8 km (37.8 miles), but the distance through the leveling network is about 105 km. Benchmarks Bench marks with that separation could be expected to have a relative accuracy in orthometric height of about 0.01 m between them. The final adjustment constrained PBM 1-500 horizontally and the three existing NSRS bench marks vertically (NAVD88 orthometric height). The estimated variance factor was 1.11. The vertical confidence region at the 95% level for PBM 1-500 from this adjustment was 0.007 m. This, combined with the estimated accuracy of the geoid model, gives an estimated accuracy of the GPS derived orthometric height at PBM 1-500 of ± 0.03 m. An additional check is given by comparing the NAVD88 orthometric height determined in this project to the NAVD88 height determined in 2003 by precise differential levels from **Z 337**, with a difference of 0.004 m.

b. The next adjustment constrained UPTC ARP horizontally and the nearest benchmark to the project, Z 337, vertically (NAVD88 orthometric height). The misclosures in orthometric height at the two HARN/benchmarks were then computed: -0.010 m at V 25 and -0.027 m at TTS 64 K. These misclosures were within the expected range, so the subsequent orthometric height adjustment constrained the three NSRS benchmarks to their published NAVD88 heights, along with the horizontal position of UPTC ARP. The standard deviation of unit weight was 1.34. This adjustment provided the adjusted NAVD88 orthometric height for the new station, EAST BRANCH.

c. The final adjustment constrained the three CORS and the two HARN stations in all three dimensions (latitude, longitude, and ellipsoidal height. The standard deviation of unit weight was 4.42. This adjustment provided the adjusted latitude, longitude, and ellipsoidal height for the new station, EAST BRANCH, as well as the NSRS benchmark Z 337.

H-10. Supplemental Deformation Surveys. A combination of GPS and conventional methods was used in the deformation survey of the six alignment pins nominally online between M1 and M2. A base receiver (Trimble 5700) was running on primary control monument PBM 1-500. Two Trimble R8 GNSS receivers were used to occupy M2, A5, A4, A3, A2, A1, and M1, in order (see Figure H-4). Each occupation had at least 15 minutes common occupation time with adjacent stations. The alignment pins (A1 thru A5) were occupied using a standard tribrach with a precise rotatable optical plummet. Height of antenna measurements were taken as slope measurements to the blue band on the antenna housing, and then corrected to the Antenna

Reference Point (ARP). M1 and M2 were occupied by placing a standard optical plummet directly atop the pedestal. The height of antenna measurements for the pedestals were converted to be the ARP height above the top of the 5/8" bolt. A Trimble S6 high accuracy total station was set up on pedestal M4 located upstream of the dam on the left bank, and angle and distance measurements were taken to each station during the GPS occupations. The distance measurements were corrected for atmospheric conditions and reduced to the mark-to-mark components. The GPS and conventional data were combined in a least squares adjustment to obtain adjusted coordinates for M1, M2, and alignment pins A1 through A5. These coordinates will be directly used in future surveys to monitor the movement of the alignment pins. Offsets of A1 through A5 from the M1↦M2 line were also computed to maintain backwards compatibility with previous alignment surveys.

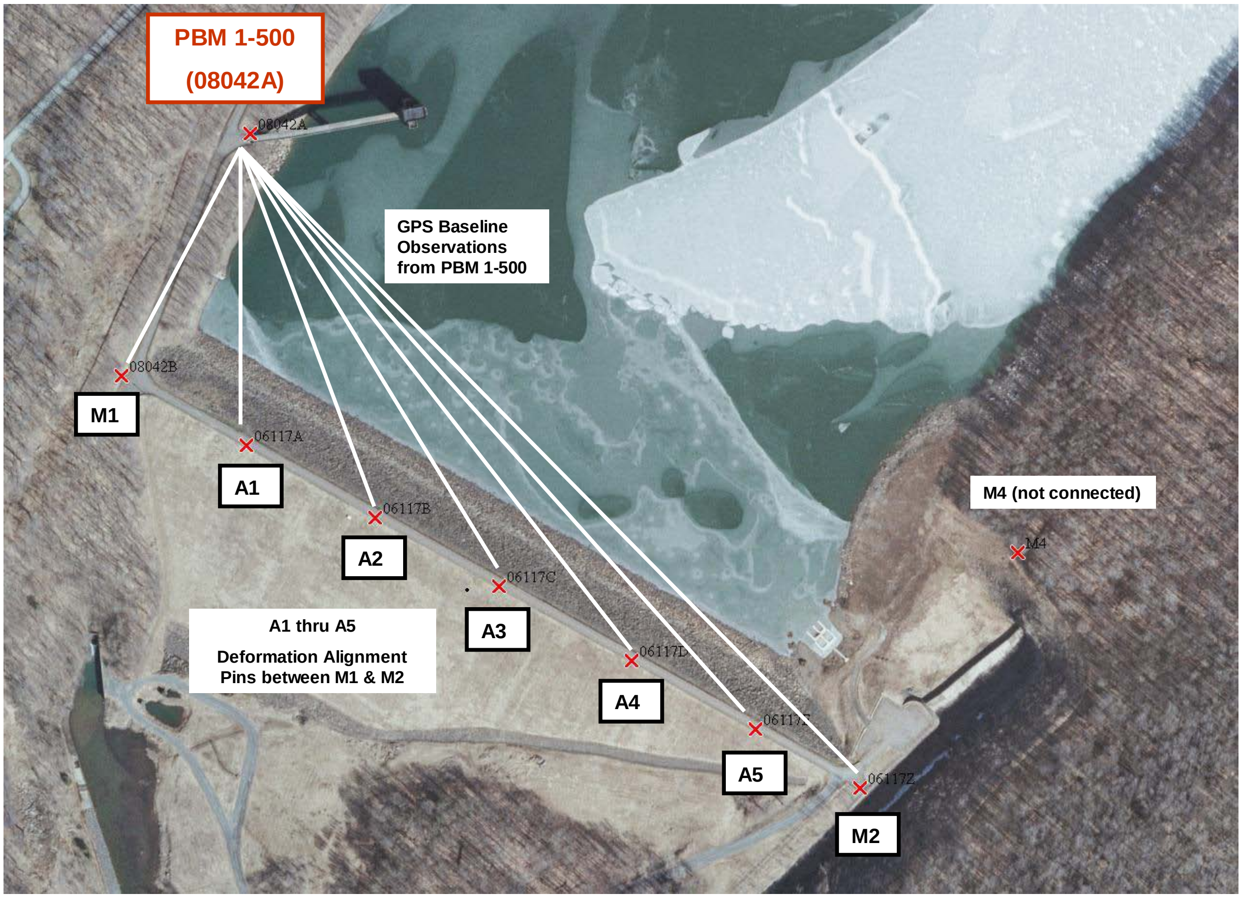


Figure H-4. Local deformation alignment points.

a. Figure H-5 shows the offsets from the line between M1 and M2 over the last several years:

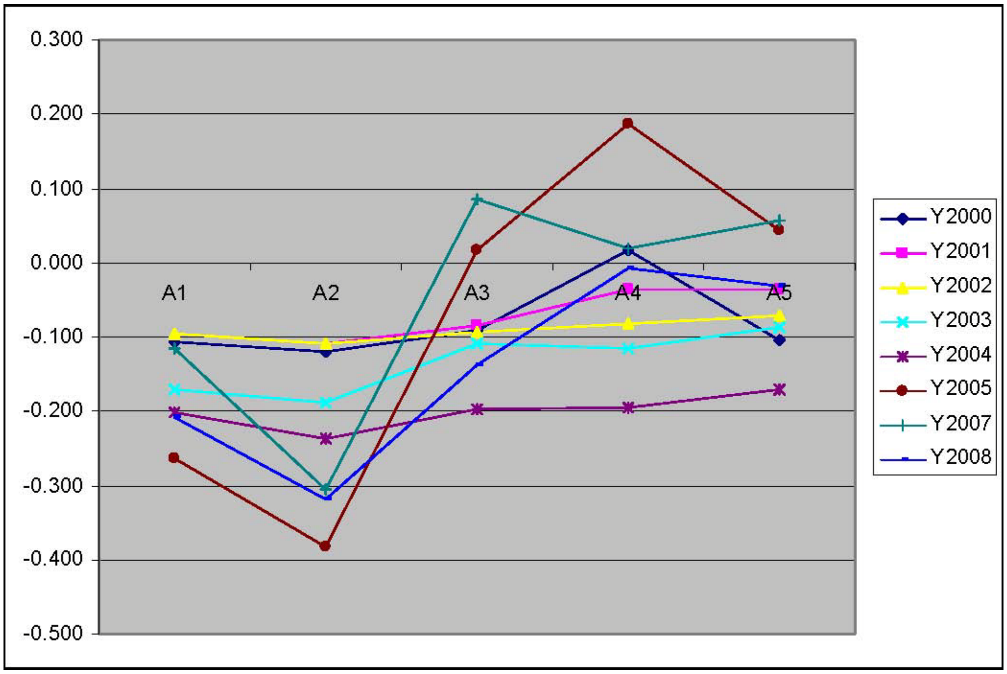


Figure H-5. Alignment point offsets from 2000 to 2008.

b. The settlement survey was executed using a DiNi 12 digital level and bar coded rod. A run was made from PBM 1-500 through each of the pedestals and alignment pins, and back to PBM 1-500 with a loop closure of 0.002 m over a distance of 1.5 km. A spur line was run from A5 down to the outflow area, and continued to the outflow gage located downstream of the dam. A loop was also run from PBM 1-500 to the gage located in the intake tower.

Table H-6. East Branch Clarion River Lake Dam Adjusted Coordinates. PA North Zone State Plane Coordinates – NAD 1983. (NSRS 2007)

Station Name	GPSID	Northing Meters	Easting Meters	NAVD88 Meters	Northing US FT	Easting US FT	NAVD88 US FT	Convergence	Scale Factor	Elevation Factor	Combine Factor
BM 1-500	08042A	155221.009	529463.570	521.442	509254.259	1737081.728	1710.764	0°33'33.91"	0.99995998	0.99992305	0.99988304
V 25	MA0095	153851.210	561809.546	338.217	504760.177	1843203.487	1109.634	0°18'10.21"	0.99995956	0.99995185	0.99991140
Z 337	MA0592	156229.738	527247.172	526.900	512563.732	1729810.096	1728.671	0°34'37.47"	0.99996038	0.99992220	0.99988258
TTS 64 K	MA0735	157712.792	501164.981	573.625	517429.386	1644238.776	1881.968	0°47'02.76"	0.99996089	0.99991491	0.99987580
A1	06117A	155026.225	529461.356	520.157	508615.208	1737074.465	1706.548	0°33'33.92"	0.99995991	0.99992325	0.99988316
A2	06117B	154981.286	529541.580	520.182	508467.770	1737337.666	1706.630	0°33'31.61"	0.99995989	0.99992326	0.99988315
A3	06117C	154938.210	529618.493	520.187	508326.444	1737590.005	1706.647	0°33'29.41"	0.99995987	0.99992325	0.99988312
A4	06117D	154891.929	529701.125	520.217	508174.603	1737861.108	1706.745	0°33'27.03"	0.99995986	0.99992325	0.99988311
A5	06117E	154848.789	529778.141	520.313	508033.069	1738113.784	1707.060	0°33'24.82"	0.99995984	0.99992323	0.99988307
M2	06117Z	154812.168	529843.522	521.536	507912.922	1738328.287	1711.073	0°33'22.95"	0.99995983	0.99992304	0.99988286
M1	08042B	155069.850	529383.483	521.504	508758.334	1736818.978	1710.968	0°33'36.15"	0.99995992	0.99992304	0.99988297
M4	M4	154959.204	529941.779	526.281	508395.322	1738650.654	1726.640	0°33'20.18"	0.99995988	0.99992228	0.99988217

c. Updated NAVD88 elevations at the project site as determined by differential levels from PBM 1-500 are shown in Table H-7.

Table H-7. Updated NAVD88 Elevations.

Station Name	NAVD88 meters	NAVD88 US FT	Description
1-500	521.442	1710.764	BM on intake bridge parapet wall
A1	520.157	1706.548	Alignment pin
A2	520.182	1706.630	Alignment pin
A3	520.187	1706.647	Alignment pin
A4	520.217	1706.745	Alignment pin
A5	520.313	1707.060	Alignment pin
BOLT	520.385	1707.296	Bolt on floor of intake tower near gage
FLOOR	520.378	1707.273	Floor elevation in intake tower near gage, +0.090 m up to sill
M1	521.504	1710.968	Pedestal (top of bolt)
M2	521.536	1711.073	Pedestal (top of bolt)
M3	521.312	1710.338	Pedestal (top of bolt)
TABLE	521.213	1710.013	Table surface in intake tower, +0.027 m up to knife edge
TBM1	485.854	1594.006	Anchor bolt
TBM2	466.543	1530.650	Square painted on NW corner of building, Weir 6
TBM3	467.505	1533.806	Square painted on south end of left bank training wall at outflow
TBM4	465.819	1528.275	Top of angle iron for weir gage at downstream end of spillway
TBM5	467.489	1533.753	Nail in triple black cherry, upstream of gage house, set by USGS
TBM6	467.331	1533.235	Nail in red maple, downstream of gage house, set by USGS
TBM7	465.774	1528.127	Bolt (lower of 2) protruding from downstream side of gage

d. A portion of the published NGS NSRS description for PBM 1-500 (i.e., EAST BRANCH PID = DK7088) is shown below:

```
**********************************************************************
DK7088  HT_MOD      -  This is a Height Modernization Survey Station.
DK7088  DESIGNATION -  EAST BRANCH
DK7088  PID         -  DK7088
DK7088  STATE/COUNTY-  PA/ELK
DK7088  USGS QUAD   -  GLEN HAZEL (1969)
DK7088                        *CURRENT SURVEY CONTROL
DK7088* NAD 83(2007)-  41 33 40.56863(N)    078 35 44.27202(W)     ADJUSTED
DK7088* NAVD 88     -       521.44    (meters)     1710.8     (feet)   GPS OBS
DK7088  EPOCH DATE  -        2002.00
DK7088  X           -     945,126.312 (meters)                      COMP
DK7088  Y           -  -4,685,460.249 (meters)                      COMP
DK7088  Z           -   4,209,591.236 (meters)                      COMP
DK7088  LAPLACE CORR-           1.76  (seconds)                     USDV2009
DK7088  ELLIP HEIGHT-         489.730 (meters)          (10/02/08) ADJUSTED
DK7088  GEOID HEIGHT-         -31.70  (meters)                      GEOID09
DK7088  HORZ ORDER  -  B
DK7088  ELLP ORDER  -  THIRD     CLASS II
DK7088.The horizontal coordinates were established by GPS observations
DK7088.and adjusted by the TERRA SURV in October 2008.
DK7088.The datum tag of NAD 83(2007) is equivalent to NAD 83(NSRS2007).
**********************************************************************
```

H-11. Comparative Analysis of LIDAR Mapping. A comparative analysis of existing mapping data was performed at the East Branch Dam site in 2009. This involved comparisons with design (as built) data, gage reference elevations, and established pool reference elevations. The comparative analysis performed by Photo Science involved the use of three independent data sources. The “hydraulic” data source used in the analysis consisted of 2006, high-resolution, PaMAP LIDAR elevation data obtained by Photo Science from the Pennsylvania State University, Institute of State and Regional Affairs, Center for Geospatial Information Services located in Middletown, PA. The “geodetic” data source used in the analysis consisted of spillway survey profiles in *.csv format established by Photo Science sub consultant, TerraSurv Inc., in May of 2008. Lastly, the USACE supplied, “East Branch Dam Plan Elevation and Section Drawing” in PDF format dated 30 September 1982 was utilized to compare both the hydraulic and geodetic data sources against the original dam and spillway design elevations. Using these three sources a comparative analysis of the survey profiles along the top of the dam and spillway structures was performed by measuring, comparing, and recording survey elevations along each profile to their respective engineering design elevation and the existing LIDAR surface elevation.

H-12. Data Source Projection/Datum. In order to perform the comparative analysis it was necessary to ensure that all data sources were in the same projection, datum, and units of measurement. The horizontal projection/datum established for the analysis was the Pennsylvania State Plane Coordinate System (PASPCS), North Zone, North American Datum 1983 (NAD83). The vertical datum established for the analysis was expressed in orthometric heights using North American Vertical Datum 1988 (NAVD88). Both horizontal and vertical units were expressed in US Survey Feet.

H-13. East Branch Dam Plan Elevation and Section Drawing. An Adobe PDF file of the East Branch Dam Plan Elevation and Section Drawing was provided by the Pittsburgh District (see Figure H-6). The drawing identifies a design elevation of 1,707.0 feet at the top of the dam structure and 1,685.0 feet at the top of the spillway structure. Although the vertical datum is not explicitly identified on this design drawing, the drawing predates by some 6 years the release of the NAVD88, and therefore an assumption was made for NGVD29 elevations. To support the analysis it was necessary to convert these design elevations to NAVD88. Using NGS BM Z339, which is in the immediate vicinity of the dam and spillway, USACE personnel reviewed the NGS data sheet for BM Z339 and computed a (-) 0.49 foot difference between NGVD29 to NAVD88. Photo Science then applied the (-) 0.49 foot reduction to the NGVD29 design elevations resulting in the computed NAVD88 elevations of 1,706.51 feet for the top of dam structure and 1,684.51 feet for the top of the spillway structure. These computed NAVD88 design elevations were then used in the comparative analysis.

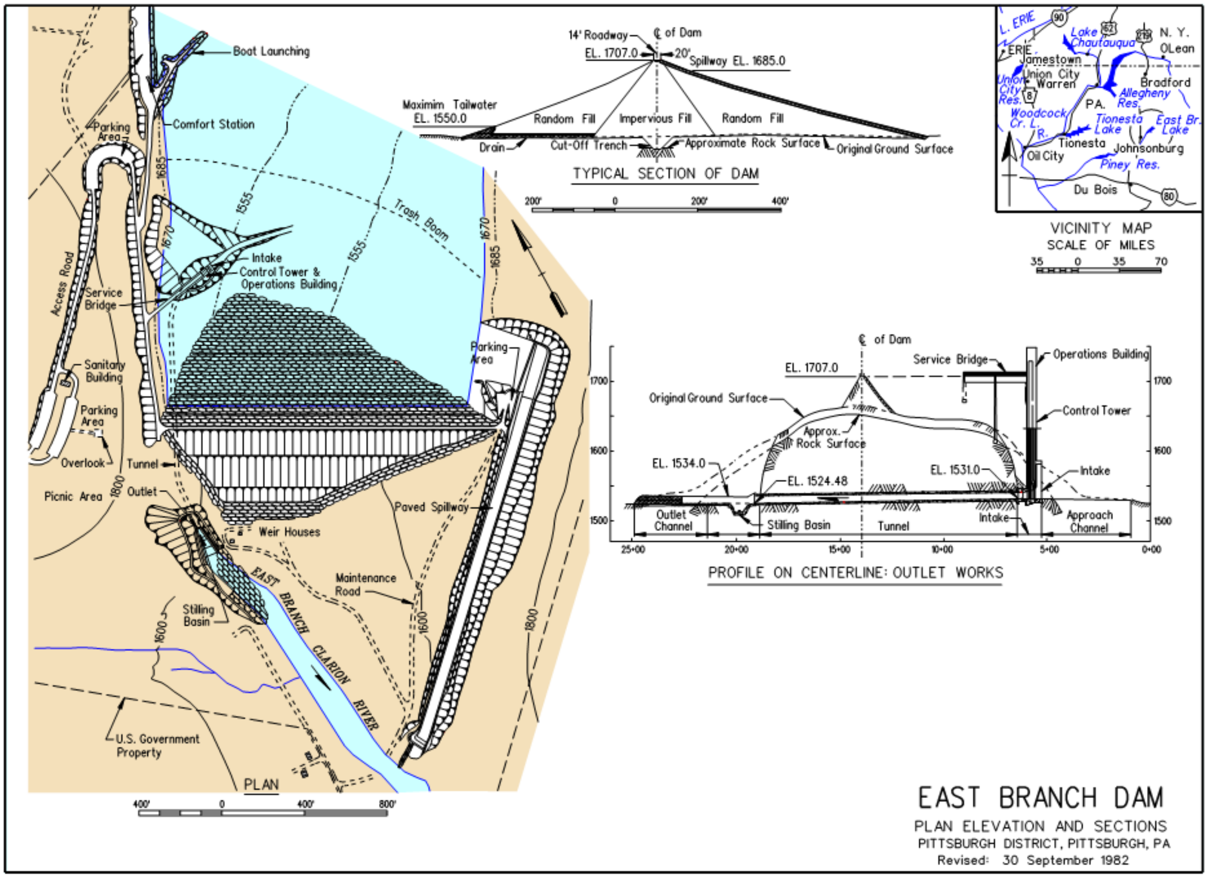


Figure H-6. East Branch Dam plan/elevation drawing.

H-14. Survey Profile Dataset. In May of 2008, Photo Science sub consultant, TerraSurv Inc., performed a series of field surveys at East Brach Clarion River Lake Dam to establish a primary control network, deformation monitoring and profiling. TerraSurv developed profiles along the dam and uncontrolled spillway. The profile data in *.csv format of the dam and spillway obtained under this task order was supplied to Photo Science for use in the comparative analysis. A total of 36 points were collected along the top of the dam and an additional 7 points were collected on the top of the spillway. Figure H-7 depicts the individual profile stations of both the dam and spillway on top of the 2006 PaMAP orthophoto imagery. Photo Science converted the survey data provided by TerraSurv from UTM, Zone 17N, NAD83 (meters) to Pennsylvania State Plane Coordinate System, North Zone, NAD83 (feet). Elevation values were provided in NAVD88, meters and converted to feet. The dam and spillway profile data provided the "geodetic" input in the comparative analysis.

Figure H-7. Dam and Spillway Profile Stations displayed with
2006 PaMAP Ortho Imagery.

H-15. <u>PaMAP LIDAR Elevation Dataset</u>. The “hydraulic” data source used to support the comparative analysis was the State of Pennsylvania’s spring 2006 PaMAP LIDAR elevation dataset. Photo Science obtained the classified LIDAR point cloud data in native LAS file format covering the dam and spillway area from the Pennsylvania State University, Institute of State and Regional Affairs, Center for Geospatial Information Services located in Middletown, PA. The LIDAR data was acquired by the PaMAP program in the spring of 2006 during leaf off conditions. As depicted in Figures H-8 and H-9, Photo Science utilized the bare earth point class contained in the LAS file to create a ground surface of the dam and spillway area.

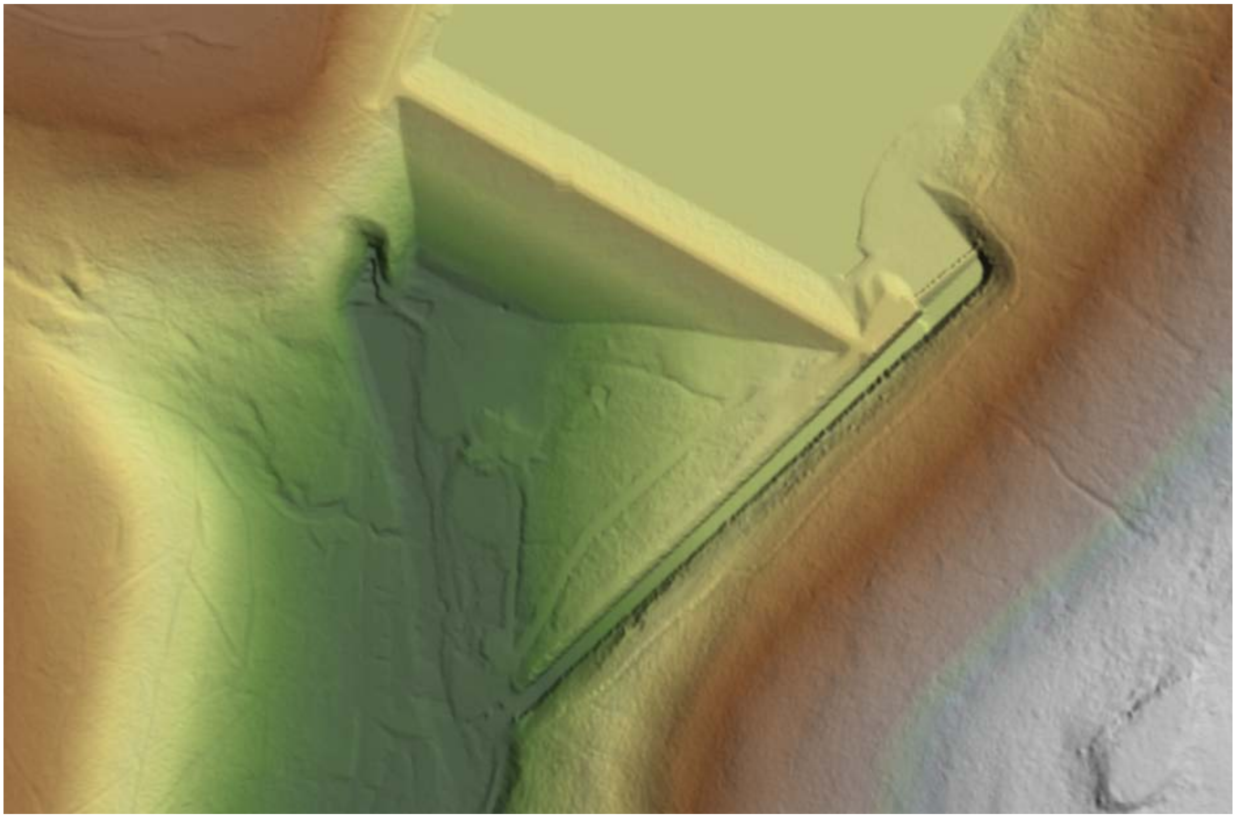


Figure H-8. Top View of East Branch Clarion River Dam & Spillway.

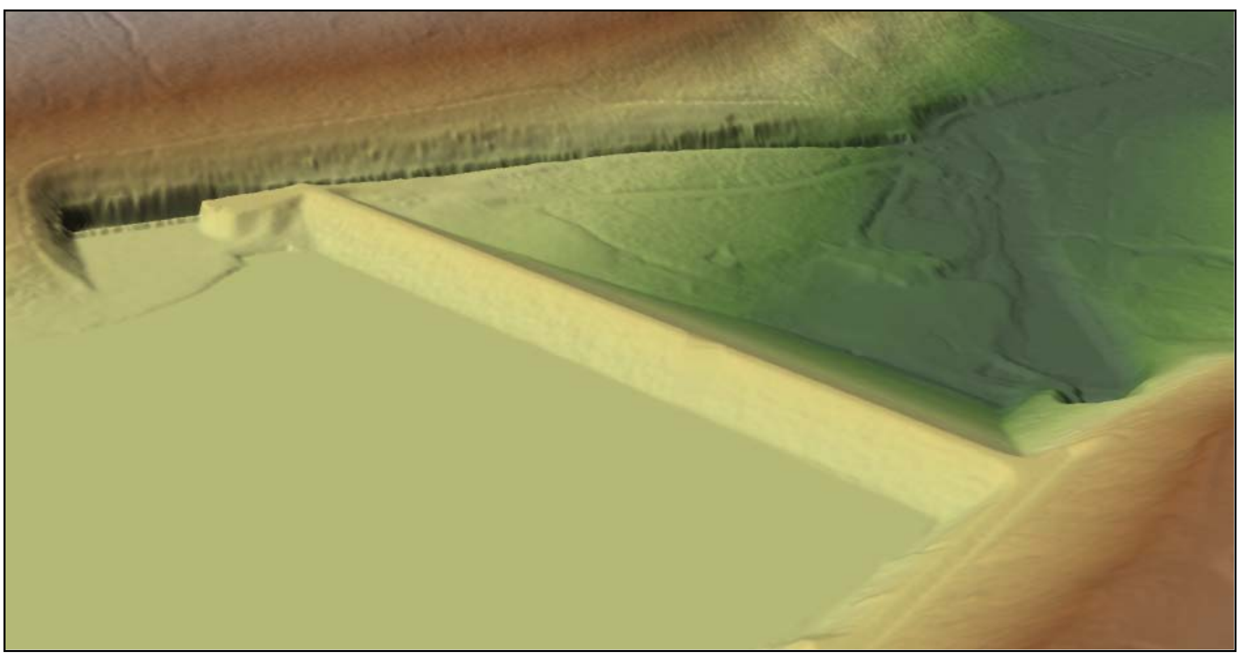


Figure H-9. Isometric view above the East Branch Clarion River Dam & Spillway.

a. The PaMAP LIDAR bare earth data set (in Elk County, PA) was designed to achieve 18.5 cm (0.61feet) vertical RMSE for LIDAR bare earth elevation surface in open terrain. The data was independently tested by PaMAP Quality Assurance Consultant, Dewberry. As shown in Figure H-10, the accuracy of the bare earth in open terrain achieved an RMSE of 0.34 feet and a consolidated RMSE of 0.54 feet for all categories tested.

Dewberry

8401 Arlington Boulevard
Fairfax, Virginia 22031-4666

703 849 0396
703 849 0182 fax
www.dewberry.com

Date: April 7, 2008

Subject: Report of Vertical Accuracy Testing of LiDAR Data for LandAirWoolpert Lidar Block

Project Name: PAMAP
Product Tested: LiDAR DTM
State Plane Zone: Pennsylvania North and South NAD83
Production Block: LandAirWoolpert Block, 2006
Accepted/Rejected: Accepted
Production Company: BAE

Counties in Production Block:
- Elk
- Cameron
- Clearfield
- Centre
- Cambria
- Blair

Table 1. Vertical Accuracy Statistics per NSSDA/FEMA Guidelines

100 % of Totals	RMSE (ft) Open Terr. Spec=0.61ft	Mean (ft)	Median (ft)	Skew	Std Dev (ft)	# of Points	Min (ft)	Max (ft)
Consolidated	0.54	0.22	0.12	2.70	0.49	100	-0.54	2.99
Open Terrain	0.34	0.11	0.12	0.26	0.33	19	-0.40	0.73
High Grass	0.31	0.14	0.11	0.61	0.28	21	-0.26	0.81
Brush	0.74	0.44	0.15	1.53	0.61	20	-0.11	1.99
Forest	0.74	0.34	0.30	3.32	0.68	20	-0.54	2.99
Urban	0.35	0.07	-0.05	1.42	0.36	20	-0.31	0.89

Figure H-10. Dewberry Vertical Accuracy Report of 2006 LIDAR Block covering Elk County, PA.

b. The PaMAP Bare Earth LIDAR surface generated for the dam and spillway provided the “hydraulic” input in the comparative analysis.

H-16. Data Processing. A rectangular polygon was placed around the dam and spillway area, buffered by 1000 feet, for the purposes of reviewing and validating the visual quality the bare earth surface generated from the PaMAP LIDAR data. Minor editing of the bare earth points was performed to improve the quality of the final surface used to perform the analysis. TerraScan and TerraModeler software packages were used to perform for all data classification, manual cleanup, and data analysis. Once the bare earth surface was generated the technician imported the coordinate locations of the survey profiles into the project workspace. These included 36 points along the top of the dam (Figure H-11) and an additional 7 points along the top of the spillway. The profile station locations were then intersected with the 3-D bare earth LIDAR surface. Interpolated LIDAR elevations were then generated using the software at each profile station location. The common coordinates of each survey profile station along with its surveyed elevation, its LIDAR derived elevation and the constant design elevation for the dam and spillway were then exported from TerraScan and imported into Microsoft Excel 2007 for a final statistical analysis. The tabular and graphed results of the analysis of both the dam and spillway are shown in Section H-20 through H-23.

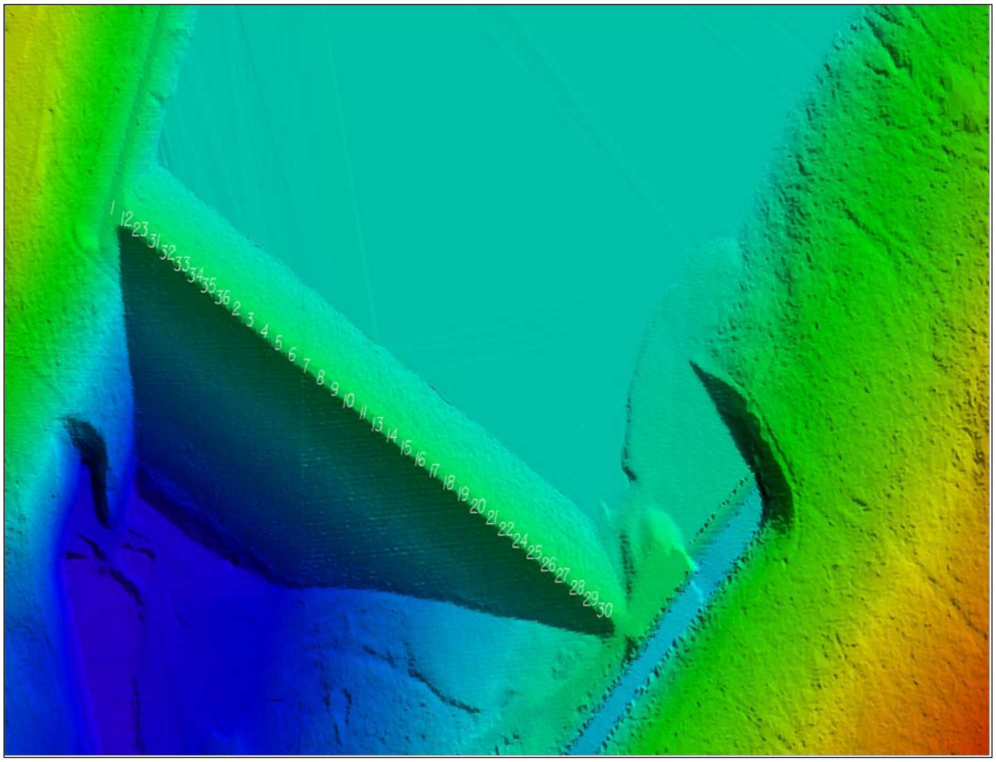


Figure H-11. Bare Earth LIDAR Surface showing Dam profile stations.

H-17. Comparative Analysis Observations. The field survey elevations established along the top of the dam and spillway were consistently higher than the plan elevation. For the dam the magnitude was approximately one half foot and approximately three tenths of a foot for the spillway. The elevations along the top of the dam and spillway established from the LIDAR elevation model were also consistently higher than the plan elevation, but there was considerably more "noise" and variability in the LIDAR elevations as compared to the other two elevation sources. This noise is likely a result of both the nature of LIDAR elevation data, which is acquired in an aerial platform flown several thousand feet above the ground, and the lack of breakline data along the tops of the slopes on the dam that would have improved the performance of the LIDAR only data in modeling the top of the dam. The "noise" in the LIDAR data is not necessarily unusual. As described earlier in this document, this LIDAR dataset was acquired to support a 2-foot contour equivalent surface and as such, included a requirement for an 18.5 cm, or 0.61 feet root mean square error (RMSE). The RMSE basically defines the 68 percent confidence interval, or put in other words, 68 percent of the elevation points within this dataset should fit the actual surface of the earth within 18.5 cm, or 0.61 feet. By visual inspection of the graphs for both the dam and the spillway profiles we can see that the LIDAR elevations fit the ground elevations established by field survey within 0.5 feet at most of the comparison locations, which would fit within our statistical expectations for the LIDAR data based on the accuracy standard

H-18. Project Glossary.

LIDAR- Light Detection and Ranging.

Average dZ – the average elevation value from the list of each series of readings.

Minimum dZ – the minimum elevation value from the list of each series of readings.

Maximum dZ – the maximum elevation value from the list of each series of readings.

MSE - Mean Square Error is achieved by calculating the square of the deviations of points from their true position, summing up the measurements, and then dividing by the total number of points.

RMSE – Root Mean Square Error is calculated by taking the square root of the MSE.

Standard Deviation – measure of how widely values are dispersed from the average dZ.

H-19. Methodology for Calculating the dZ Values.

dZ (Survey/Plan) – Plan elevation was subtracted from the surveyed elevation.

dZ (Survey/LIDAR) – LIDAR elevation was subtracted from the survey elevation.

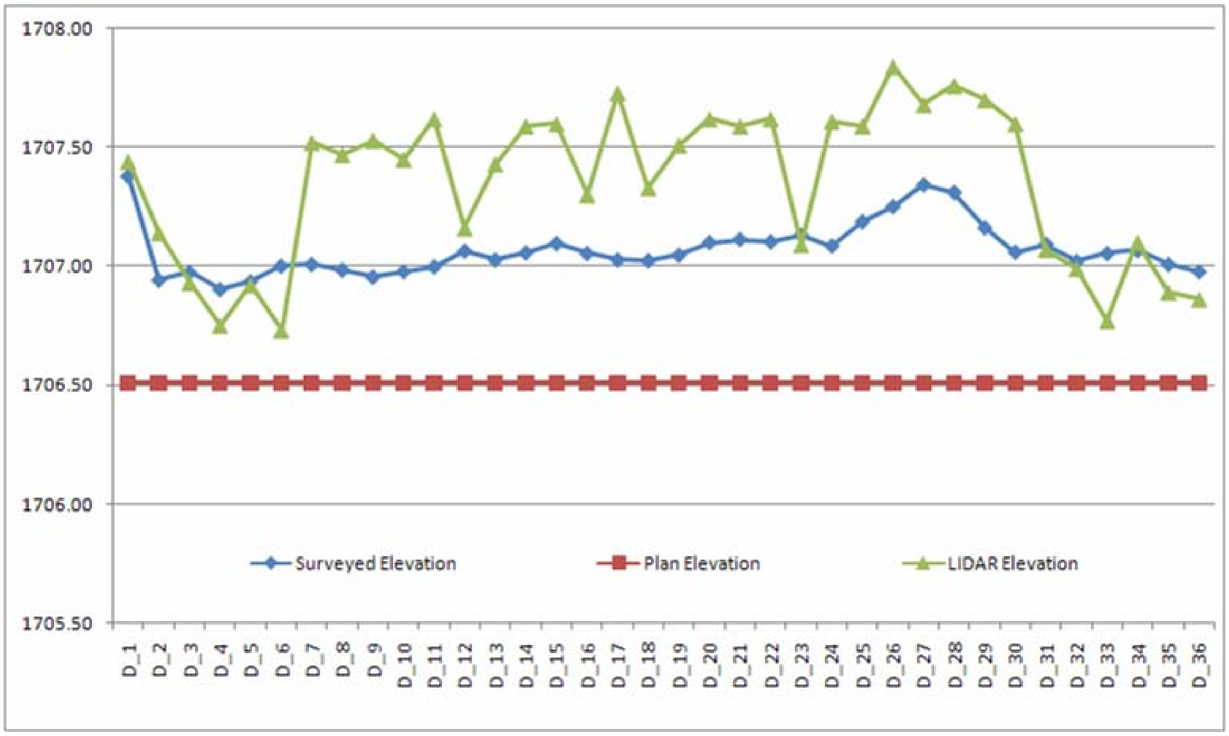
dZ (Plan/LIDAR) – LIDAR elevation was subtracted from the plan elevation.

H-20. Statistical Analysis—Dam.

Dam Statistical Analysis			
Average Dz	0.56	-0.278	-0.838
Minimum Dz	0.39	-0.70	-1.33
Maximum Dz	0.87	0.28	-0.22
RMSE	0.571	0.398	0.899
St. Dev.	0.11	0.289	0.33

Point	Elevation			dZ			Easting	Northing
	Surveyed	Plan	LIDAR	Survey/Plan	Survey/LIDAR	Plan/LIDAR		
D_1	1707.38	1706.51	1707.44	0.87	-0.06	-0.93	1736840.37	508754.83
D_2	1706.94	1706.51	1707.14	0.43	-0.20	-0.63	1737189.90	508558.95
D_3	1706.98	1706.51	1706.93	0.46	0.04	-0.42	1737229.23	508537.05
D_4	1706.90	1706.51	1706.75	0.39	0.15	-0.24	1737269.25	508514.86
D_5	1706.94	1706.51	1706.92	0.43	0.02	-0.41	1737309.73	508492.06
D_6	1707.00	1706.51	1706.73	0.49	0.27	-0.22	1737349.08	508469.89
D_7	1707.01	1706.51	1707.52	0.50	-0.51	-1.01	1737390.15	508446.86
D_8	1706.99	1706.51	1707.47	0.47	-0.49	-0.96	1737429.28	508425.16
D_9	1706.96	1706.51	1707.53	0.44	-0.58	-1.02	1737468.98	508402.62
D_10	1706.98	1706.51	1707.45	0.47	-0.47	-0.94	1737508.45	508380.57
D_11	1707.00	1706.51	1707.62	0.49	0.62	1.11	1737549.94	508357.56
D_12	1707.06	1706.51	1707.16	0.55	-0.10	-0.65	1736879.94	508733.03
D_13	1707.03	1706.51	1707.43	0.52	-0.40	-0.92	1737590.29	508334.64
D_14	1707.06	1706.51	1707.59	0.55	-0.53	-1.08	1737630.73	508312.62
D_15	1707.10	1706.51	1707.60	0.59	-0.50	-1.09	1737671.51	508289.43
D_16	1707.05	1706.51	1707.30	0.54	-0.25	-0.79	1737711.33	508267.09
D_17	1707.03	1706.51	1707.73	0.52	-0.70	-1.22	1737752.00	508244.48
D_18	1707.02	1706.51	1707.33	0.51	-0.31	-0.82	1737792.72	508221.76
D_19	1707.05	1706.51	1707.51	0.54	-0.46	-1.00	1737832.91	508199.68
D_20	1707.10	1706.51	1707.62	0.59	-0.52	-1.11	1737873.90	508176.72
D_21	1707.11	1706.51	1707.59	0.60	-0.48	-1.08	1737914.50	508154.14
D_22	1707.10	1706.51	1707.62	0.59	-0.52	-1.11	1737953.58	508131.90
D_23	1707.13	1706.51	1707.09	0.62	0.04	-0.58	1736918.46	508711.93
D_24	1707.09	1706.51	1707.61	0.58	-0.52	-1.10	1737993.20	508109.57
D_25	1707.19	1706.51	1707.59	0.68	-0.40	-1.08	1738034.14	508086.55
D_26	1707.25	1706.51	1707.84	0.74	-0.59	-1.33	1738074.32	508063.43
D_27	1707.34	1706.51	1707.68	0.83	-0.34	-1.17	1738114.04	508041.69
D_28	1707.31	1706.51	1707.76	0.80	-0.45	-1.25	1738155.30	508018.97
D_29	1707.16	1706.51	1707.70	0.65	-0.54	-1.19	1738193.35	507997.10
D_30	1707.06	1706.51	1707.60	0.55	-0.54	-1.09	1738232.99	507974.79
D_31	1707.09	1706.51	1707.07	0.58	0.02	-0.56	1736957.60	508690.05
D_32	1707.02	1706.51	1706.99	0.51	0.03	-0.48	1736995.53	508667.85
D_33	1707.05	1706.51	1706.77	0.54	0.28	-0.26	1737036.58	508644.95
D_34	1707.07	1706.51	1707.10	0.56	-0.03	-0.59	1737074.70	508623.45
D_35	1707.01	1706.51	1706.89	0.50	0.12	-0.38	1737111.47	508602.91
D_36	1706.98	1706.51	1706.86	0.47	0.12	-0.35	1737151.46	508580.47

H-21. Dam Statistical Analysis – Graph.

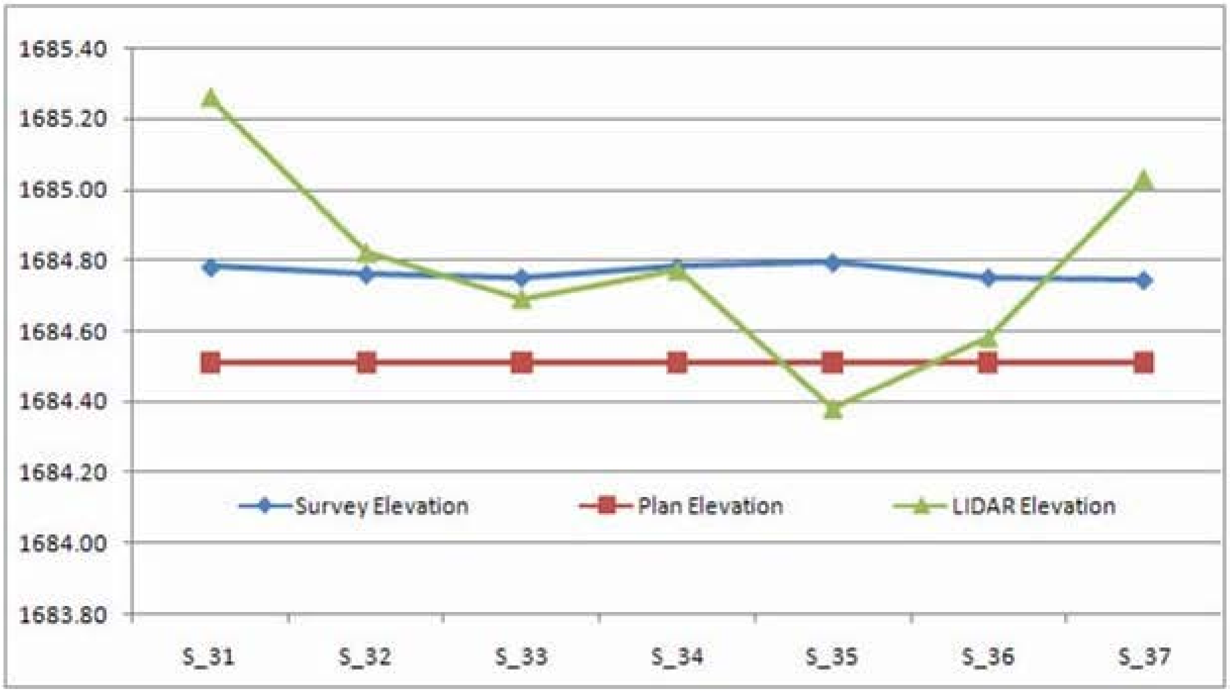


H-22. Statistical Analysis—Spillway.

Spillway Statistical Analysis			
Average Dz	0.256	-0.024	-0.28
Minimum Dz	0.23	-0.48	-0.75
Maximum Dz	0.28	0.41	0.13
RMSE	0.256	0.272	0.387
St. Dev.	0.019	0.293	0.289

Point	Elevation			dZ			Easting	Northing
	Surveyed	Plan	LIDAR	Survey/Plan	Survey/LIDAR	Plan/LIDAR		
S_31	1684.78	1684.51	1685.26	0.27	-0.48	-0.75	1738491.30	508110.83
S_32	1684.76	1684.51	1684.82	0.25	-0.06	-0.31	1738513.26	508128.62
S_33	1684.75	1684.51	1684.69	0.24	0.06	-0.18	1738535.40	508146.52
S_34	1684.78	1684.51	1684.77	0.27	0.01	-0.26	1738557.30	508164.61
S_35	1684.79	1684.51	1684.38	0.28	0.41	0.13	1738579.46	508182.41
S_36	1684.75	1684.51	1684.58	0.24	0.17	-0.07	1738601.59	508200.15
S_37	1684.74	1684.51	1685.03	0.23	-0.29	-0.52	1738624.87	508219.20

H-23. Spillway Statistical Analysis – Graph.



H-24. East Branch Control Tower Gage. The following is a copy of a 2008 USGS gage inspection report for the Control Tower gage. Note that elevations are referenced to both "MSL" and local gage (and electric tape) datums. The 2008 TerraSurv surveys described above subsequently provided relationships for this gage to the NSRS (NAVD88). The legacy datum (MSL) should be retained along with its relationship to the updated NAVD88 elevations. Figure H-12 is a close up of the gage reference point. Figure H-13 is a copy of the U-SMART datasheet for this gage.

Gage Station Description 03027000
East Branch Clarion River Lake, PA

Responsible Office U.S. Geological Survey, Pittsburgh Field Office, 1000 Church Hill Rd., Pittsburgh, PA 15205 (412) 490-3800

Most recent revision: 5/13/2008 Revised by: ajruddy

LOCATION.--Lat 41°33'35", long 78°35'40" referenced to North American Datum of 1927, Elk County, PA, Hydrologic Unit 05010005, gage house in control tower at East Branch Clarion River Dam on East Branch Clarion River, 1.7 miles northeast of Glen Hazel, and 7.5 miles upstream from confluence with West Branch Clarion River.

ROAD LOG.--To reach station from Johnsonburg travel east on Bendingo Rd. from Johnsonburg to village of Glen Hazel. At Glen Hazel make left turn at "T" intersection onto Glen Hazel Rd. (SR240011). Follow Glen Hazel Rd. across bridge over the East Branch Clarion River and proceed 1.0 mile. Make right turn onto Corps of Engineers access road at sign. Follow access road to Corps office and obtain key for access and directions if necessary (Glen Hazel, 7 1/2 minute quadrangle).

DRAINAGE AREA.--72.4 mi2.

ESTABLISHMENT AND HISTORY.--June 1952 to Oct. 1991 and from July 2005 to current year. Prior to October 1970 published as "East Branch Clarion River Reservoir".

GAGE.--Sutron (model 8210) data collection platform at top of concrete stilling well built into gate tower building. Recorders are referenced to an electric tape-gage at 1710.323 ft. gage datum. Elevation of gage datum provided by Corps of Engineers. Electric tape index is 210.323 ft gage datum. Electric tape-gage reading plus 1,500.00 equals reservoir elevation to sea level. Corps Conventions for Recording Pool Levels: E.T. plus 1500 ft is sea level for pool reading. Electric Tape reading minus 85 ft is DCP reading for transmissions.

RESERVOIR: Rock faced earthfill dam with a capacity of 83,300 acre-ft. Range in usual operation is between 1,651 ft and 1,670 ft above sea level. Full range of operation is between 1,555 ft (sill of outlet gates) and 1,685 ft (full pool).

CONTROL.--Spillway and gate opening are the control factors.

DISCHARGE.--Controlled by two outlet gates whose dimensions are 3' by 4' and 1' by 1.5'.

FLOODS.--The high water of June 24, 1972 reached an elevation of 1,685.55 ft (85,010 acre-ft).

POINT OF ZERO FLOW.--Elevation of sill of the outlet gates is 15.0 ft. gage datum.

REGULATION AND DIVERSIONS.--Reservoir is operated for flood control, low-flow augmentation of Clarion River and recreational use.

ACCURACY.--Records good.

COOPERATION.--The station is maintained cooperatively by the U.S. Army Corps of Engineers and the U.S. Geological Survey.

REFERENCE MARKS.-- BM 1-500 COE brass disk on top of parapet wall at service bridge to control tower on upstream right bank. Elevation is 1711.003 ft, MSL. BM M-1 COE pipe monument right bank just downstream of dam access road. Elevation is 1711.209 ft, MSL. BM M-2 COE pipe monument, left bank just downstream of dam access road. Elevation is 1711.293 ft, MSL.

DATE OF LAST LEVELS. Last run: Jun 15, 2006; Next run: Jun 14, 2009; Frequency: 3 years; Status: OPEN

Figure H-12. East Branch Control Tower gage reference point.

USACE Survey Marker Archive & Retrieval Tool Datasheet Type: New

Designation: Gauge East Branch (inflow)
Project: East Branch
Stamping:
PID NGS: COE:
State: Pennsylvania
County: Elk
District Pittsburgh
Nearest Town: Johnsonburg
USGS Quad:
T.R.S.:
Nearest Hwy/Mi:
B/L Sta/Off:
Date Recovered: Apr 22, 2008
By: CELRP (LeBlanc)
Condition/Stability: Good
Setting/Monument Type:
Owner:
GPS Suitable: Yes (●) No
Obstructions: N E S W

- Horizontal -		- Vertical -	
Datum: NAD83 ( )		Datum: NAVD88 ( )	
Lat: 41.56140000	N	Elevation Ht: 1,710.323	
Lon: 78.59470000	W	Ellip Ht:	Ft
Local Accuracy:	10-m+	Local Accuracy:	2-cm
NSRS Accuracy:	10-m+	NSRS Accuracy:	0.25'
Survey/Computation Method: Scaled		Survey/Computation Method: Geodetic Levels	
Date Observed: May 20, 2008		Date Observed: May 20, 2008	Geoid09

Description/Comments:
East Branch Clarion River Lake, PA (EBRP)

Rovergages.com
http://www2.mvr.usace.army.mil/WaterControl/stationinfo2.cfm?sid=EBRP1&fid=EBRP1&dt=S

Access:

- Tidal/Hydraulic Gauge Relationships -
Owner/Code: USGS
Gauge ID: EBRP Epoch:

- Datum -	- Elevation -
Gage Index	210.323
DCP Datum	85
Select	
Select	

Zone: Northing: Easting: Convergence: CSF:

- Horizontal View - - Close-Up View -

Required Fields In Red Reset Form Submit U-SMART ver 1.1 9/24/2009

Figure H-13. U-SMART datasheet for East Branch Control Tower gage.

APPENDIX I

Control Surveys: Bois Brule Levee and Drainage District (St. Louis District)

I-1. Purpose. This appendix provides an example of establishing primary NSRS control and supplemental local control on a levee segment along the Mississippi River. This example is typical of the survey procedures employed to establish NSRS control on any levee segment. This project was completed as part of the St. Louis District's effort in updating levee inventory information for inclusion in the National Levee Database (NLD). The database survey was performed by PBS&J—reference report "National Levee Foot Print Database Surveys," Contract W9133L-05-D-0003 DJ06, dated 19 May 2008.

I-2. Project Location. The Bois Brule Levee and Drainage District is located in northern Perry County, Missouri. The protected area is located on the right bank of the Mississippi River. The total length of flood protection is 38.84 miles long. This includes 38.7 miles of earthen levee (204,308.82 feet), 0.03 miles (146.52 feet) of floodwall, and 0.04 miles (190.99 feet) of closure structures. The protected area is roughly 26,350 acres.

I-3. Survey Control Methods Used to Connect Levees to the NSRS. GPS (RTK) survey methods were employed to establish control on various levee segments along the Mississippi River, as shown in Figure I-1. In the St. Louis area, a RTN network was used. North and south of the St. Louis RTN coverage (including the Bois Brule Levee District), standard RTK methods were employed. This involved recovering at least two published NSRS control points near the levee segment and using these points as a RTK base station. RTK checks between NSRS points were made to confirm the reliability of the NSRS points. Supplemental topographic surveys of levee features were made using RTK techniques.

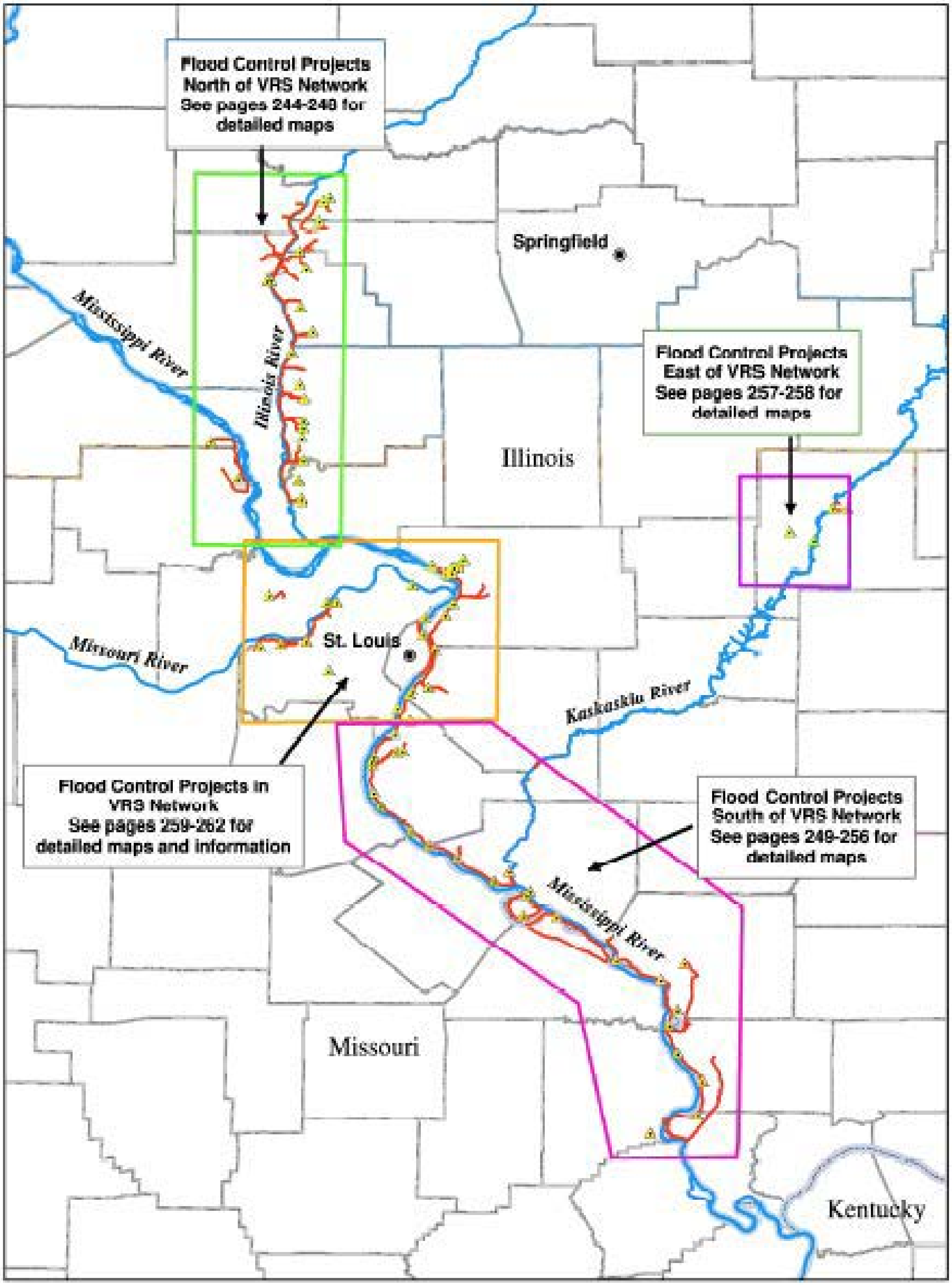


Figure I-1. Overview of survey control used to reference St. Louis District levees.

I-4. Bois Brule Primary Control Points. As shown on Figure I-2, two NSRS control bench marks were recovered in the vicinity of the Bios Brule Levee District—"R 323" (PID=HB1394) and "L 289" (PID=HB1377). These two NSRS points are approximately 10 miles apart. They were designated as PPCPs for this levee project. They are close enough to check internal RTK site calibration. Both points can be occupied with RTK base stations. NGS datasheets for these two points are at the end of this appendix.

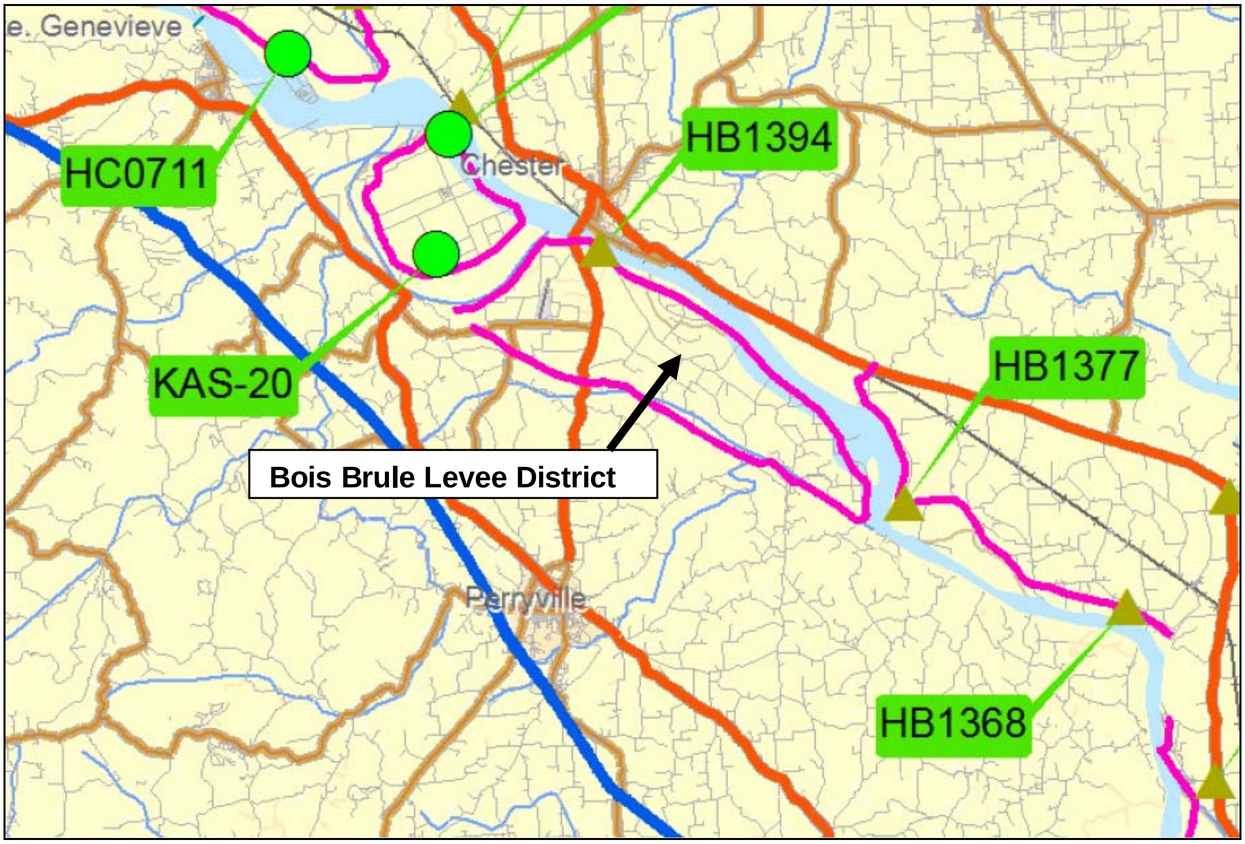
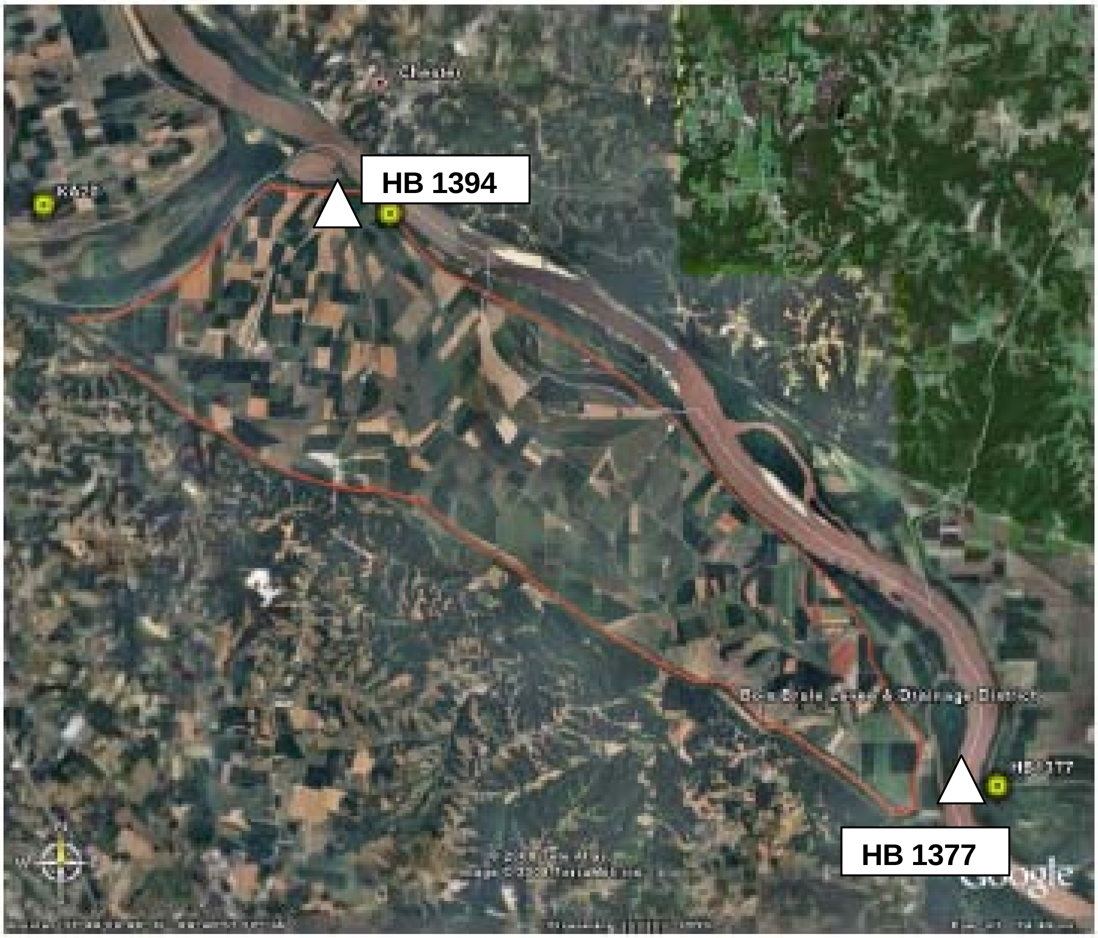


Figure I-2. NSRS control recovered vicinity of Bois Brule Levee District.

I-5. Survey Control / Data Collection. The following Figures I-3, I-4, and I-5 depict the primary NSRS control relative to the levee district boundary and the updated field station descriptions prepared for the NLD inventory report.



Survey Control / Data Collection

Vertical Datum: NAVD88
Horizontal Datum: NAD83 (Epoch: 2004.65)
Coordinates: Missouri State Plane East

Station Name/PID	North	East	Elevation NAVD 88	Description	Stamping
HB1394	749651.50 (MO EAST)	1014719.93 (MO EAST)	389.8	STEEL ROD W/O SLEEVE	R 323 1981
HB1377	704076.55 (MO EAST)	1064237.47 (MO EAST)	381.87	STEEL ROD W/O SLEEVE	L 289 1981

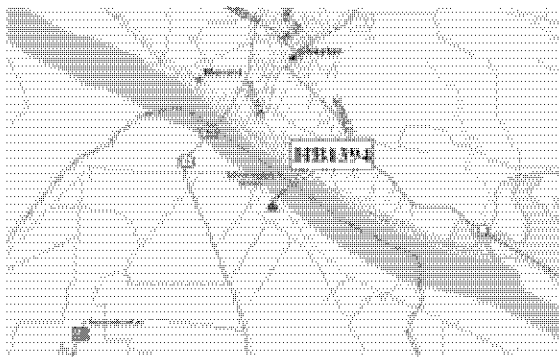
Figure I-3. NSRS control for PPCPs HB1394 and HB1377 (Bois Brule Levee District).

St. Louis District National Levee Inventory Database

Survey Control Data Sheet

District: St. Louis

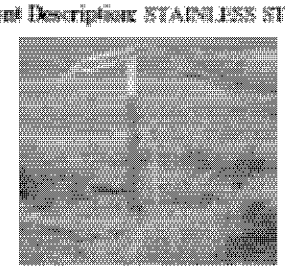
Station: R 323
NGS PID: HB1394
Vicinity Map: DeLorme Street Atlas USA 2006
State: MO
Region: Mid West
Nearest Town: Chester, IL
USGS Quad: CHESTER (1993)
Nearest Hwy: I-55
Hwy Mile Post: N/A
B/L Station: N/A
B/L Offset: N/A
Levees: Bois Brule



Station Name: R 323 **NGS PID:** HB1394 **Date Set:** 1981 **Set By:** NGS **Date Recovered:** N/A **Recovered By:** N/A **Owner:** NGS	**NAD83** **Lat:** 37 53 25.83285 **Lon:** 89 49 33.44123 **MO SPCS83 East Zone US SF** **Northing:** 749631.50 **Easting:** 1014719.97 **NAVD88(2004.65)** **Elev:** 389.80' **Scale Factor/Convergence:** N/A	**Controlling Stations:** N/A **Survey/Computation Method:** The orthometric height was determined by differential leveling and adjusted by the NATIONAL GEODETIC SURVEY in June 1991. Horizontal was determine using static, dual-frequency observation. **Local Network Accuracy:** N/A N/A

Monument Location: THE STATION IS LOCATED ABOUT 24.1 KM (14.95 MI) NORTH-NORTHEAST OF PERRYVILLE, MO. AND ABOUT 3.2 KM (2.00 MI) SOUTH OF AND ACROSS THE MISSISSIPPI RIVER FROM CHESTER, IL., ALONG THE SOUTHWEST SIDE OF THE RIVER, IN THE GRASS NEAR THE TOP SOUTHWEST EDGE OF THE LEVEE ROAD, NEAR THE NORTHWEST END OF A FIELD ACCESS TRACK ROAD ON THE RIGHT, AND ACROSS THE LEVEE ROAD FROM A RIVER ACCESS ROAD ON THE LEFT. OWNERSHIP--ARMY CORPS OF ENGINEERS. TO REACH THE STATION FROM THE JUNCTION OF STATE HIGHWAY 51 AND THE LEVEE ROAD, NEAR THE SOUTHWEST END OF THE BRIDGE OVER THE MISSISSIPPI RIVER, SOUTHWEST OF CHESTER, IL., GO SOUTHEASTERLY, 1.64 KM (1.00 MI) ALONG THE LEVEE ROAD TO THE STATION ON THE RIGHT EDGE OF THE ROAD, JUST NORTHWEST OF A WITNESS POST. THE STATION IS 12.4 M (40.7 FT) SOUTHEAST OF A CULVERT RUNNING UNDER THE LEVEE, WITH THE ENDS OPEN TO A DITCH ON THE SOUTHWEST SIDE OF THE LEVEE AND A VALVE ON THE NORTHEAST SIDE OF THE LEVEE, 3.4 M (11.2 FT) SOUTHWEST OF THE LEVEE CENTER, 0.4 M (1.3 FT) NORTHWEST OF A WITNESS POST, AND THE STATION IS ABOUT 0.3 M (1.0 FT) BELOW THE ROAD LEVEL AND FLUSH WITH THE GROUND SURFACE. BY R.G. HAYES. NOTE--THE DATUM POINT IS A PUNCH MARK ON THE TOP CENTER OF A STAINLESS STEEL DATUM POINT WHICH IS CRIMPED TO THE TOP OF A STAINLESS STEEL ROD, DRIVEN TO A DEPTH OF 4.3 M, (14.1 FT) ENCASED IN A 5-INCH PVC PIPE WITH NGS LOGO CAP, SURROUNDED BY CONCRETE. ACCESS TO THE DATUM POINT IS THROUGH THE 5-INCH LOGO CAP.

Monument Description: STAINLESS STEEL ROD W/O SLEEVE (10 FT.+)



I-56

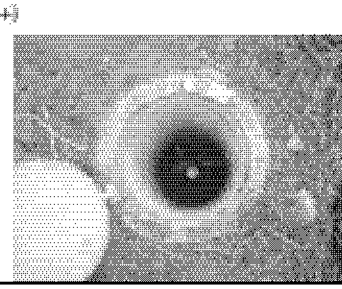


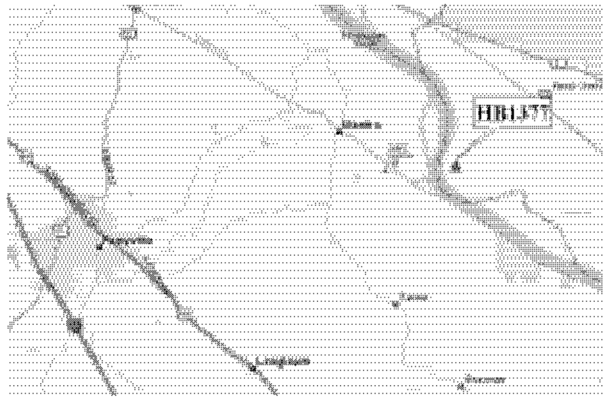
Figure I-4. Datasheet description for PBM "R 323" (HB1394) (Bois Brule Levee District).

St. Louis District National Levee Inventory Database

Survey Control Data Sheet

District: St. Louis

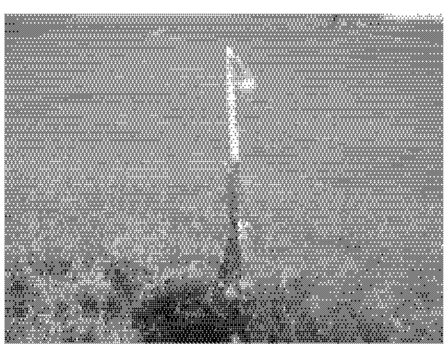
Station: L 289
NGS PID: HB1377
Vicinity Map: DeLorme Street Atlas USA 2006
State: IL
Region: Mid West
Nearest Town: Jacob, IL
USGS Quad: ROCKWOOD (1994)
Nearest Hwy: I-55
Hwy Mile Post: N/A
B/L Station: N/A
B/L Offset: N/A
Levees: Bois Brule., Degognia



Station Name: L 289 **NGS PID:** HB1377 **Date Set:** 1981 **Set By:** NGS **Date Recovered:** N/A **Recovered By:** N/A **Owner:** NGS	**NAD83** **Lat:** 37 45 51.30943 **Lon:** 89 39 21.89599 **MO SPCS83 East Zone US SF** **Northing:** 704076.55' **Easting:** 1064237.47' **NAVD88(2004.65)** **Elev:** 381.87' **Scale Factor/Convergence:** N/A	**Controlling Stations:** N/A **Survey/Computation Method:** The orthometric height was determined by differential leveling and adjusted by the NATIONAL GEODETIC SURVEY in June 1991. Horizontal was determine using static, dual-frequency observation. **Local Network Accuracy:** N/A N/A

Monument Location: 8.1 KM (5.05 MI) SOUTH FROM CORA. 8.1 KILOMETERS (5.05 MILES) SOUTH ALONG THE TOP OF THE MAIN LEVEE FROM THE INTERSECTION OF THE MISSOURI PACIFIC RAILROAD IN CORA TO A BEND IN THE LEVEE AND THE MARK ON THE LEFT, AT THE JUNCTION OF A SPUR LEVEE LEADING SOUTHEAST, IN LINE WITH THE CENTER OF THE SPUR LEVEE, 4.57 METERS (15.0 FEET) NORTH OF THE CENTER OF THE LEVEE ROAD AND 0.46 METERS (1.5 FEET) EAST OF A METAL WITNESS POST. THE MARK IS 0.46 METERS W FROM A WITNESS POST. THE MARK IS 0.15 M BELOW TOP OF LEVEE.

Monument Description: STAINLESS STEEL ROD IN SLEEVE (10 FT.+)



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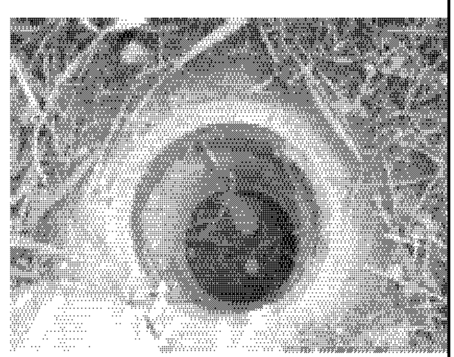

Figure I-5. Datasheet description for PBM "L 289" (HB1377) (Bois Brule Levee District).

I-6. Bois Brule Levee District Project Features Surveyed Relative to NSRS Primary Control Points HB1394 and HB1377. RTK feature and topographic surveys were performed relative to the PPCPs cited above. The top elevation of the levee varies from 381.94 to 395.09 feet with an average top width of 7.43 feet along the earthen levee sections. The features associated with the levee structure are listed below:

Boreholes: None Captured
Encroachment Points: 183
Flood Fight Points: None Captured
Crossing Points: 141
Failure Points: None Captured
Relief Wells: 427
Piezometers: 23
Pump Stations: 4
Sand Boils: None Captured
Closure Structure Count: 3
Cross Sections: 29
Floodwall Lines: 4
Gravity Drains: 26
Rehab Lines: None Captured
Toe Drains: None Captured

The twenty-nine cross-sections were taken along the levee at stations 8+07, 75+34, 150+98, 182+25, 237+31, 313+64, 371+80, 436+29, 493+32, 575+88, 637+63, 704+83, 790+03, 832+25, 880+66, 920+53, 971+90, 1037+63, 1103+81, 1144+14, 1208+87, 1272+00, 1336+40, 1388+08, 1476+63, 1541+14, 1581+70, 1640+48, and 1725+92.

I-7. Use of RTN Networks for Referencing Levee Control. Figure I-6 depicts RTN network coverage in the St. Louis region. Levee control and NLD inventory points can be directly surveyed from such an RTN network. Prior to performing supplemental surveys, the RTN network is calibrated against existing NSRS control in the vicinity of the levee—as highlighted in green in Figure I-6. Figures I-7 and I-8 show the results of the RTN calibration checks at selected NSRS points. These results indicate vertical checks are within ± 0.25 ft tolerances. The calibration differences at NSRS "tie points" should be applied to local observations—i.e., "site calibration."

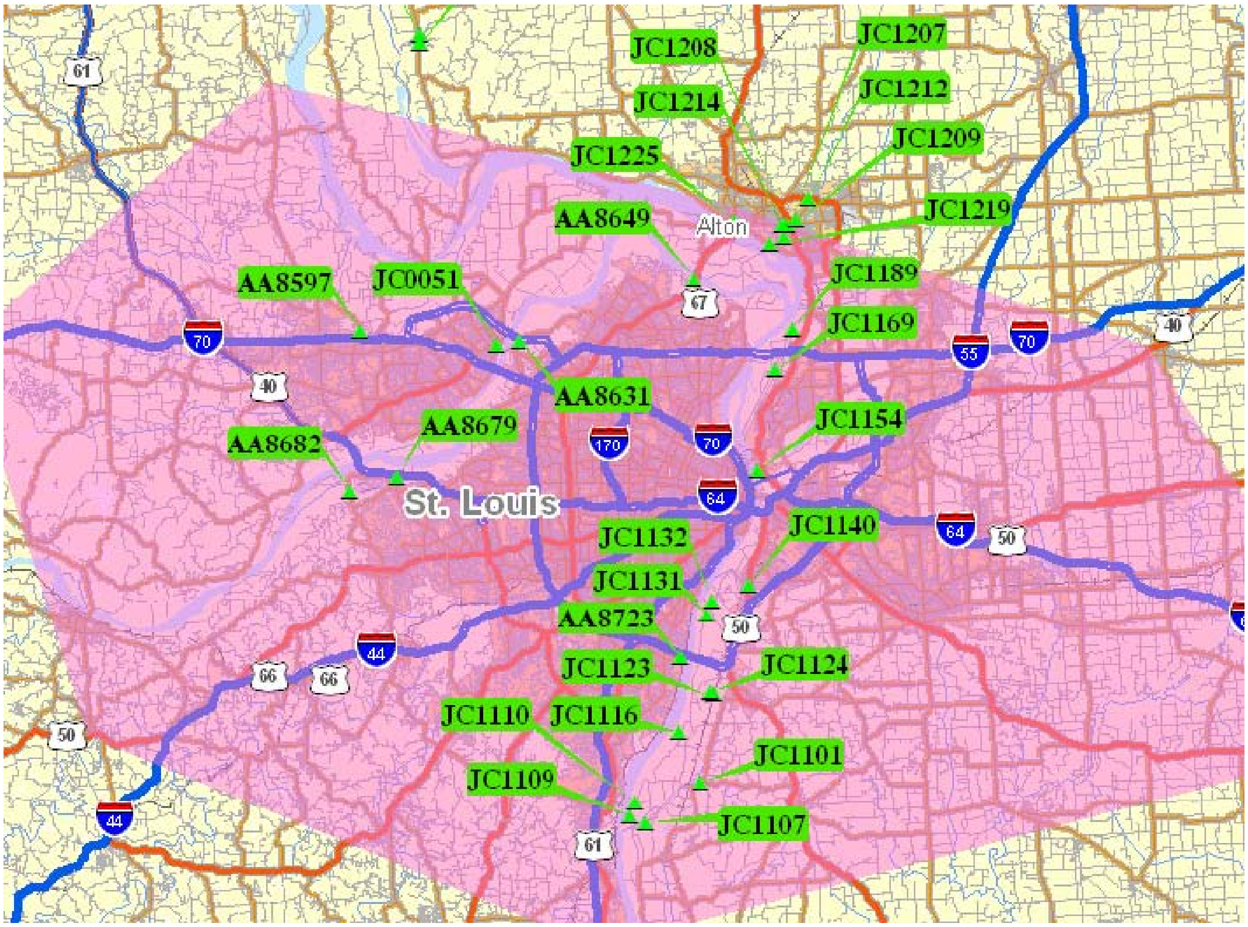

Figure I-6. RTN coverage in the St. Louis region and NSRS check points.

VRS BASE STATION	NGS STATION	DISTRICTS USED IN
SRDX	DI2212	Monarch Chesterfield
SIAI	(No PID)	Metro East, Chain of Rocks, Wood River
SIHQ	DH7921	Metro East, Chain or Rocks, Prairie DuPont, Columbia
WIFH	DI2210	Metro East, Chain of RocksPrairie DuPont
TWMW	DI2208	Prairie DuPont, Columbia, Fish Lake
FWIF	(No PID)	Nutwood, Eldred,Spankey

Figure I-7. RTN (VRS) site calibration points for various levee segments in St. Louis area.

RMS TABLE							
PROJECT:	Midwest RTK Network RMS Check						
DATE:	April 30, 2007						
TIE POINT	N	E	Z	REF(BM)	N	E	Z
AA8597	1083358.929	777759.266	528.897	AA8597	1083358.936	777759.200	529.0
AA8631	1076448.529	833046.498	458.900	AA8631	1076448.429	833046.410	459
AA8649	1094248.925	894738.346	486.682	AA8649	1094248.849	894738.276	487
AA8723	965171.227	882610.836	416.358	AA8723	965171.290	882610.656	416.4
JC0051			527.494	JC0051			527.73
JC1107			413.786	JC1107			413.99
JC1132			426.435	JC1132			426.32
JC1212 (O/S)			468.830	JC1212 (O/S)			468.93
JC1225 (O/S)			554.390	JC1225 (O/S)			554.52
HORIZ.RMS =	0.13						
VERT.RMS =	0.17						

Figure I-8. RTN (VRS) site calibration "Published – Observed" differences.

I-8. NGS Data Sheet for R 323 (PID HB1394).

```
HB1394 ***********************************************************************
HB1394  FBN         -  This is a Federal Base Network Control Station.
HB1394  DESIGNATION -  R 323
HB1394  PID         -  HB1394
HB1394  STATE/COUNTY-  MO/PERRY
HB1394  USGS QUAD   -  CHESTER (1993)
HB1394
HB1394                         *CURRENT SURVEY CONTROL
HB1394  ___________________________________________________________________
HB1394* NAD 83(2007)-  37 53 25.83293(N)    089 49 33.44072(W)     ADJUSTED
HB1394* NAVD 88     -       118.811  (meters)     389.80   (feet)  ADJUSTED
HB1394  ___________________________________________________________________
HB1394  EPOCH DATE  -        2002.00
HB1394  X           -      15,309.625 (meters)                     COMP
HB1394  Y           -  -5,039,949.530 (meters)                     COMP
HB1394  Z           -   3,895,914.999 (meters)                     COMP
HB1394  LAPLACE CORR-           0.93  (seconds)                    USDV2009
HB1394  ELLIP HEIGHT-          89.321 (meters)          (02/10/07) ADJUSTED
HB1394  GEOID HEIGHT-         -29.48  (meters)                     GEOID09
HB1394  DYNAMIC HT  -         118.730 (meters)     389.53  (feet)  COMP
HB1394
HB1394  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
HB1394  Type    PID    Designation                       North   East  Ellip
HB1394  -------------------------------------------------------------------
HB1394  NETWORK HB1394 R 323                              0.41   0.31   1.12
HB1394  -------------------------------------------------------------------
HB1394  MODELED GRAV-     979,943.3   (mgal)                       NAVD 88
HB1394
HB1394  VERT ORDER  -  FIRST     CLASS II
HB1394
HB1394.The horizontal coordinates were established by GPS observations
HB1394.and adjusted by the National Geodetic Survey in February 2007.
HB1394
HB1394.The datum tag of NAD 83(2007) is equivalent to NAD 83(NSRS2007).
HB1394.See National Readjustment for more information.
HB1394.The horizontal coordinates are valid at the epoch date displayed above.
HB1394.The epoch date for horizontal control is a decimal equivalence
HB1394.of Year/Month/Day.
HB1394
HB1394.The orthometric height was determined by differential leveling
HB1394.and adjusted in June 1991.
HB1394
HB1394.The X, Y, and Z were computed from the position and the ellipsoidal ht.
HB1394
HB1394.The Laplace correction was computed from USDV2009 derived deflections.
HB1394
HB1394.The ellipsoidal height was determined by GPS observations
HB1394.and is referenced to NAD 83.
HB1394
HB1394.The geoid height was determined by GEOID09.
HB1394
HB1394.The dynamic height is computed by dividing the NAVD 88
HB1394.geopotential number by the normal gravity value computed on the
HB1394.Geodetic Reference System of 1980 (GRS 80) ellipsoid at 45
HB1394.degrees latitude (g = 980.6199 gals.).
HB1394
HB1394.The modeled gravity was interpolated from observed gravity values.
HB1394
HB1394;                   North         East     Units Scale Factor Converg.
HB1394;SPC MO E     -   228,494.237   309,287.266   MT  0.99997661   +0 24 50.3
HB1394;UTM  16      - 4,197,432.411   251,495.217   MT  1.00036068   -1 44 11.3
```

```
HB1394
HB1394!             -  Elev Factor  x  Scale Factor =   Combined Factor
HB1394!SPC MO E     -   0.99998598  x   0.99997661  =   0.99996259
HB1394!UTM  16      -   0.99998598  x   1.00036068  =   1.00034666
HB1394
HB1394                          SUPERSEDED SURVEY CONTROL
HB1394
HB1394  ELLIP H (02/11/04)   89.315  (m)                       GP(       ) 4 1
HB1394  NAD 83(1997)-  37 53 25.83260(N)    089 49 33.44129(W) AD(       ) B
HB1394  ELLIP H (03/31/98)   89.326  (m)                       GP(       ) 3 1
HB1394  NAVD 88 (03/31/98)  118.81   (m)          389.8    (f) LEVELING    3
HB1394  NGVD 29 (??/??/??)  118.742  (m)          389.57   (f) ADJUSTED    1 2
HB1394
HB1394.Superseded values are not recommended for survey control.
HB1394.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
HB1394.See file dsdata.txt to determine how the superseded data were derived.
HB1394
HB1394_U.S. NATIONAL GRID SPATIAL ADDRESS: 16SBG5149597432(NAD 83)
HB1394_MARKER: I = METAL ROD
HB1394_SETTING: 49 = STAINLESS STEEL ROD W/O SLEEVE (10 FT.+)
HB1394_SP_SET: STAINLESS STEEL ROD
HB1394_STAMPING: R 323 1981
HB1394_MARK LOGO: NGS
HB1394_PROJECTION: FLUSH
HB1394_MAGNETIC: N = NO MAGNETIC MATERIAL
HB1394_STABILITY: B = PROBABLY HOLD POSITION/ELEVATION WELL
HB1394_SATELLITE: THE SITE LOCATION WAS REPORTED AS SUITABLE FOR
HB1394+SATELLITE: SATELLITE OBSERVATIONS - October 05, 2009
HB1394_ROD/PIPE-DEPTH: 4.30 meters
HB1394
HB1394  HISTORY     - Date     Condition        Report By
HB1394  HISTORY     - 1981     MONUMENTED       NGS
HB1394  HISTORY     - 19970226 GOOD             NGS
HB1394  HISTORY     - 19970626 GOOD             NGS
HB1394  HISTORY     - 19990830 GOOD             NGS
HB1394  HISTORY     - 20030724 GOOD             MODNR
HB1394  HISTORY     - 20030804 GOOD             MODNR
HB1394  HISTORY     - 20050315 GOOD             MODNR
HB1394  HISTORY     - 20091005 GOOD             MODNR
HB1394
HB1394                          STATION DESCRIPTION
HB1394
HB1394'DESCRIBED BY NATIONAL GEODETIC SURVEY 1981
HB1394'2.95 KM (1.85 MI) SOUTH FROM CHESTER.
HB1394'1.35 KILOMETERS (0.85 MILE) SOUTHWEST ALONG ILLINOIS STATE HIGHWAY 150
HB1394'AND MISSOURI STATE HIGHWAY 51 FROM THE JUNCTION REILY ROAD AND THE
HB1394'TOLL BOOTH OF THE BRIDGE IN CHESTER, THENCE 1.6 KILOMETERS (1.0 MILE)
HB1394'SOUTHEAST ALONG THE TOP OF THE MAIN LEVEE TO THE MARK ON THE RIGHT,
HB1394'4.42 METERS (14.5 FEET) SOUTHWEST OF THE CENTER OF THE LEVEE ROAD AND
HB1394'0.46 METER (1.5 FEET) SOUTHEAST OF A METAL WITNESS POST.
HB1394'THE MARK IS 0.46 METERS  NW FROM A WITNESS POST.
HB1394'THE MARK IS ABOVE LEVEL WITH TOP OF LEVEE.
HB1394
HB1394                          STATION RECOVERY (1997)
HB1394
HB1394'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1997 (CSM)
HB1394'THE STATION IS LOCATED ABOUT 24.1 KM (14.95 MI) NORTH-NORTHEAST OF
HB1394'PERRYVILLE, MO.  AND ABOUT 3.2 KM (2.00 MI) SOUTH OF AND ACROSS THE
HB1394'MISSISSIPPI RIVER FROM CHESTER, IL., ALONG THE SOUTHWEST SIDE OF THE
HB1394'RIVER, IN THE GRASS NEAR THE TOP SOUTHWEST EDGE OF THE LEVEE ROAD,
HB1394'NEAR THE NORTHWEST END OF A FIELD ACCESS TRACK ROAD ON THE RIGHT, AND
HB1394'ACROSS THE LEVEE ROAD FROM A RIVER ACCESS ROAD ON THE LEFT.
HB1394'OWNERSHIP--ARMY CORPS OF ENGINEERS.  TO REACH THE STATION FROM THE
```

```
HB1394'JUNCTION OF STATE HIGHWAY 51 AND THE LEVEE ROAD, NEAR THE SOUTHWEST
HB1394'END OF THE BRIDGE OVER THE MISSISSIPPI RIVER, SOUTHWEST OF CHESTER,
HB1394'IL., GO SOUTHEASTERLY, 1.64 KM (1.00 MI) ALONG THE LEVEE ROAD TO THE
HB1394'STATION ON THE RIGHT EDGE OF THE ROAD, JUST NORTHWEST OF A WITNESS
HB1394'POST.  THE STATION IS 12.4 M (40.7 FT) SOUTHEAST OF A CULVERT RUNNING
HB1394'UNDER THE LEVEE, WITH THE ENDS OPEN TO A DITCH ON THE SOUTHWEST SIDE
HB1394'OF THE LEVEE AND A VALVE ON THE NORTHEAST SIDE OF THE LEVEE, 3.4 M
HB1394'(11.2 FT) SOUTHWEST OF THE LEVEE CENTER, 0.4 M (1.3 FT) NORTHWEST OF A
HB1394'WITNESS POST, AND THE STATION IS ABOUT 0.3 M (1.0 FT) BELOW THE ROAD
HB1394'LEVEL AND FLUSH WITH THE GROUND SURFACE.  BY R.G.  HAYES. NOTE--THE
HB1394'DATUM POINT IS A PUNCH MARK ON THE TOP CENTER OF A STAINLESS STEEL
HB1394'DATUM POINT WHICH IS CRIMPED TO THE TOP OF A STAINLESS STEEL ROD,
HB1394'DRIVEN TO A DEPTH OF 4.3 M, (14.1 FT) ENCASED IN A 5-INCH PVC PIPE
HB1394'WITH NGS LOGO CAP, SURROUNDED BY CONCRETE.  ACCESS TO THE DATUM POINT
HB1394'IS THROUGH THE 5-INCH LOGO CAP.
HB1394
HB1394                          STATION RECOVERY (1997)
HB1394
HB1394'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1997 (CSM)
HB1394'RECOVERED AS DESCRIBED.
HB1394
HB1394                          STATION RECOVERY (1999)
HB1394
HB1394'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 1999 (RB)
HB1394'RECOVERED AS DESCRIBED
HB1394'
HB1394
HB1394                          STATION RECOVERY (2003)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2003 (WW)
HB1394'RECOVERED AS DESCRIBED.
HB1394
HB1394                          STATION RECOVERY (2003)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2003 (BDC)
HB1394'RECOVERED IN GOOD CONDITION.
HB1394
HB1394                          STATION RECOVERY (2005)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2005 (MJC)
HB1394'RECOVERED AS DESCRIBED. DESCRIPTION AND TO REACH ARE ADEQUATE.
HB1394
HB1394                          STATION RECOVERY (2009)
HB1394
HB1394'RECOVERY NOTE BY MO DEPT OF NAT RES 2009 (MJC)
HB1394'
HB1394'THE STATION IS LOCATED IN T37N R11E, IN USS 440.
HB1394'
HB1394'IT IS 11 FT. SW OF THE CENTER OF LEVEE ROAD, 92.2 FT. WNW OF THE NORTH
HB1394'I-BEAM GATE POST, 95.1 FT. NW OF THE SOUTH I-BEAM GATE POST AND 1.0
HB1394'FT. NW OF A CARSONITE WITNESS POST.
HB1394'
```

I-9. NGS Data Sheet for L 289 (PID HB1377).

```
HB1377 ***********************************************************************
HB1377  CBN          -  This is a Cooperative Base Network Control Station.
HB1377  DESIGNATION -  L 289
HB1377  PID          -  HB1377
HB1377  STATE/COUNTY-  IL/JACKSON
HB1377  USGS QUAD    -  ROCKWOOD (1994)
HB1377
HB1377                         *CURRENT SURVEY CONTROL
HB1377  ___________________________________________________________________
HB1377* NAD 83(2007)-  37 45 51.31029(N)    089 39 20.89642(W)     ADJUSTED
HB1377* NAVD 88     -       116.393  (meters)     381.87   (feet)  ADJUSTED
HB1377  ___________________________________________________________________
HB1377  EPOCH DATE  -        2002.00
HB1377  X           -      30,328.286 (meters)                     COMP
HB1377  Y           -  -5,048,474.489 (meters)                     COMP
HB1377  Z           -   3,884,844.775 (meters)                     COMP
HB1377  LAPLACE CORR-          -0.37  (seconds)                    USDV2009
HB1377  ELLIP HEIGHT-          87.252 (meters)        (02/10/07) ADJUSTED
HB1377  GEOID HEIGHT-         -29.16  (meters)                     GEOID09
HB1377  DYNAMIC HT  -         116.313 (meters)     381.60  (feet)  COMP
HB1377
HB1377  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
HB1377  Type    PID    Designation                      North   East  Ellip
HB1377  -------------------------------------------------------------------
HB1377  NETWORK HB1377 L 289                             1.20   0.73   1.98
HB1377  -------------------------------------------------------------------
HB1377  MODELED GRAV-     979,933.4   (mgal)                       NAVD 88
HB1377
HB1377  VERT ORDER  -  FIRST     CLASS II
HB1377
HB1377.The horizontal coordinates were established by GPS observations
HB1377.and adjusted by the National Geodetic Survey in February 2007.
HB1377
HB1377.The datum tag of NAD 83(2007) is equivalent to NAD 83(NSRS2007).
HB1377.See National Readjustment for more information.
HB1377.The horizontal coordinates are valid at the epoch date displayed above.
HB1377.The epoch date for horizontal control is a decimal equivalence
HB1377.of Year/Month/Day.
HB1377
HB1377.The orthometric height was determined by differential leveling
HB1377.and adjusted in June 1991.
HB1377
HB1377.The X, Y, and Z were computed from the position and the ellipsoidal ht.
HB1377
HB1377.The Laplace correction was computed from USDV2009 derived deflections.
HB1377
HB1377.The ellipsoidal height was determined by GPS observations
HB1377.and is referenced to NAD 83.
HB1377
HB1377.The geoid height was determined by GEOID09.
HB1377
HB1377.The dynamic height is computed by dividing the NAVD 88
HB1377.geopotential number by the normal gravity value computed on the
HB1377.Geodetic Reference System of 1980 (GRS 80) ellipsoid at 45
HB1377.degrees latitude (g = 980.6199 gals.).
HB1377
HB1377.The modeled gravity was interpolated from observed gravity values.
HB1377
HB1377;                    North         East     Units Scale Factor Converg.
HB1377;SPC IL W     -   121,927.701   745,011.038   MT  0.99996612   +0 18 46.3
```

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HB1377;SPC IL W     -   400,024.47  2,444,257.05   sFT  0.99996612   +0 18 46.3
HB1377;UTM  16      - 4,182,980.653   266,061.861   MT  1.00027412   -1 37 37.9
HB1377
HB1377!             -  Elev Factor  x  Scale Factor =   Combined Factor
HB1377!SPC IL W     -   0.99998631  x   0.99996612  =   0.99995243
HB1377!UTM  16      -   0.99998631  x   1.00027412  =   1.00026042
HB1377
HB1377                          SUPERSEDED SURVEY CONTROL
HB1377
HB1377  NAD 83(1997)-  37 45 51.31026(N)    089 39 20.89637(W) AD(       ) A
HB1377  ELLIP H (09/15/03)   87.253  (m)                       GP(       ) 4 1
HB1377  NGVD 29 (??/??/??)  116.316  (m)          381.61   (f) ADJUSTED    1 2
HB1377
HB1377.Superseded values are not recommended for survey control.
HB1377.NGS no longer adjusts projects to the NAD 27 or NGVD 29 datums.
HB1377.See file dsdata.txt to determine how the superseded data were derived.
HB1377
HB1377_U.S. NATIONAL GRID SPATIAL ADDRESS: 16SBG6606182980(NAD 83)
HB1377_MARKER: I = METAL ROD
HB1377_SETTING: 59 = STAINLESS STEEL ROD IN SLEEVE (10 FT.+)
HB1377_SP_SET: STAINLESS STEEL ROD IN SLEEVE
HB1377_STAMPING: L 289 1981
HB1377_MARK LOGO: NGS
HB1377_PROJECTION: FLUSH
HB1377_MAGNETIC: N = NO MAGNETIC MATERIAL
HB1377_STABILITY: A = MOST RELIABLE AND EXPECTED TO HOLD
HB1377+STABILITY: POSITION/ELEVATION WELL
HB1377_SATELLITE: THE SITE LOCATION WAS REPORTED AS SUITABLE FOR
HB1377+SATELLITE: SATELLITE OBSERVATIONS - March 08, 2006
HB1377_ROD/PIPE-DEPTH: 21.9 meters
HB1377_SLEEVE-DEPTH  : 6.10 meters
HB1377
HB1377  HISTORY     - Date     Condition        Report By
HB1377  HISTORY     - 1981     MONUMENTED       NGS
HB1377  HISTORY     - 20020805 GOOD             NGS
HB1377  HISTORY     - 20060308 GOOD             ILDT
HB1377
HB1377                          STATION DESCRIPTION
HB1377
HB1377'DESCRIBED BY NATIONAL GEODETIC SURVEY 1981
HB1377'8.1 KM (5.05 MI) SOUTH FROM CORA.
HB1377'8.1 KILOMETERS (5.05 MILES) SOUTH ALONG THE TOP OF THE MAIN LEVEE FROM
HB1377'THE INTERSECTION OF THE MISSOURI PACIFIC RAILROAD IN CORA TO A BEND IN
HB1377'THE LEVEE AND THE MARK ON THE LEFT, AT THE JUNCTION OF A SPUR LEVEE
HB1377'LEADING SOUTHEAST, IN LINE WITH THE CENTER OF THE SPUR LEVEE,
HB1377'4.57 METERS (15.0 FEET) NORTH OF THE CENTER OF THE LEVEE ROAD AND
HB1377'0.46 METERS (1.5 FEET) EAST OF A METAL WITNESS POST.
HB1377'THE MARK IS 0.46 METERS  W FROM A WITNESS POST.
HB1377'THE MARK IS 0.15 M BELOW TOP OF LEVEE.
HB1377
HB1377                          STATION RECOVERY (2002)
HB1377
HB1377'RECOVERY NOTE BY NATIONAL GEODETIC SURVEY 2002 (BE)
HB1377'RECOVERED AS DESCRIBED
HB1377'
HB1377'
HB1377
HB1377                          STATION RECOVERY (2006)
HB1377
HB1377'RECOVERY NOTE BY ILLINOIS DE
```

APPENDIX J

Establishing NSRS Elevations on 15 Dam and Reservoir Projects in Pittsburgh District

J-1. General. The following example outlines the survey actions performed by Pittsburgh District to establish NAVD88 elevations on 15 dam and reservoir projects throughout the District. The procedures used in this example project are also applicable to levee projects extending over large geographical areas. This work outlined below was performed by Terrasurv, Inc. under a contract to the Pittsburgh District. Adjusted GPS observations, from which NAVD 88 elevations were derived, were submitted to NGS for inclusion in the in the NSRS. NSRS referenced NAVD88 elevations were established at Primary Project Control Points (PPCPs) at each dam and reservoir site.

J-2. Background. As part of a Corps-wide review of project datums initiated in 2007 (i.e., the Comprehensive Evaluation of Project Datums or CEPD), the Pittsburgh District undertook an assessment of all their projects. Part of this effort was to evaluate the origin and accuracy of the vertical datums in use at each of the sixteen dam and reservoir projects within the District's civil works area of responsibility. This CEPD review determined that none of these projects had a documented connection to the NSRS. While all of the projects nominally had NGVD29 elevations, the source and accuracy of many was in question. In addition, the data was not in a format conducive to updating to NAVD88.

a. East Branch Project. The Clarion River East Branch Reservoir was used as a pilot project. GPS survey methods were used on the East Branch project to provide ties to NSRS bench marks. The data was then formatted and submitted to the NGS for inclusion in the NSRS. Additional details on this specific project are outlined in Appendix F.

b. Remaining reservoir projects. After completing the East Branch Project, the District decided to implement a similar process to establish an NSRS referenced PPCP at each of the remaining 15 reservoir projects. These projects encompassed an area of approximately 26,000 square miles in western Pennsylvania, a small portion of western New York, eastern Ohio, and northern West Virginia.

J-3. Project Description. Figure J-1 depicts the 16 dam and reservoir projects contained in Pittsburg District's civil works boundary. The geodetic survey scheme shown on the right side of the figure was developed to establish vertical (and horizontal) control on an existing primary monitoring bench mark at each of the remaining 15 project sites. These geodetic control surveys were performed during annual O&M programmed deformation monitoring surveys at each dam—significantly reducing costs in updating datum references. (Pittsburgh District estimates the cost to perform the NSRS connection at each site was approximately $8,000). The observed GPS baseline data was then processed, adjusted, and submitted to NGS for inclusion in the NSRS. The District's 23 lock and dam projects shown on Figure J-1 were not included in this overall project. These projects could be linked to this reservoir control network at some future date.

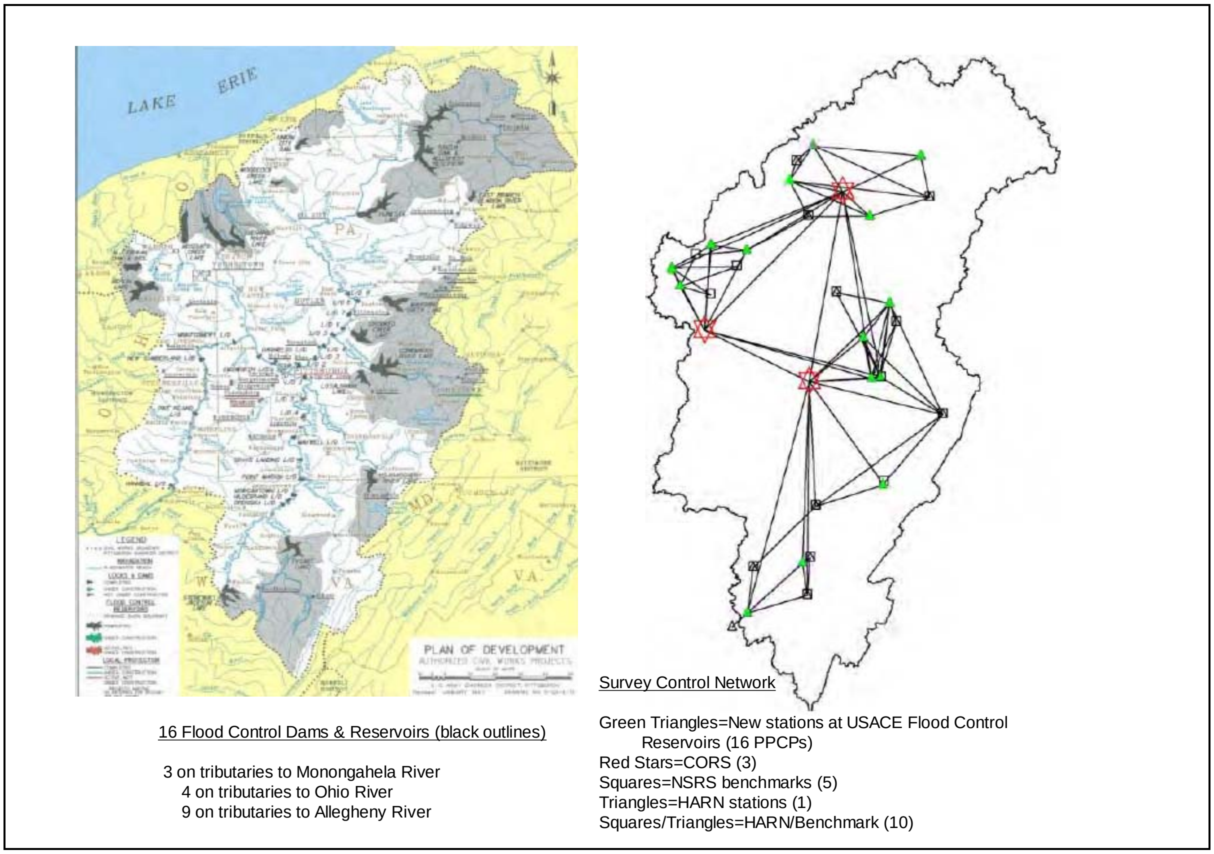


Figure J-1. GPS data collection network used to establish primary control at 16 dam and reservoir projects in Pittsburg District.

J-4. GPS Network Decision. The A-E contractor (Terrasurv, Inc.) had been performing annual deformation surveys at the reservoirs since 2005. Many of these prior surveys utilized GPS to provide external control for the deformation analysis. Although CORS/OPUS observations at each site would have provided adequate NAVD88 elevations (accurate to < ± 0.25 ft), it was decided that previously observed GPS baselines could be readily incorporated into a higher-accuracy geodetic network encompassing all projects. This network would effectively link all the District's projects to the NSRS and to one another. Accordingly, GPS observations from 2005, 2006, and 2007 were examined to extract observations that would be useful in the current network survey. A "Primary Project Control Point" (PPCP) was selected at each reservoir—typically one of the structural deformation monitoring points (pedestals). Some reservoirs had a second point included in the network due to multiple GPS observation in previous years.

J-5. Existing NSRS Control. Because the primary purpose of the project was to establish NAVD88 elevations at each reservoir, emphasis was placed on using NSRS marks that were stable bench marks. Because there were numerous ties to CORS, horizontal accuracy of the existing NSRS stations was secondary. Most of the existing NSRS control stations used in Pennsylvania and West Virginia were both horizontal and vertical control. In Ohio, it was not possible to find any such stations in the vicinity of the reservoirs, therefore three vertical-only bench marks were used there, along with a local CORS. Table J-1 lists the existing NSRS control used on this project.

Table J-1. Existing NSRS Control.

Station Name	PID	Horizontal Ellipsoidal Accuracy	Ortho Order	State	Comments	Stability	Nearest Reservoir
R HAYES	AA9347	0.65/1.04	GPS	WV	HARN	D	Stonewall
C 317	JW1099	1.22/2.12	I-2	WV	HARN/BM	A	Tygart
P 322	JW1091	1.04/1.67	I-2	WV	HARN/BM	A	Tygart/Stonewall
W 319	JX1767	1.18/1.78	I-2	WV	HARN/BM	B	Stonewall
A 121	JW0568	----	I-2	PA	V only	D	Youghiogheny
T 404	KX1902	1.30/2.27	I-2	PA	HARN/BM	B	Mahoning
E 402	KX1814	0.77/1.33	I-2	PA	HARN/BM	B	Mahoning/Crooked Creek
G 316	KX0579	0.71/1.35	II-0	PA	HARN/BM	D	Youghiogheny
E 313	JW1043	0.61/1.16	I-2	WV	HARN/BM	B	Youghiogheny/Tygart
TTS 64 K	MA0735	0.31/0.51	II-0	PA	HARN/BM	A	East Branch/Kinzua
D 406	MB1777	0.86/1.53	I-2	PA	HARN/BM	B	Woodcock/Union Cty
E 408	MA1819	0.77/1.39	I-2	PA	HARN/BM	B	Tionesta
D 156	MB0852	----	2-0	OH	V only	B	Mosquito/Kirwan
R 147	KY1127	----	2-0	OH	V only	B	Berlin
AAA	MB0984	----	2-0	OH	V only	B	Shenango
PAPT	DF7986	CORS	CORS	PA	CORS		
UPTC	AI8355	CORS	CORS	PA	CORS		
LSBN	DF4054	CORS	CORS	OH	CORS		

J-6. Conemaugh Dam Bench Marks. In addition to the existing NSRS marks described above, two stable NSRS bench marks (PID KX1047 and PID KX1045) were recovered near the Conemaugh Dam, and a short level line was run between them to establish a new mark which was capable of being occupied by GPS receivers. This level line was run twice with invar bar code rods, once in 2005 and again in 2008. The misclosure between these marks relative to the published elevations was 0.0065 m. PBM 52A appeared to have been disturbed.

J-7. Datums. The horizontal datum used for this network project is the NAD83 (NSRS 2007). The vertical datum is NAVD88. The geoid model used was GEOID 2003.

J-8. GPS Observations. Observations were made over a time span from May of 2005 through January of 2009. All observations were made using Trimble dual frequency receivers.

a. The receivers used included the following:

(1) two R8 GNSS models, with integral antennas.

(2) one R6 model, with integral antenna.

(3) two 5700 models, one with a Zephyr Geodetic antenna and the other with a Zephyr antenna.

(4) one 4800 model, with integral antenna.

(5) one 4700 model, with a microcenter L1/L2 antenna without ground plane.

(6) one 4400 model with a compact L1/L2 antenna, without ground plane.

b. Data was obtained from the CORS sessions with a sufficient amount of data to ensure an accurate solution. The survey pedestals were occupied by placing a standard survey tribrach onto the stainless steel plate. The 5/8" rod typically protrudes about 12 mm above the plate. The height of instrument measurement is with respect to the top of this rod. Most of the ground points (disks) were occupied using fixed height tripods as shown in Figure J-2. A few were occupied with standard survey tripods. All height of instrument measurements were reduced to vertical values from the mark to the Antenna Reference Point (ARP), which is the bottom of the antenna housing.

Figure J-2. GPS data collection at a primary structural monitoring point (PPCP) near the dam.

J-9. GPS Data Processing. The GPS observables were downloaded from the receivers and processed using the Weighted Ambiguity Vector Estimator (WAVE) processor in Trimble Geomatics Office, version 1.63. The precise ephemeris (IGS Rapid) was used for processing baselines. Not all baselines were processed—several of the lines had higher than normal statistics, indicating the presence of noise in the data. Many of the baselines were measured twice, and therefore had an independent verification of correct integer ambiguity resolution. Seven processed baselines were disabled in the data analysis phase. Many of the baselines were measured in more than one session. The independently determined baseline components were transformed from an Earth Centered Earth Fixed (ECEF) system to a local horizon system (N-E-U). All of the new reservoir project stations were occupied at least once. Two existing NSRS control stations, “R HAYES” and “D 406,” were occupied a single time.

J-10. Least Squares Adjustments. The data was adjusted using ADJUST, a least squares adjustment program from the NGS. The occupation information was processed to form an ADJUST “B-file.” The processed baselines were parsed to form an input file in the ADJUST “G-file” format.

a. Minimally constrained NSRS adjustment. The first adjustment constrained the CORS “PAPT ARP” station to the published NAD83 (epoch 2002.0) position (latitude, longitude, and ellipsoidal height). The resultant standard deviation of unit weight was 6.75. The misclosures at the other NSRS stations and the other two CORS used are shown in Table J-2.

Table J-2. Minimally Constrained Adjustment of NSRS Control Holding “PAPT ARP” Fixed.

Station	Azimuth	Distance (m)	Δ Ellipsoidal Height (m)
R HAYES	196	0.007	-0.010
C 317	315	0.003	-0.012
P 322	188	0.008	-0.031
W 319	214	0.004	0.009
T 404	116	0.013	0.018
E 402	242	0.008	-0.010
G 316	200	0.007	-0.011
E 313	31	0.002	0.028
TTS 64 K	255	0.016	-0.002
D 406	238	0.009	0.005
E 408	251	0.007	0.012
UPTC ARP	129	0.005	-0.007
LSBN ARP	272	0.013	0.002

The misclosures on the above NSRS control stations were considered excellent, especially given the network spans an area of about 65,000 square kilometers (24,000 square miles).

b. Minimally Constrained Orthometric Height Adjustment. Next, a minimally constrained adjustment was done holding the published NAVD88 orthometric height of bench mark “T 404” (PID KX1902), and “PAPT ARP” fixed horizontally. The misclosures at the other NSRS marks with published orthometric heights are shown in Table J-3 and graphically on Figure J-3.

Table J-3. Minimally Constrained Adjustment Holding “T 404” (V) and “PAPT ARP” (3D).

Station	Δ Ortho Height
R HAYES (GPS Derived Ortho Height)	0.010 m
CONEMAUGH BRIDGE (leveling by Terrasurv)	-0.022 m
C 317	-0.007 m
P 322	0.000 m
W 319	0.030 m
A 121 (checked by levels to adjacent BM)	0.066 m
E 402	-0.008 m
G 316	-0.030 m
E 313	0.038 m
TTS 64 K	-0.006 m
D 406	-0.001 m
E 408	0.015 m
D 156	-0.015 m
R 147	0.037 m
AAA	0.027 m

c. The bench mark with the highest misclosure, “A 121” (0.066 m), was checked against an adjacent NSRS mark using precise leveling techniques and equipment (Second Order procedures). This mark had four repeat baselines, two in 2005 and two in 2008. The vertical component of the baseline from YOUGHIOGHENY M1 to A 121 was - 49.818 m, - 49.806 m, - 49.815 m, and - 49.800 m, with a range of ± 0.010 m about the mean value of - 49.810 m. Therefore, this mark is believed to be stable and the lines going to it are of high accuracy. This mark is located on the eastern side of two mountain ridges (Chestnut and Laurel Ridges), whereas the rest of the project is located on the western side of the ridges. The geoid model used (2003) may be affected by these ridges. In order to ensure an accurate GPS derived orthometric height at the nearby Youghiogheny Reservoir, this mark was included as a constraint in the final constrained orthometric height adjustment. All of the other existing marks were within ± 0.037 m in the vertical (orthometric height) component.

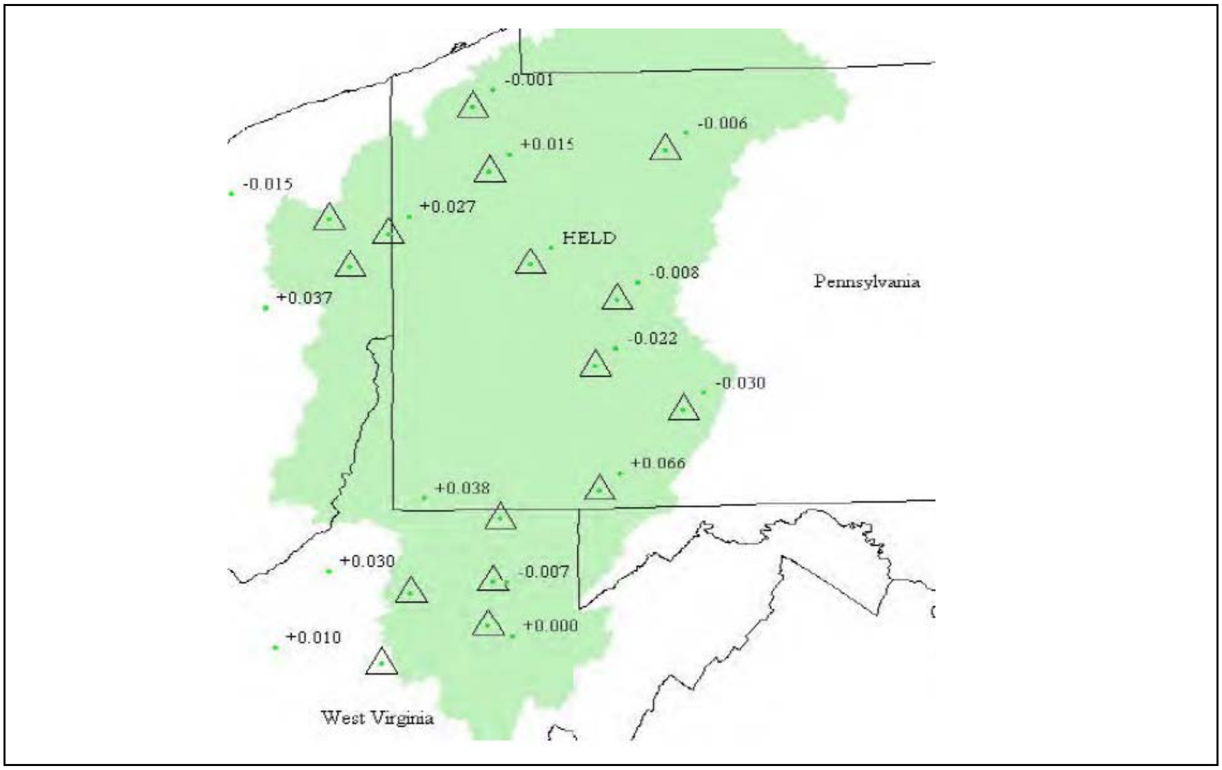


Figure J-3. Misclosures resulting from a minimally constrained vertical adjustment constraining height on a HARN bench mark near the center of the project. (Misclosures shown in meters)

d. Constrained adjustment (ellipsoid heights). The next adjustment was a fully constrained adjustment that held the three CORS and the eleven existing NSRS stations with published NAD1983 (NSRS2007) latitudes, longitude, and ellipsoidal heights. This adjustment had a standard deviation of unit weight of 8.165. This adjustment provided the adjusted latitudes, longitudes, and ellipsoidal heights for the network.

e. Constrained adjustment (orthometric heights). The final constrained adjustment held CORS "PAPT ARP" fixed horizontally, and fifteen stations with leveled NAVD88 orthometric heights were constrained vertically in the NGS ADJUST program. The standard deviation of unit weight was 7.20 and the residual RMS was ± 0.021 m. This adjustment provided the NAVD88 GPS derived orthometric heights for the network. The final adjustment results are listed in Table J-4.

Table J-4. Adjusted Coordinates – NAD83 NSRS 2007/NAVD88.

Station Name	Latitude	Longitude	Ellipsoid Height (m)	NAVD88 Height (m)
TYGART ASE2	39º18'47.16840" N	80º01'45.99898" W	345.026	377.139
CROOKED CK M2	40º42'53.07204" N	79º30'49.51338" W	257.176	290.573
MOSQUITO M4	41º18'06.84178" N	80º45'09.48816" W	246.597	280.407
STONEWALL	39º00'15.54934" N	80º28'24.35241" W	283.918	316.238
R HAYES	38º54'47.26507" N	80º35'44.73360" W	336.208	368.608
YOUGHIOGHENY M1	39º47'55.96185" N	79º22'02.18435" W	424.933	456.671
CONEMAUGH BR	40º27'42.15164" N	79º22'02.71448" W	245.331	278.377
CONEMAUGH M3	40º28'05.77660" N	79º22'29.30277" W	286.840	319.890
LOYALHANNA	40º27'29.96032" N	79º27'09.34669" W	269.118	302.254
TIONESTA	41º28'24.00763" N	79º26'35.72847" W	322.841	355.812
UNION CITY M1	41º55'14.71075" N	79º54'11.33916" W	362.504	396.550
UNION CITY M5	41º55'14.56028" N	79º54'16.09282" W	371.722	405.770
KIRWAN M1	41º08'47.32372" N	81º04'22.71305" W	275.309	308.988
C 317	39º20'38.54354" N	79º58'03.73837" W	325.837	357.872
BERLIN M5	41º02'35.39945" N	81º00'32.70465" W	285.874	319.559
P 322	39º06'38.37991" N	79º59'42.18154" W	525.250	556.874
W 319	39º17'11.27553" N	80º25'31.19084" W	332.512	365.178
A 121	39º48'45.98850" N	79º21'33.77400" W	375.121	406.837
MAHONING	40º55'28.22823" N	79º16'58.28579" W	356.660	389.675
KINZUA M1	41º50'31.50495" N	79º00'02.33348" W	387.903	420.339
WOODCOCK	41º42'05.73148" N	80º06'07.07176" W	339.158	372.986
SHENANGO	41º15'54.80683" N	80º27'45.43598" W	250.654	284.444
KIRWAN	41º09'26.19271" N	81º04'41.85584" W	253.577	287.257
MOSQUITO	41º17'59.95573" N	80º45'30.50304" W	245.336	279.148
T 404	40º59'41.81625" N	79º43'32.90556" W	336.103	369.602
E 402	40º48'07.37960" N	79º14'00.67131" W	338.584	371.572
G 316	40º13'30.90049" N	78º52'17.31696" W	554.614	586.700
E 313	39º40'10.80731" N	79º55'09.32013" W	345.395	377.956
TTS 64 K	41º34'50.57939" N	78º56'06.94800" W	541.539	573.625
D 406	41º49'17.00008" N	80º02'40.75526" W	314.420	348.439
E 408	41º28'42.27988" N	79º57'06.99473" W	356.203	389.618
D 156	41º14'17.11949" N	80º52'54.65838" W	239.435	273.263
R 147	40º59'17.67806" N	80º45'42.03236" W	310.755	344.636
AAA	41º09'49.81358" N	80º32'32.18072" W	247.341	281.222
PAPT ARP	40º26'40.25568" N	79º57'32.12989" W	313.698	347.441
UPTC ARP	41º37'43.70199" N	79º39'50.62173" W	343.186	376.455
LSBN ARP	40º46'08.93523" N	80º48'37.47136" W	314.449	348.434

J-11. Supplemental Ties to LPCPs. At each reservoir project, supplemental ties were made to local project control points around the project. In addition, ties were made to any water level or pool gages at the sites. Differential levels were run from primary PPCP mark to secondary marks using a DiNi 12 digital level. Levels runs to outflow gage reference PBMs were run if feasible, otherwise GPS was used to transfer elevations from the primary PPCP station to the outflow bench mark. Figures J-4 illustrates connections from the primary PPCP to supplemental bench marks at a dam and the down stream outflow PBM reference point. Figure J-5 shows leveling connections to a typical reservoir pool gage.

J-12. Summary. The geodetic control network met NGS height modernization standards and established consistent NAVD 1988 vertical control for the reservoirs in the Pittsburgh District. An overall accuracy of ±0.03 m was achieved in all three dimensions in a network covering the entire 26,000 sq mile district.

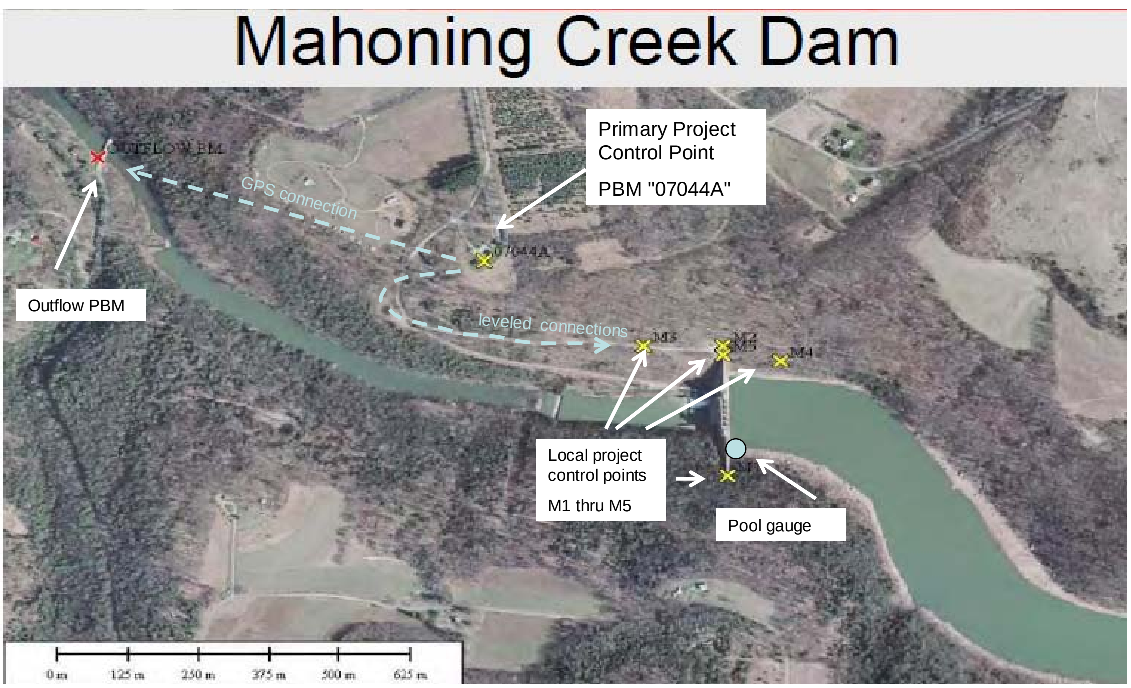


Figure J-4. Differential level connections from the PPCP to supplemental PBMs around Mahoning Creek Dam. The outfall PBM was connected by static GPS from the PPCP.

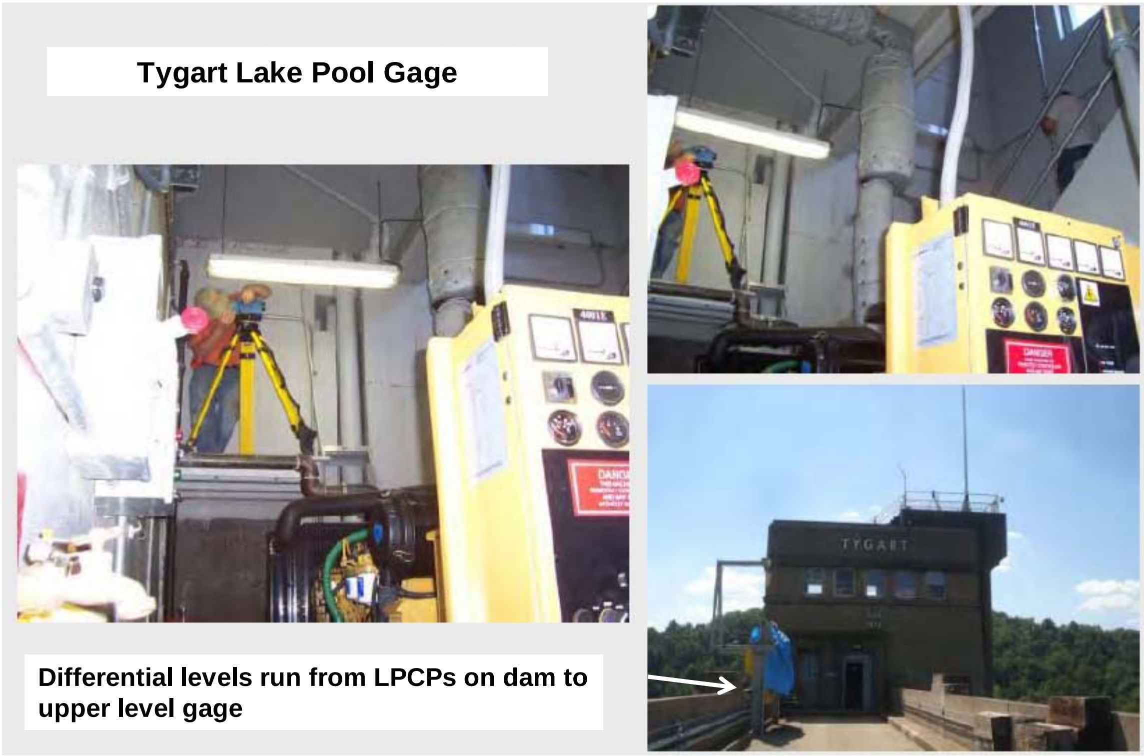


Figure J-5. Differential leveling connections to the pool gage at Tygart Lake Dam and Reservoir.

APPENDIX K

Lake Superior Navigation Project Referenced to IGLD85—Ontonagon Harbor, Michigan (Detroit District)

K-1. Purpose. This appendix contains an example of a Detroit District harbor project in Lake Superior (Figure K-1) that has been adequately referenced to the current NSRS orthometric datum and to the local reference water level datum in Lake Superior (IGLD85). The project consists of two breakwaters that have established reference baselines with local control on an IGLD85 vertical datum that is tied to a “Dynamic Height” reference plane used in the Great Lakes region—see Figure K-2. NOAA water level gages and bench marks in the Great Lakes are referenced to both orthometric heights (NAVD88) and dynamic heights (IGLD85). An overview on the establishment of IGLD85 and a list of IGLD reference datums throughout the Great Lakes and Connecting Waterways is at the end of this appendix.

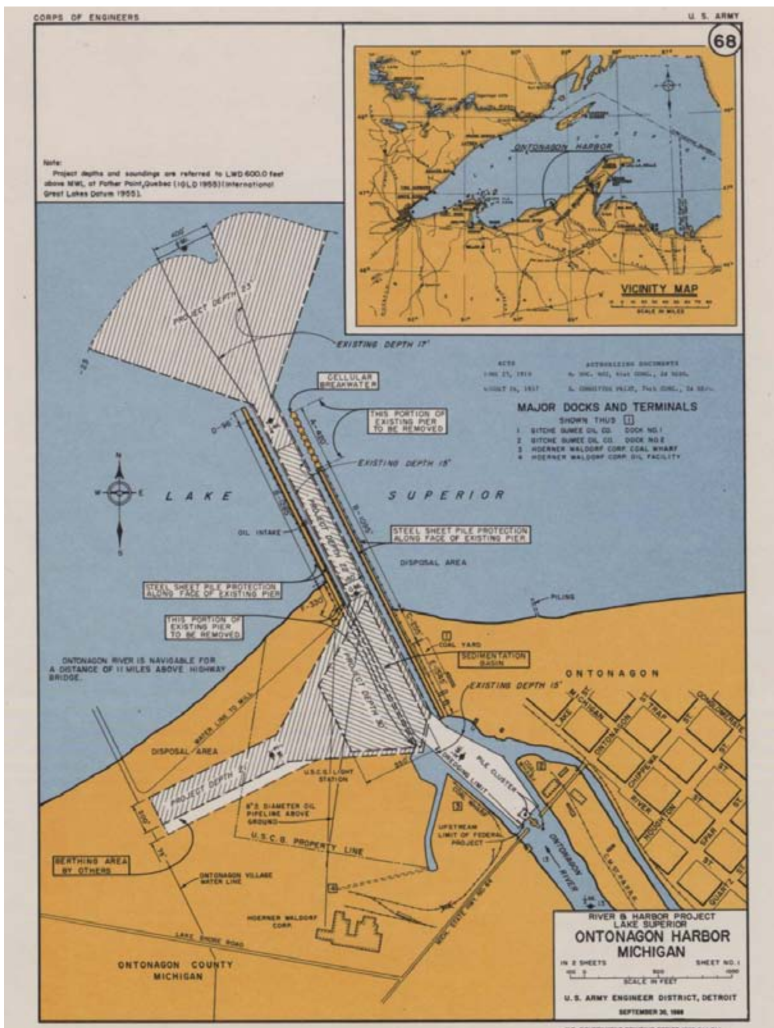


Figure K-1. Ontonagon Harbor, Michigan – Project Map.

K-2. Project Description.

ONTONAGON HARBOR, MICHIGAN

CONDITION OF IMPROVEMENT 30 SEPTEMBER 1986

EXISTING PROJECT: Authorized by the R&H Acts of 2 March 1867, 23 June 1874, 13 June 1902, 2 March 1907, 3 March 1909, 26 August 1937 and Act of 1962. Earlier authorizations (1910 and 1937) provide for a flared lake approach channel about 850 feet long to deep water and 16 feet deep with depths narrowing from 400 feet at the outer end to 150 feet opposite the outer end of the west pier; a channel between the piers 150 feet wide, 17 feet deep in the outer 250 feet, and 15 feet deep in the inner 2,200 feet; an inner basin 12 feet deep and 900 feet long, extending between lines 50 feet from the existing wharves on each side of the river, the maximum width being 200 feet; and the maintenance of this channel, the basin, and the east and west entrance piers which are 2,315 feet and 2,563 feet long, respectively.

Figure K-2. Ontonagon Harbor Michigan – Structure and federal navigation channel.

K-3. Connections to NSRS and Tidal Datum References. The Tidal BM established for the project is BM "D 135" as shown on Figure K-3. This point is published in the NSRS and has observed NAD83/GRS80 ellipsoid height observations, as excerpted from the NGS datasheet in Figure K-3.

```
RL0728  CBN          -  This is a Cooperative Base Network Control Station.
RL0728  DESIGNATION -  D 135
RL0728  PID          -  RL0728
RL0728  STATE/COUNTY-  MI/ONTONAGON
RL0728  USGS QUAD    -
RL0728
RL0728                          *CURRENT SURVEY CONTROL
RL0728  ___________________________________________________________________
RL0728* NAD 83(2007)-  46 52 20.23258(N)    089 19 19.44724(W)     ADJUSTED
RL0728* NAVD 88      -       186.877  (meters)     613.11   (feet)  ADJUSTED
RL0728  ___________________________________________________________________
RL0728  EPOCH DATE   -        2002.00
RL0728  X            -      51,683.527 (meters)                     COMP
RL0728  Y            -  -4,367,861.008 (meters)                     COMP
RL0728  Z            -   4,632,183.750 (meters)                     COMP
RL0728  LAPLACE CORR-           -4.82  (seconds)                    USDV2009
RL0728  ELLIP HEIGHT-          155.401 (meters)        (02/10/07) ADJUSTED
RL0728  GEOID HEIGHT-          -31.44  (meters)                     GEOID09
RL0728  DYNAMIC HT   -         186.907 (meters)     613.21  (feet)  COMP
RL0728
RL0728  ------- Accuracy Estimates (at 95% Confidence Level in cm) --------
RL0728  Type    PID    Designation                      North   East  Ellip
RL0728  -------------------------------------------------------------------
RL0728  NETWORK RL0728 D 135                             1.55   1.12   5.21
RL0728  -------------------------------------------------------------------
RL0728  MODELED GRAV-     980,770.6   (mgal)                        NAVD 88
RL0728
RL0728  VERT ORDER   -  FIRST     CLASS II
```

Figure K-3. Ontonagon Harbor Michigan – Tidal NGS BM D 135 Datasheet.

The 613.11 ft NAVD88 elevation of D135 is based on adjusted leveling observations. The estimated 95% confidence of the ellipsoid height is about 5 cm. Thus, this is an excellent point for use as an NOS gaging station from which all supplemental surveys can be referenced.

a. Local PBM and TBM control. Figure K-4 lists the local reference PBMs and TBMs that are used to control structure cross-section monitoring. All USACE monuments are run in level differential levels loops to the two known tidal bench marks. All elevation measurements were relative to BM"D 135" on the NAVD88 reference datum. All reported elevations on design surveys and channel depth surveys are referenced to IGLD85 datum.

b. IGLD85. IGLD85 is expressed as a dynamic height. Informally, this could also be considered as a height equivalent above mean sea level, based on work required to raise a unit mass. IGLD85 is also based on an adopted elevation at Point Rimouski/Father's Point. And, IGLD85 is realized as mean water levels at a set of master water level stations on the Great

Lakes. Due to various observational, dynamical, and satiric effects, there will be slight departures between a dynamic height and an IGLD85 height. These departures are known as hydraulic correctors, and are part of the NAVD88/IGLD85 datum transformation.

c. Conversion from NAVD88 Dynamic Height to IGLD85. The survey specifications required that structure profile data and all topographic be referenced on IGLD85, reported as true elevations. However soundings are reported as negative numbers in relationship to the datum of IGLD85 601.1 ft for Lake Superior. Thus, a (-) 26.5 ft depth reported on a condition survey is equal to an IGLD85 elevation of (601.1 - 26.5) = 574.6 ft.

(1) The water surfaces of all connecting channels and other rivers on the Great Lakes are considered to be sloping surfaces. Therefore, their Hydraulic Corrector is zero.

(2) The "Hydraulic Corrector" at each gage site on the lake has been incorporated into the data retrieval and storage process. As such, water level information is stored mechanically or electronically, at the NOAA CO-OPS or the Canadian Department of Fisheries and Oceans (DFO). Water elevations are referenced to IGLD85 and do not require any further adjustment.

(3) Hydraulic Correctors for several harbors in Lake Superior are listed below in Table K-1. IGLD85 vertical datum is based on calculated and interpolated corrections to be applied to Dynamic Heights for the Great Lakes region.

Table K-1. Lake Superior Hydraulic Correctors for IGLD85 (in meters).

PROJECT	LOCATION	HC	IHC
Duluth-Superior MN-WI	Lake Superior	0.3	--
Ontonagon Harbor MI		0.2	--
Grand Travis Bay MI		--	0.1
Saxon Harbor WI		--	0.2

IHC = Interpolated from established Hydraulic Correctors
HC = Hydraulic Correctors from “Establishment of International Great Lakes Datum” December 1995

INTERNATIONAL GREAT LAKES DATUM (1985)
Tabulation of Primary Bench Mark,
ONTONAGON

Primary PBM NO 2		Hydraulic Corrector	0.049 m
IGLD85 elev 185.443 m		Diff IGLD85-IGLD55	0.323 m

d. NOAA gage Ontonagon. The NOAA CO-OPS station data for the Ontonagon gage is shown in Figures K-4 and K-5.

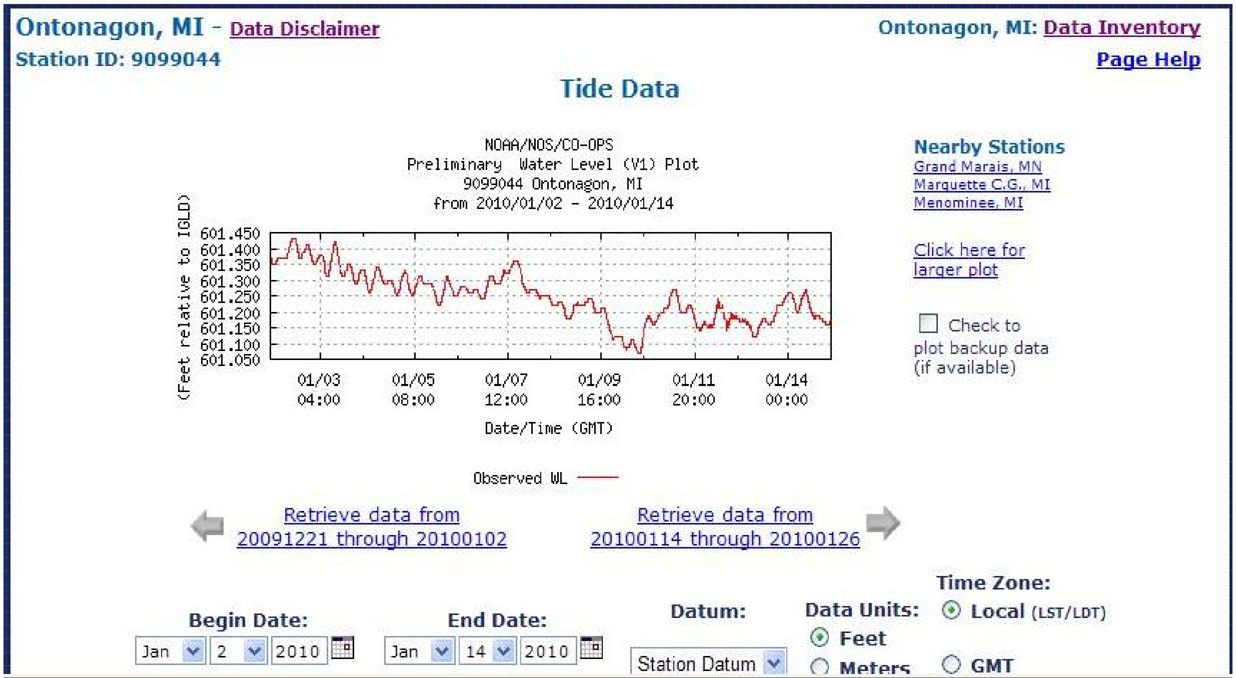
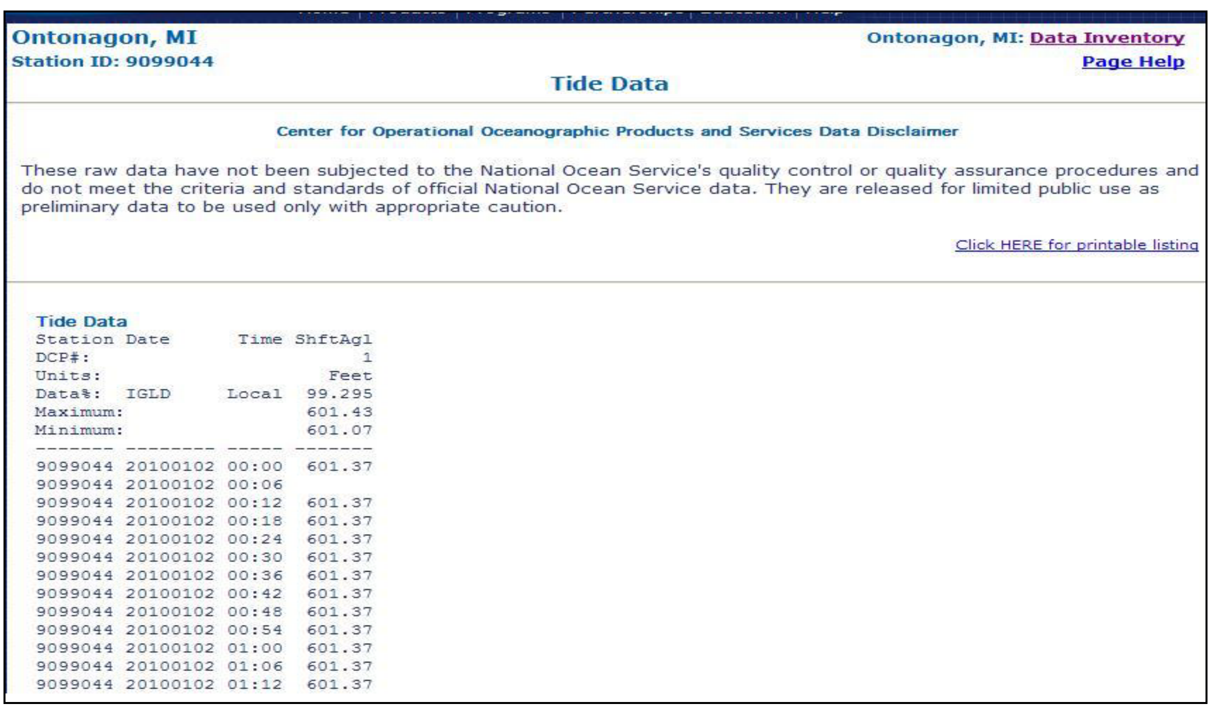


Figure K-4. Published NOAA/CO-OPS gage data in IGLD85 per CO-OPS web site.



Ontonagon, MI

Station ID: 9099044

Ontonagon, MI: Data Inventory

Page Help

Tide Data

Center for Operational Oceanographic Products and Services Data Disclaimer

These raw data have not been subjected to the National Ocean Service's quality control or quality assurance procedures and do not meet the criteria and standards of official National Ocean Service data. They are released for limited public use as preliminary data to be used only with appropriate caution.

Click HERE for printable listing

Tide Data

```
Station Date       Time ShftAgl
DCP#:                         1
Units:                     Feet
Data%:  IGLD      Local  99.295
Maximum:                 601.43
Minimum:                 601.07
------- -------- ----- -------
9099044 20100102 00:00  601.37
9099044 20100102 00:06
9099044 20100102 00:12  601.37
9099044 20100102 00:18  601.37
9099044 20100102 00:24  601.37
9099044 20100102 00:30  601.37
9099044 20100102 00:36  601.37
9099044 20100102 00:42  601.37
9099044 20100102 00:48  601.37
9099044 20100102 00:54  601.37
9099044 20100102 01:00  601.37
9099044 20100102 01:06  601.37
9099044 20100102 01:12  601.37
```

Figure K-5. Published NOAA/CO-OPS gage data in IGLD85 per WEB site in IGLD85 in Local Standard Time (LST) in six-minute intervals.

e. Sounding corrections. Based on the gage data in Figure K-5 for the date shown (2 Jan 10), the water level at Ontonagon is 0.27 ft above IGLD85 (601.1 ft). To correct all sounding data observed on 2 Jan 10 to the IGLD85 depth, one would subtract 0.27 ft from all observed soundings. The difference between NAVD88 and IGLD85 is assumed the same throughout this small project site. Figure K-6 illustrates survey depths referenced to IGLD85 on a Project Condition Survey of Ontonagon Harbor. Figures K-7a and K-7b are examples of design placement grades on the IGLD85 reference datum.

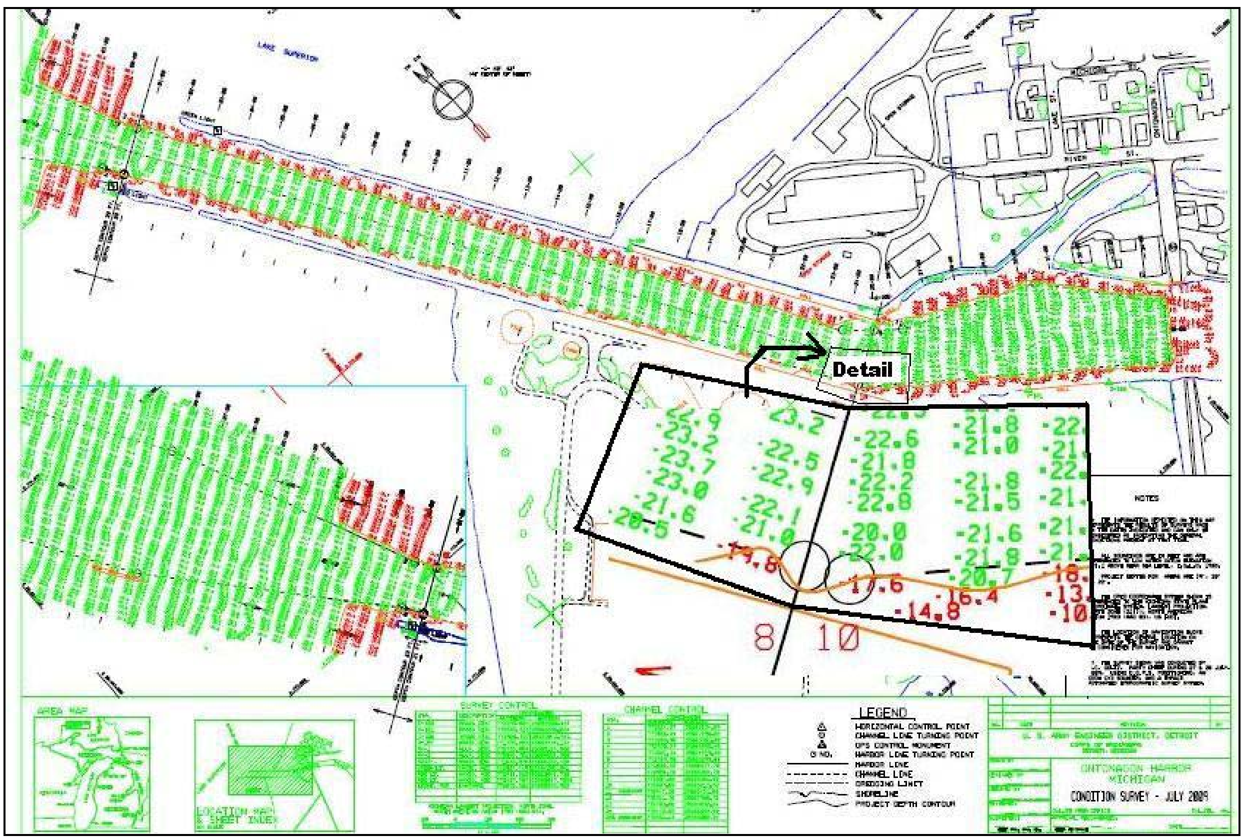


Figure K-6. Hydrographic condition survey of federal navigation channel with soundings referenced to IGLD85 (601.1 ft) Datum.

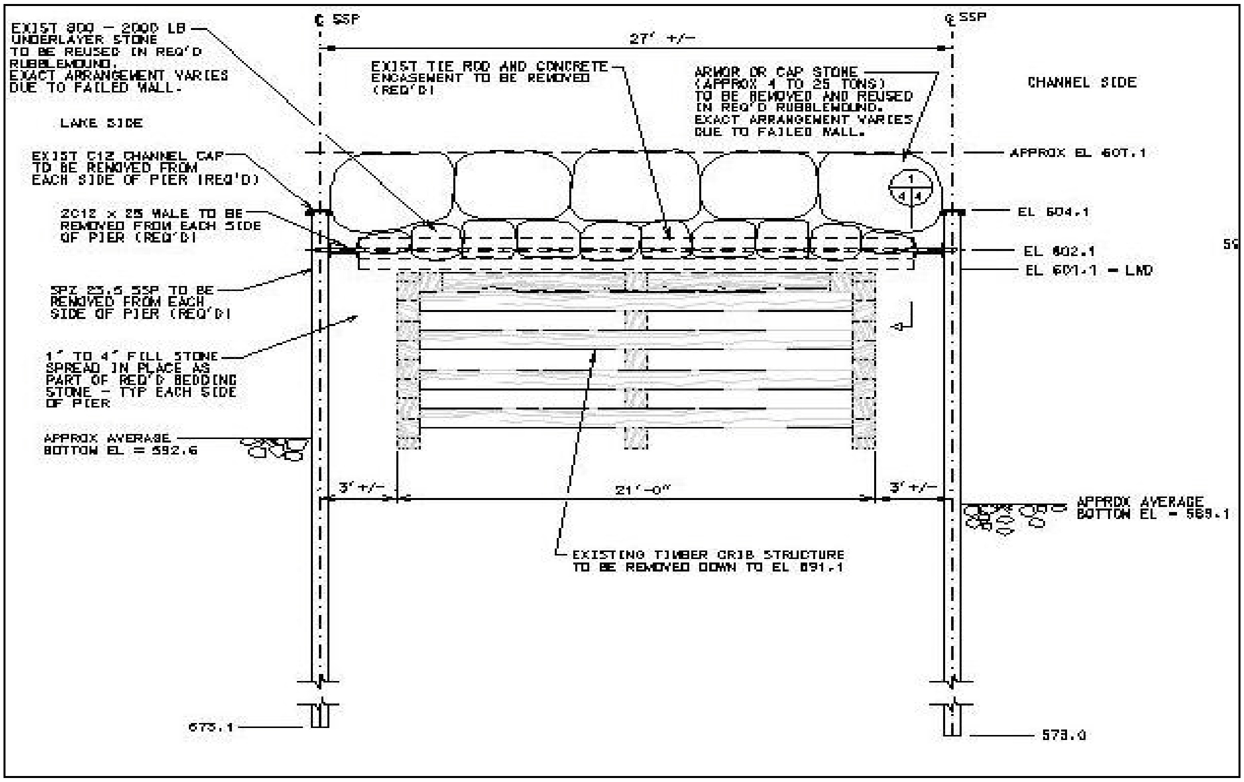


Figure K-7a. Timber crib design details referenced to IGLD85 (601.1) Datum.

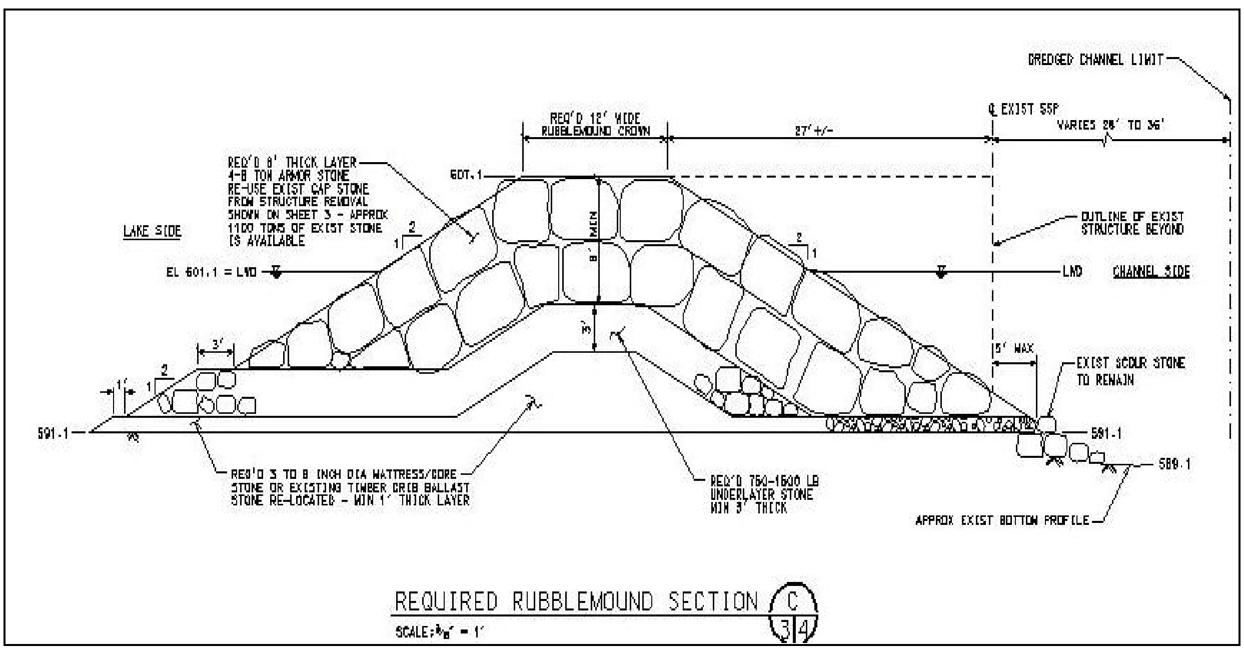


Figure K-7b. Rubblemound design details referenced to IGLD85 (601.1) Datum.

f. Geodetic and water level references and uncertainties. Based on the published data, the geodetic and "tidal datum" relationships at Tidal Bench Mark “D-135” could be tabulated as shown in Table K-2.

Table K-2. Elevations at Tidal Bench Mark “D-135.”

Datum	Elevation	Referenced From	Estimated Uncertainty	Relative to
NGVD29	613.22 ft	VERTCON transform	±0.3 ft	NSRS
IGLD85	613.01 ft	Tidal BM D-135	±0.2 ft	NGS
NAVD88	613.11 ft	NSRS	±0.1 ft	NSRS
Dynamic Ht	613.21 ft	Tidal BM D-135	±0.2 ft	NWLON

K-4. Background on Establishment of IGLD85. The following paragraphs provide additional background on the establishment of IGLD85 in the Great Lakes. They are excerpted from "*Establishment of International Great Lakes Datum (1985)*" (IJC 1995) by The Coordinating Committee on Great Lakes Basic Hydraulic and Hydrologic Data.

The establishment of the International Great Lakes Datum (1955), or IGLD (1955), was one of the first major accomplishments of the Coordinating Committee. Accordingly the reference zero point was established at Point-au-Père, Quebec and first-order leveling begun in 1953 was completed in 1961. The established bench mark elevations were published In September 1961 (A second edition was also published with some revisions In December 1979). The result of this effort was the International Great Lakes Datum 1955. This datum was implemented January 1, 1962, and used for the following 30 years, until the effects of crustal movement, the development of a common datum between Canada, the United States, and Mexico, new surveying methods, and the deterioration of the zero reference point gauge location made it desirable to revise the datum. The Vertical Control-Water Levels Subcommittee undertook the revision of IGLD (1955) beginning In 1976 and this effort has resulted in International Great Lakes Datum (1985) ... The development of the NAVD (1988) ... was to Include vertical control networks of the U.S., Canada and Mexico, as well as International Great Lakes Datum data. For NAVD (1988), a minimum-constraint adjustment was performed also holding fixed the primary bench mark at Pointe-au-Père/Rimouski, Therefore, IGLD (1985) and NAVD (1988) are one and the same. The only difference between IGLD (1985) and NAVD (1988) is that the IGLD (1985) bench mark elevations are published as dynamic heights and the NAVD (1988) *elevations are published as Helmert orthometric heights Geopotential numbers for individual bench marks are the same in both height systems.*

<u>*Dynamic height values*</u>*. The surveying and mapping community uses several different heights systems. Two systems, orthometric and dynamic heights, are relevant to the establishment of IGLD (1985) and NAVD (1988). The geopotential numbers for individual bench marks are the same in both height systems. The requirement in the Great Lakes basin to provide an accurate measurement of potential hydraulic head is the primary reason for adopting dynamic heights, it should be noted that dynamic heights are basically geopotential numbers scaled by a constant of 980.6199 gals, normal gravity at sea level at 45 degrees latitude. Therefore, dynamic heights are also an estimate of the hydraulic head. Consequently points that have the same geopotential number have the same dynamic height. Following are some of the advantages of dynamic heights:*

(1) In crustal movement studies, differences in the dynamic elevation of bench marks from lake to lake can be compared regardless of the route along which the leveling is done. This is also possible in the orthometric height system and with geopotential numbers.

(2) Difference in dynamic heights and in geopotential numbers give an accurate measure of the potential hydraulic head between selected points. This is not true of orthometric heights.

<u>*Hydraulic Corrector*</u>*. The water surfaces of the Great Lakes are considered to be geopotentially equal. Therefore, on any particular lake, at the time a new vertical datum is established, all Mean Water Level (MWL) values for gauging stations around the lake should coincide. The MWL is the average water surface for the summer months (June - September) for the years 1982-1988 referenced to the gauging station Primary Bench Mark dynamic height ... the MWL at each gauging station was treated as a bench mark In the network adjustment. Following the adjustment ..., the MWL values at each gauging station on a lake were slightly different. The differences are due to cumulative differences in the leveling adjustments. The Committee decided to apply a Hydraulic Corrector so each gauge on a lake has the same MWL as the Master Station for the lake. This is accomplished by holding the Master Station as the controlling value and comparing all other gauging stations to it. The Master Stations for each lake are:*

Lake Ontario	*Oswego, New York*
Lake Erie	*Fairport, Ohio*
Lake St. Clair	*St. Clair Shores, Michigan*
Lake Huron	*Harbor Beach, Michigan*
Lake Michigan	*Harbor Beach, Michigan*
Lake Superior	*Marquette, Michigan*

The Hydraulic Corrector (HC) was obtained by subtracting the MWL at the Master Station (MWL_{Master}) *from the MWL at the subordinate gauging station in question* (MWL_{Sub}). *The answer retains its arithmetic sign. The Hydraulic Corrector may be positive or negative and is subtracted algebraically.*

$$HC = MWL_{Sub} - MWL_{Master}$$

where:

HC = Hydraulic Corrector for subordinate gauge.

MWL_{Sub} = Mean Water Level at Subordinate Gauging Station on a lake for the summer months (June - September) of 1982 - 1988. The MWL is referenced to the Subordinate Gauging Station Primary Bench Mark Dynamic Height.

MWL_{Master} = Mean Water Level at Lake Master Station for the summer M W' months (June - September) of 1982 - 1988. The MWL is referenced to the Master Station Primary Bench Mark Dynamic Height.

The water surface elevation ($WS_{IGLD1985}$) is obtained by subtracting the Hydraulic Corrector (HC) from the Dynamic Water Surface Elevation ($WS_{Dynamic}$).

$$WSI_{GlD1985} – WS_{Dynamic} - HC$$

where:

$WS_{IGLD1985}$ = Published Water Surface Elevation on IGLD (1985) for a selected gauging station. The value may be an instantaneous value, or a daily, monthly, or annual mean.

$WS_{Dynamic}$ = Water Surface elevation referenced to Dynamic Height.

HC = Hydraulic Corrector for a selected gauging station. The value may be positive or negative

The Hydraulic Corrector at each gauge site on the lake has been incorporated into the data retrieval and storage process. As such, water level information stored at the site mechanically or electronically, at the National Oceanic and Atmospheric Administration (NOAA) or the Department of Fisheries and Oceans (DFO) computers, or in printed form, are in IGLD (1985) and do not require any further adjustment

The advantages of IGLD (Ì985), leading to the Coordinating Committee recommendations, may be summarized as follows:

(1) Elevations, consistent with one another as of a recent date (1985), are provided for bench marks throughout the Great Lakes-St, Lawrence River system, with the reference zero at Pointe-au- Père/Rimouski.

(2) The elevations given on this datum are based on the dynamic principle, and are therefore more suitable for hydraulic studies. Elevations on this new datum will greatly facilitate hydraulic, hydrographic and other engineering studies.

K-5. Tabulation of Great Lakes and Connecting Channels Water Level Datums (NOAA CO-OPS).

STATION NUMBER	LWD (Meters)	LWD (Feet)	STATION NAME, STATE & BODY OF WATER
ST. LAWRENCE RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)			
8311030	73.88	242.4	Ogdensburg, NY St. Lawrence River
8311062	74.07	243.0	Alexandria Bay, NY St. Lawrence River
LAKE ONTARIO (CHART DATUM/LWD 74.2 M - 243.3 FT.)			
9052000	74.2	243.3	Cape Vincent, NY Lake Ontario
9052030	74.2	243.3	Oswego, NY Lake Ontario
9052058	74.2	243.3	Rochester, NY Lake Ontario NY
9052076	74.2	243.3	Olcott, NY Lake Ontario
NIAGARA RIVER (NON NAVIGABLE WATERS)			
9063007	N/A	N/A	Ashland Ave., NY Niagara Falls-Below the falls
9063009	N/A	N/A	American Falls, NY Niagara Falls-Above the Falls
9063012	N/A	N/A	Niagara Intake, NY Niagara River – Power diversion water intakes
LAKE ERIE (CHART DATUM/LWD 173.5 M - 569.2 FT.)			
9063020	173.5	569.2	Buffalo, NY Lake Erie
9063028	173.5	569.2	Sturgeon Point, NY Lake Erie
9063038	173.5	569.2	Erie, PA Lake Erie
9063053	173.5	569.2	Fairport, OH Lake Erie
9063063	173.5	569.2	Cleveland, OH Lake Erie
9063079	173.5	569.2	Marblehead, OH Lake Erie
9063085	173.5	569.2	Toledo, OH Lake Erie
9063090	173.5	569.2	Fermi Power Plant, MI Lake Erie
DETROIT RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)			
9044020	173.58	569.5	Gibraltar, MI Detroit River
9044030	173.95	570.7	Wyandotte, MI Detroit River
9044036	174.08	571.1	Fort Wayne, MI Detroit River
9044049	174.34	572.0	Windmill Point, MI Detroit River

K-5 (Continued). Tabulation of Great Lakes and Connecting Channels Water Level Datums (NOAA CO-OPS).

STATION NUMBER	LWD (Meters)	LWD (Feet)	STATION NAME, STATE & BODY OF WATER
LAKE ST. CLAIR (CHART DATUM/LWD 174.4 M - 572.3 FT.)			
9034052	174.4	572.3	St. Clair Shores, MI Lake St. Clair
ST. CLAIR RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)			
9014070	174.58	572.8	Algonac, MI St. Clair River
9014080	175.08	574.4	St. Clair State Police, MI, St. Clair River
9014084	175.35	575.3	Marysville, MI St. Clair River (in-active)
9014087	175.50	575.8	Dry Dock, MI St. Clair River
9014096	175.77	576.7	Dunn Paper, MI St. Clair River
9014098	175.93	577.2	Fort Gratiot, MI St. Clair River
LAKE HURON (CHART DATUM/LWD 176.0 M - 577.5 FT.)			
9075002	176.0	577.5	Lakeport, MI Lake Huron
9075014	176.0	577.5	Harbor Beach, MI Lake Huron
9075035	176.0	577.5	Essexville, MI Lake Huron
9075059	176.0	577.5	Harrisville, MI Lake Huron
9075065	176.0	577.5	Alpena, MI Lake Huron
9075080	176.0	577.5	Mackinaw City, MI Lake Huron
9075099	176.0	577.5	De Tour Village, MI Lake Huron
LOWER - ST. MARY'S RIVER (CHART DATUM/LWD-SEE STATION VALUE FOR SLOPING SURFACE)			
9076024	176.03	577.5	Rock Cut, MI St. Mary's River
9076024	176.12	577.8	West Neebish Island, MI
9076028	176.12	577.8	Lookout Station #4, MI St Mary's River
9076032	176.29	578.4	Little Rapids, MI St Mary's River
9076060	176.38	578.7	U.S. Slip, MI St. Mary's River
UPPER - ST. MARY'S RIVER (CHART DATUM/LWD - SEE STATION VALUE FOR SLOPING SURFACE)			
9076070	183.00	600.4	S.W. Pier, MI St. Mary's River

K-5 (Concluded). Tabulation of Great Lakes and Connecting Channels Water Level Datums (NOAA CO-OPS).

STATION NUMBER	LWD (Meters)	LWD (Feet)	STATION NAME, STATE & BODY OF WATER
LAKE MICHIGAN (CHART DATUM/LWD 176.0 M - 577.5 FT.)			
9087023	176.0	577.5	Ludington, MI Lake Michigan
9087031	176.0	577.5	Holland, MI Lake Michigan
9087044	176.0	577.5	Calumet Harbor, IL Lake Michigan
9087057	176.0	577.5	Milwaukee, WI Lake Michigan
9087068	176.0	577.5	Kewaunee, WI Lake Michigan
9087072	176.0	577.5	Sturgeon Bay Canal, WI Lake Michigan
9087079	176.0	577.5	Green Bay, WI Lake Michigan
9087088	176.0	577.5	Menominee, MI Lake Michigan
9087096	176.0	577.5	Port Inland, MI Lake Michigan
LAKE SUPERIOR (CHART DATUM/LWD 183.2 M - 601.1 FT.)			
9099004	183.2	601.1	Point Iroquois, MI Lake Superior
9099018	183.2	601.1	Marquette C.G. MI Lake Superior
9099044	183.2	601.1	Ontonagon, MI Lake Superior
9099064	183.2	601.1	Duluth, MN Lake Superior
9099090	183.2	601.1	Grand Marais, MN Lake Superior

APPENDIX L

Computing Historical Subsidence Rates in Southeast Louisiana from USACE Gage Data

L-1. Purpose. This appendix describes procedures for evaluating subsidence rates based on long-term gage records. It is extracted from internal studies performed in the USACE New Orleans District.

L-2. Abstract. The New Orleans District records stream and tide stages at numerous gaging sites throughout its district. Many of these stage data sets extend as far back as 1940. The data through 1998 have been published by the District in "Stage and Discharge" books and much of it has been converted to digital format. As such, it is often readily accessible, reasonably well documented, and may provide an independent means to investigate and determine reliable rates of local subsidence and/or validate rates determined via geodetic survey analysis.

L-3. Introduction. Subsidence, or the generally downward motion of the earth's surface with respect to some "fixed" vertical datum (e.g., NAVD88 or a particular Mean Sea Level epoch), is perceived as a significant threat to coastal Louisiana. It has been detected in both geodetic leveling data throughout the region and at several NOAA tide gages along the Louisiana coast (Shinkle and Dokka, 2004; Dokka, 2006)[1]. From the referenced studies, it appears that rates of subsidence in the region are spatio-temporally variant and often significantly greater than even the highest estimated rates of eustatic sea-level rise. Consideration of the forces and conditions that cause subsidence - and the proportion that each force or condition contributes to total subsidence at a given point and at a given time (Gonzalez and Tornqvist, 2006; Dokka, 2006) - is beyond the scope of this work. The sole purpose here is to extract reliable historical rates of subsidence at various USACE gages and, when/if possible, determine the relationships among the various vertical geodetic datum/epochs and local mean-sea-level epochs.

L-4. Data. Five USACE tide gage stations in the New Orleans area were selected for this study. They are as follows: 76040 - The Intracoastal Waterway (IWW) at the Paris Road Bridge; 76060 - The Inner Harbor Navigation Canal (IHNC) at the Seabrook Bridge; 76120 - The IHNC at the Florida Avenue Bridge; 85675 - Lake Pontchartrain at Irish Bayou; 85700 – Lake Pontchartrain at the Rigolets (see Figures L-1, L-2, and L-3).

[1] References cited in this Appendix are listed in Section L-10.

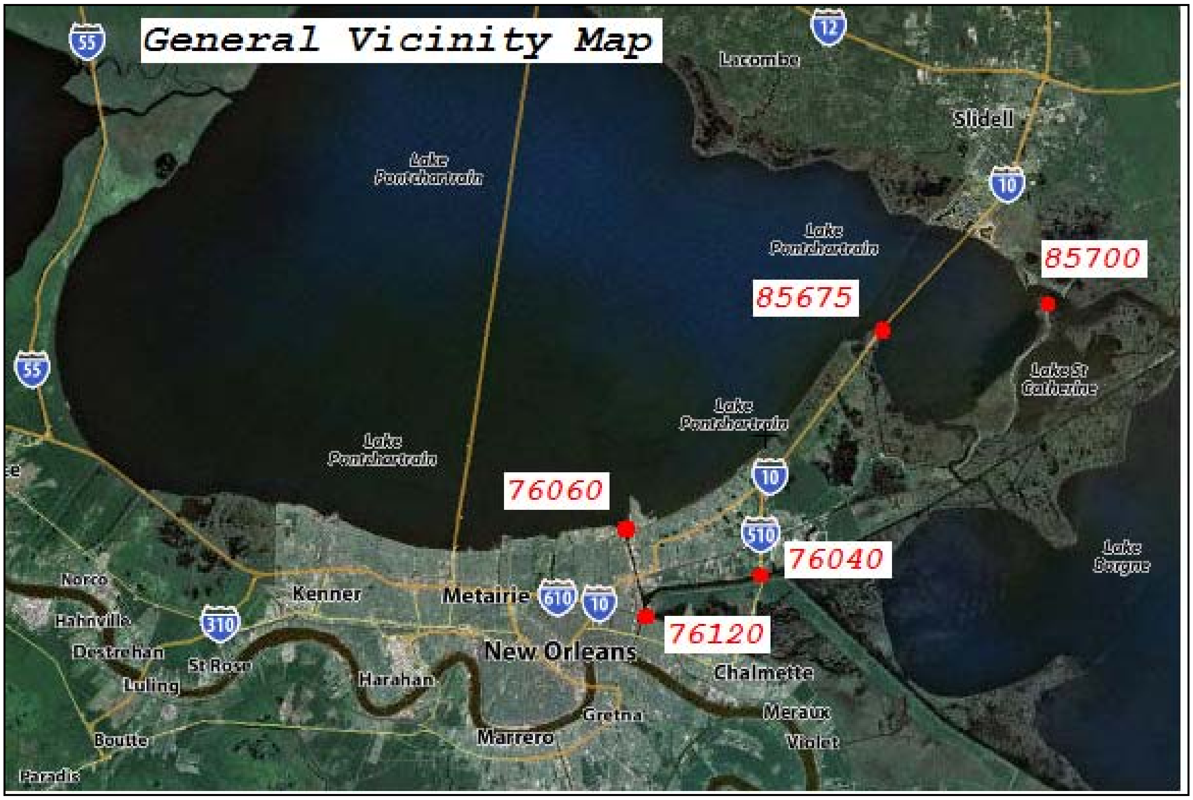


Figure L-1. Vicinity map of Greater New Orleans showing gage locations.

a. These tidal gages were selected because of their proximity and relevance to post-Katrina reconstruction activities in New Orleans East, the availability of near-continuous, long-term (40+ years) digital data, and adequate documentation of periodic gage inspections and adjustments. This collection of gages also encompasses an area of detailed study on modern-day tectonic subsidence in coastal Louisiana performed by Dokka (2006).

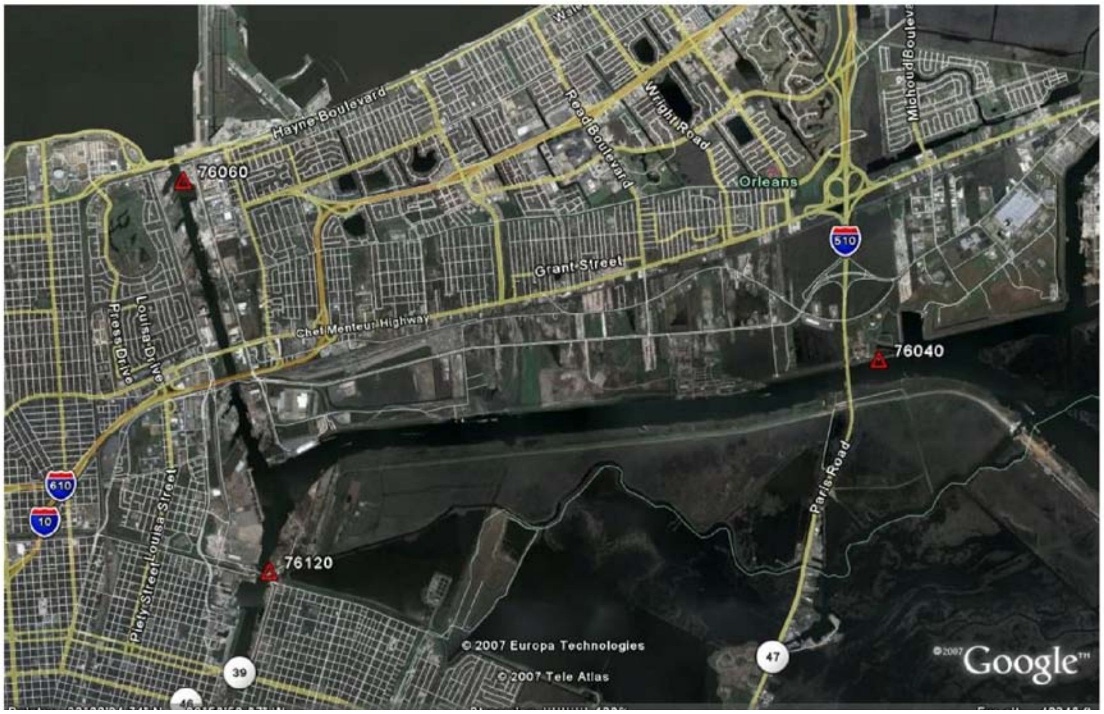


Figure L-2. Vicinity map of East New Orleans showing gage locations.

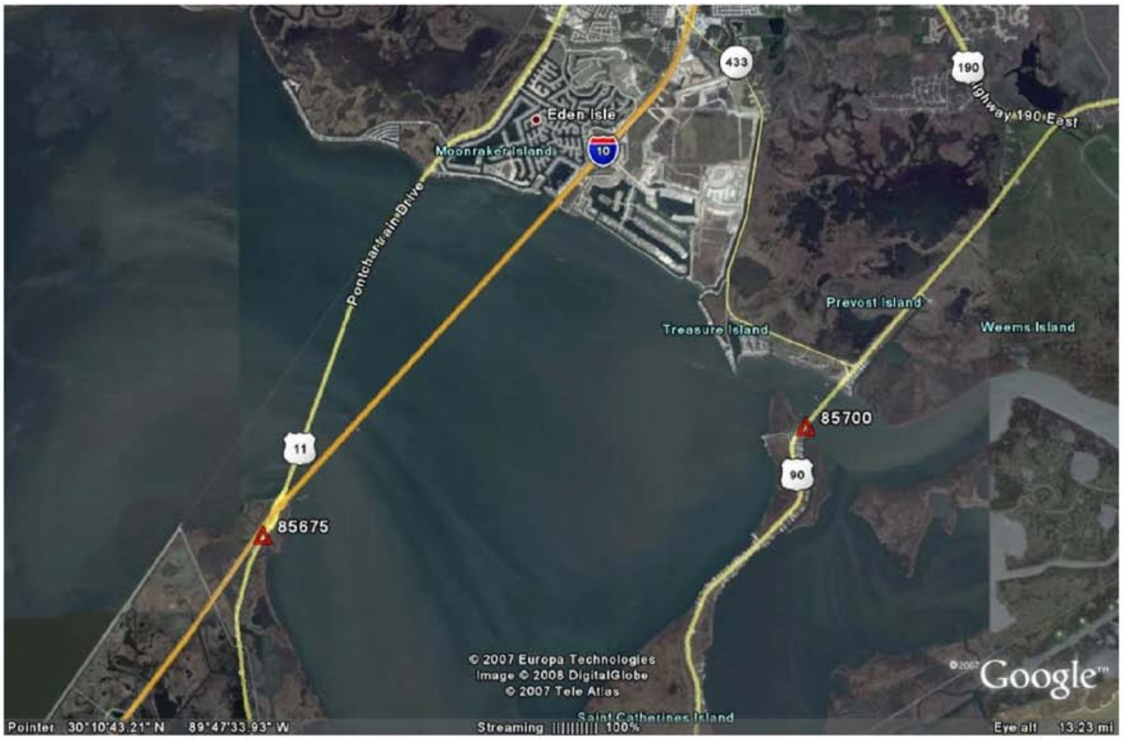


Figure L-3. Vicinity map of Western Lake Pontchartrain showing gage locations.

b. The available digital data ranges for the selected gages are shown in Table L-1. The data sets are, for the most part, continuous through their respective time periods, but there are several gaps (of days, weeks, and sometimes months) in each of the data sets. More significantly, all of the original data sets are nominally referred to as "Daily 8 A.M. Stage Readings in Feet." Occasionally, stage readings were made directly by an observer at times other than 8 A.M. In addition, the type of gage employed, its precise location, and the type of structure on which it was mounted were all altered one or more times at each of the gaging sites over the life of its operation. As such, the data sets may be somewhat "noisy," as compared to a theoretically continuous data set of hourly or six-minute-interval stage readings from a single, calibrated instrument at a fixed location.

Table L-1. The Available Digital Data Ranges for Selected Gages

Gage	Year Period Began	Year Period Ended
76040	1959	2007
76060	1962	2005
76120	1944	2003
85675	1959	2000
85700	1961	2001

c. The most significant artifacts evident in each of the data sets were the discontinuities resulting from the deliberate change in the vertical position of the gage zero with respect to nearby benchmarks. These alterations were periodically carried out so that the “zero” of the gage would correspond to the “zero” of a particular epoch of a vertical geodetic datum (see Figure L-4, for example). Fortunately, these vertical movements or adjustments of the gage zeros are reasonably well documented in the gage inspection records together with the explanatory notes in the “Stages and Discharges” books. Accounting for these adjustments was essential to the development of continuous, normalized data sets of daily, 8 A.M. stage readings from which monthly means could be reliably computed and subsidence rates determined.

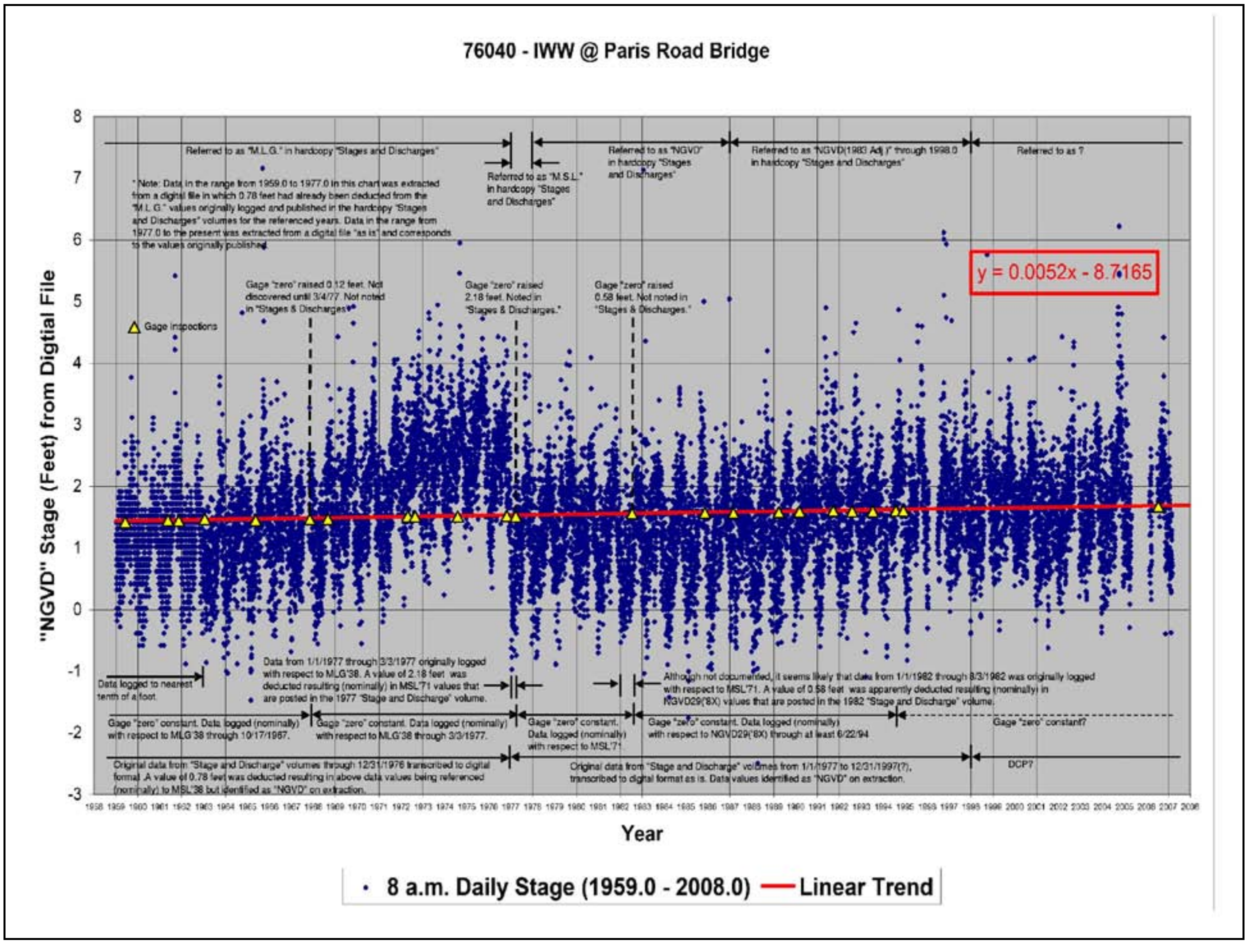


Figure L-4. Gage readings for gage 76040 – IWW @ Paris Road.

L-5. Data Normalization. In normalizing the digital stage data, it was first necessary to account for differences between the digital stage values extracted from the USACE database, and those corresponding values recorded in the “Stages and Discharges” books. These differences are due to a bulk shift - applied on extraction by USACE software - to that portion of a given stage data set originally recorded with respect to a gage zero intentionally offset from nominal mean-sea-level (i.e., “Mean-Low-Gulf” or “-10.00 ft MSL” or “-20.55 ft MSL). This is apparently done in order to roughly harmonize it with subsequent stage values nominally referenced to MSL/NGVD. Removal of this shift was necessary to reproduce the actual stage values recorded in the “Stage and Discharge” books. Following this adjustment, the digital stage data were corrected for intentional vertical movement of the gage zero due to epoch updates as indicated in the inspection records. The desire here was to compute, as nearly as possible, the stage data set that would have been generated if the gage zero had never been intentionally moved from its initial vertical position with respect to the surrounding terrain and associated benchmarks. Applying the corrections for gage zero movement noted in Figure L-4 resulted in the “normalized” data set shown in Figure L-5.

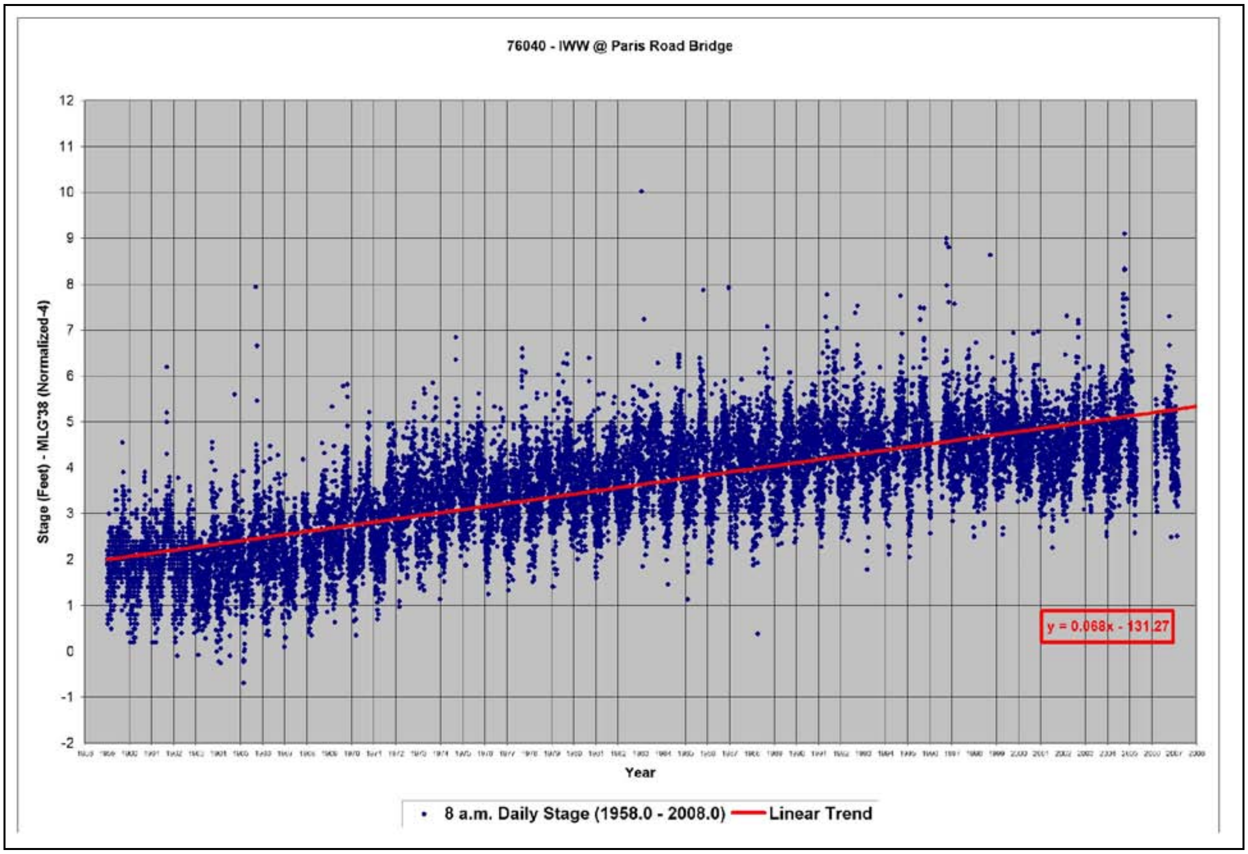


Figure L-5. Normalized gage readings for gage 76040 – IWW @ Paris Road.

a. The review, analysis, and processing steps undertaken with gage 76040, as indicated above in Figures L-4 and L-5, were similarly carried out with respect to the remaining four gages. As a final quality assurance step, all five normalized data sets were differenced against one another (see Figures L-6 through L-15). If all intentional gage zero movement has been accounted for (and if no accidental zero movement or loss of calibration has occurred), then one would expect the graph of the differences to be relatively consistent, smooth and continuous. If subsidence rates were generally the same at any two gages under consideration, the graph would be essentially flat as well. Where rates differ, one would expect that the magnitude of that rate difference would be born out in the slope of the graph.

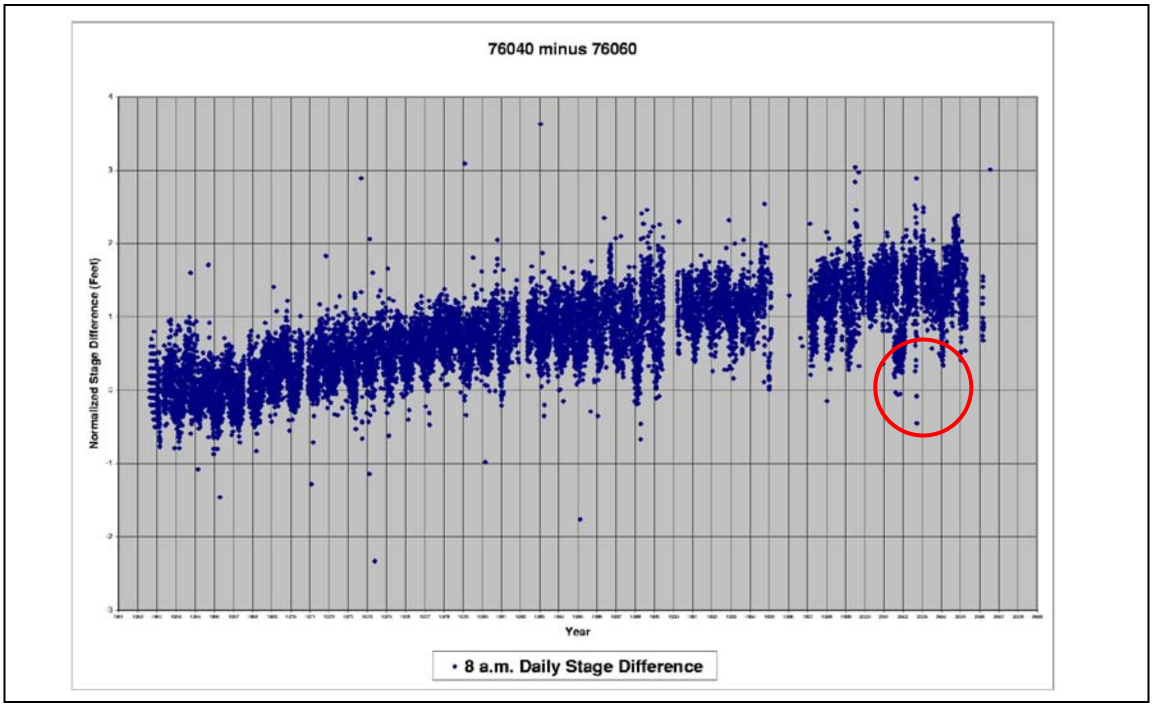


Figure L-6. Normalized gage readings for Gage 76040 – Gage 76060.

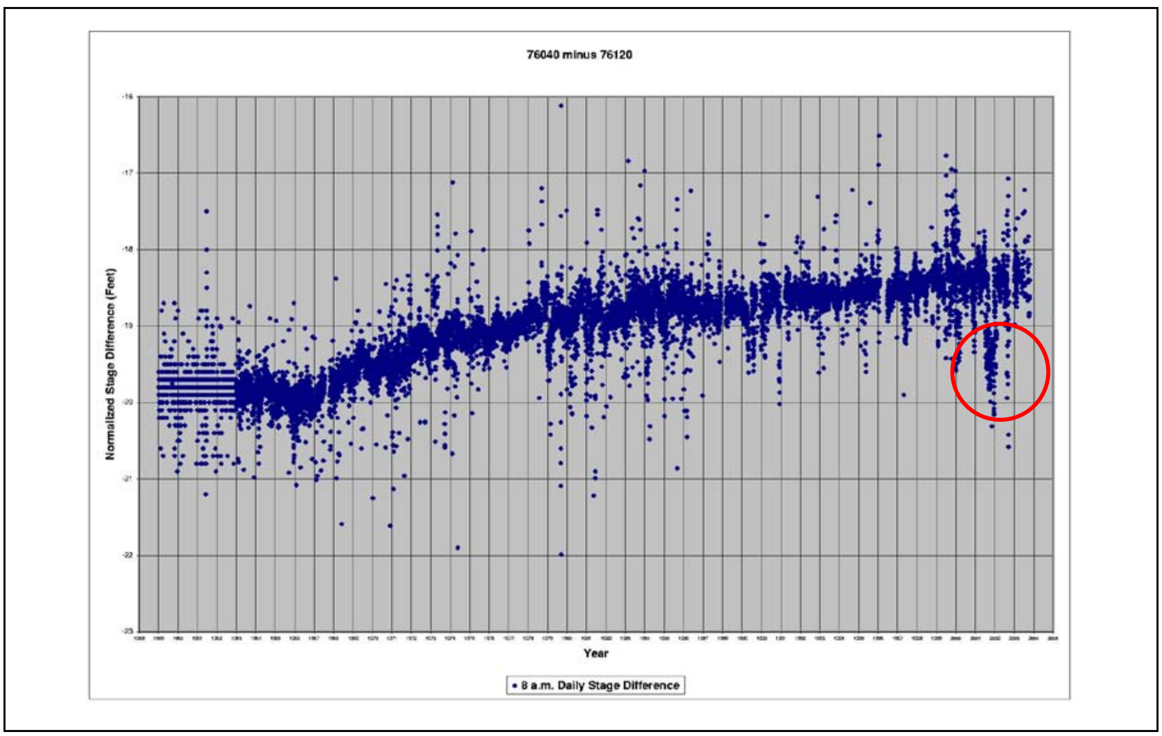


Figure L-7. Normalized gage readings for Gage 76040 – Gage 76120.

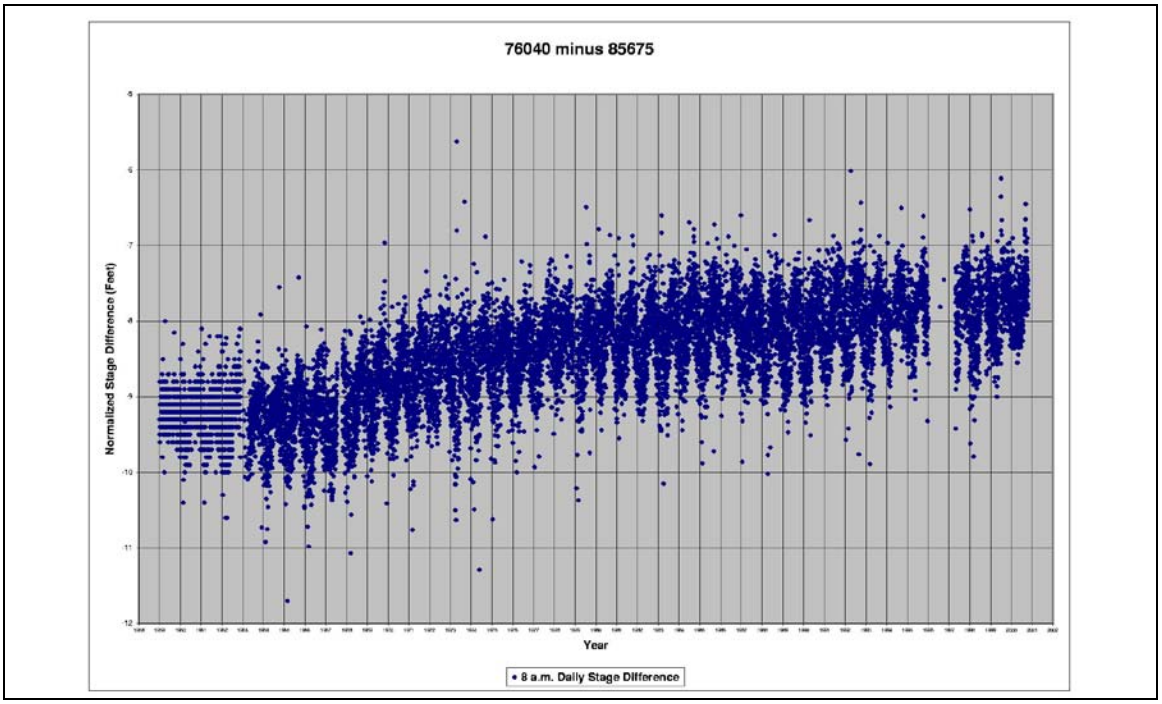


Figure L-8. Normalized gage readings for Gage 76040 – Gage 85675.

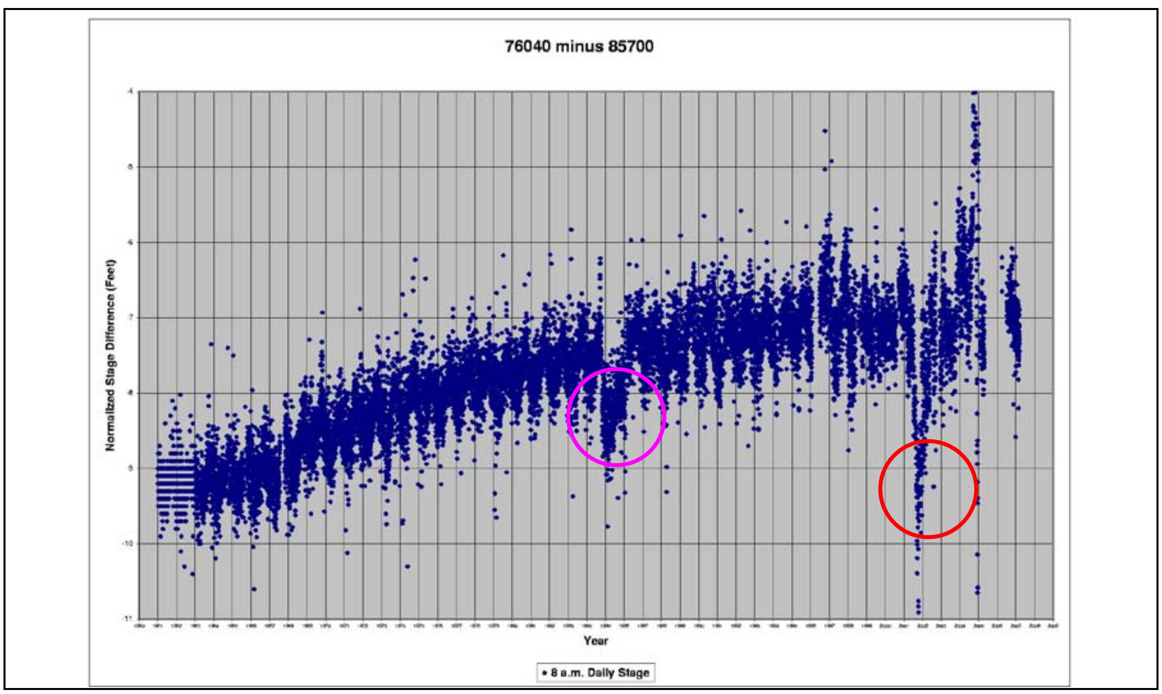


Figure L-9. Normalized gage readings for Gage 76040 – Gage 85700.

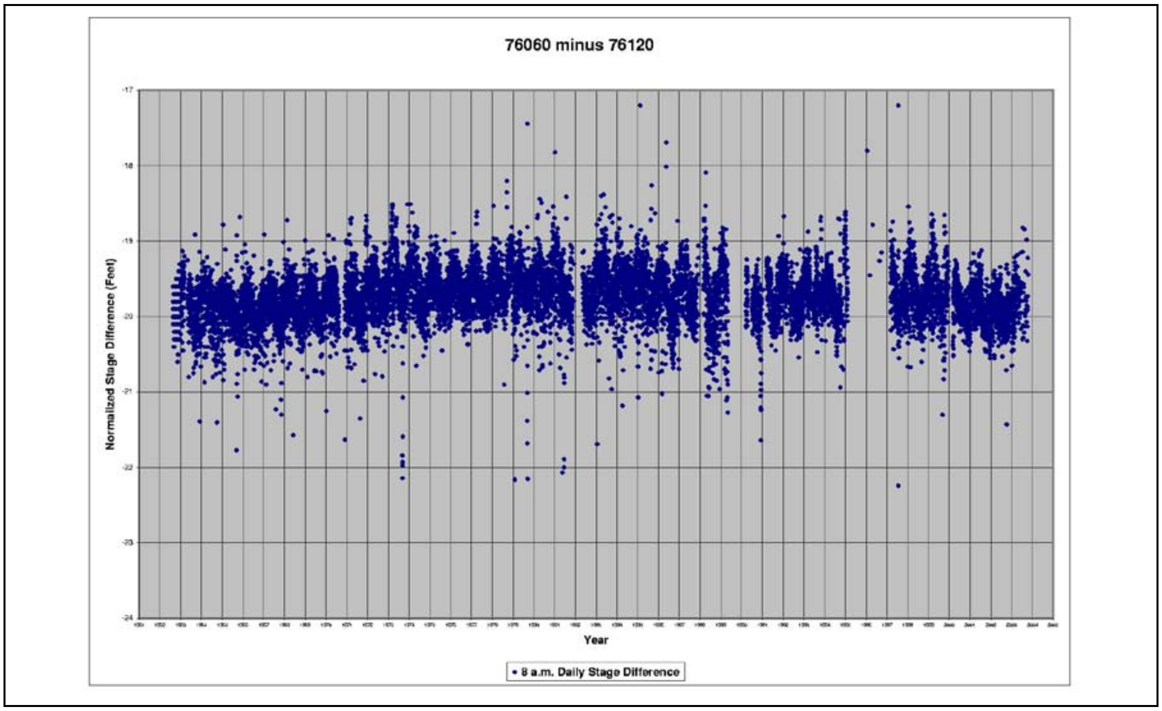


Figure L-10. Normalized gage readings for Gage 76060 – Gage 76120.

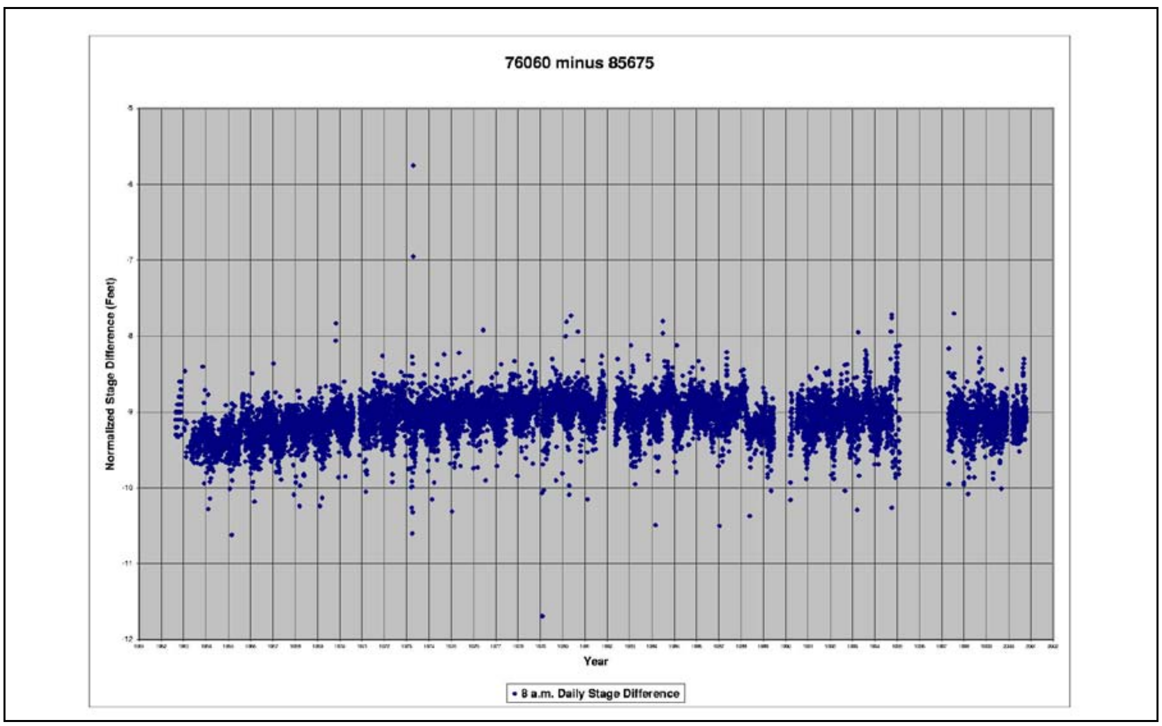


Figure L-11. Normalized gage readings for Gage 76060 – Gage 85675.

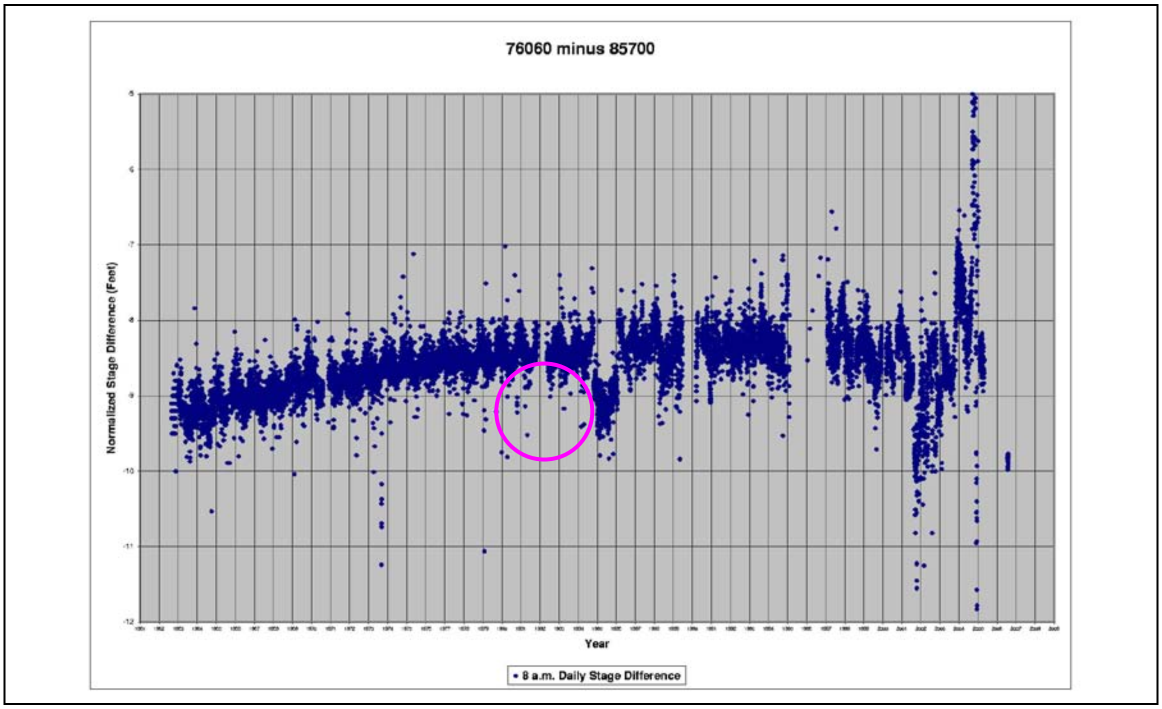


Figure L-12. Normalized gage readings for Gage 76060 – Gage 85700.

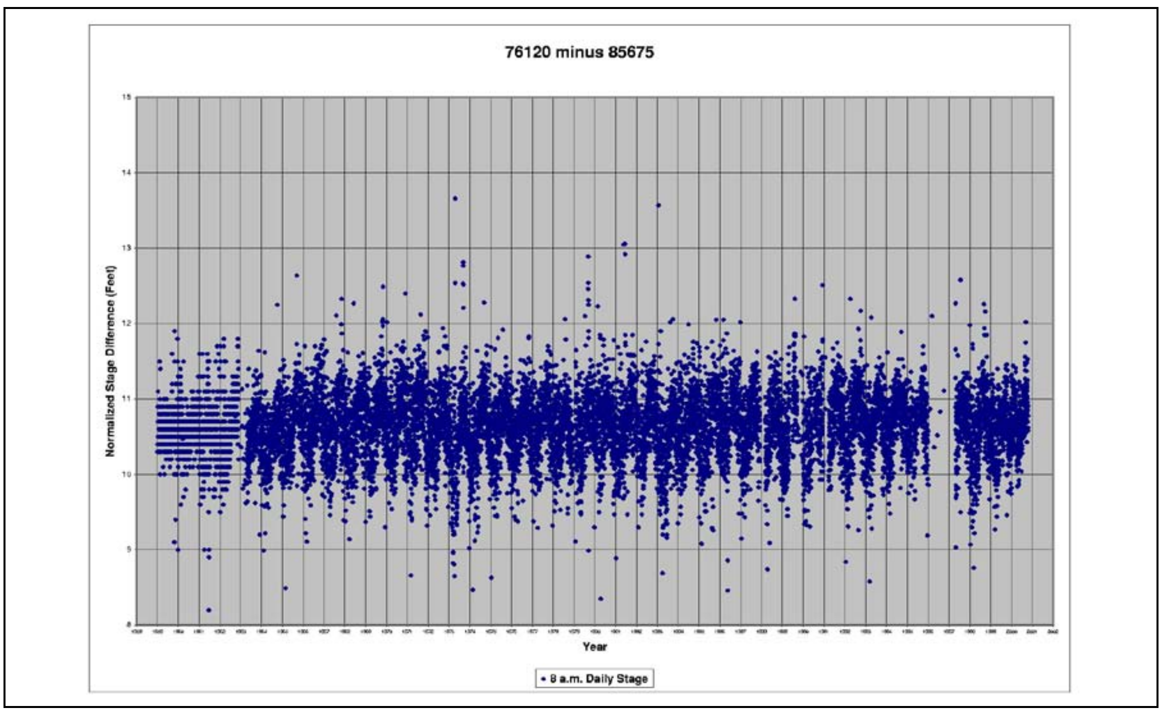


Figure L-13. Normalized gage readings for Gage 76120 – Gage 85675.

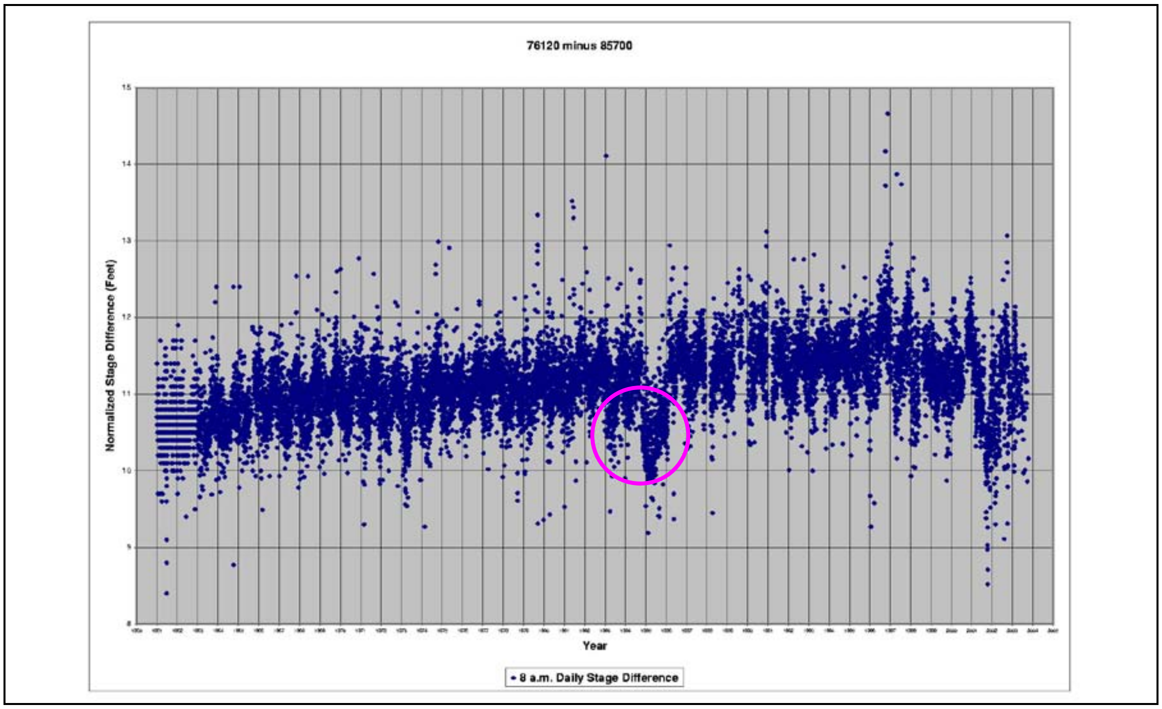


Figure L-14. Normalized gage readings for Gage 76120 – Gage 85700.

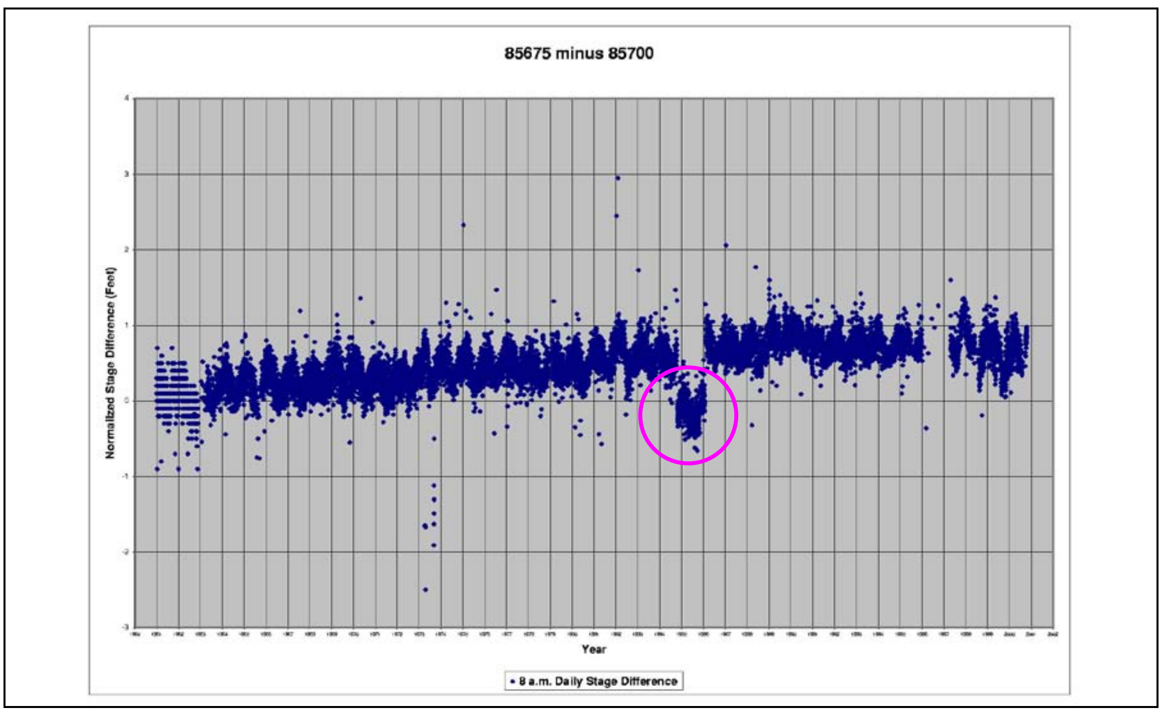


Figure L-15. Normalized gage readings for Gage 85675 – Gage 85700.

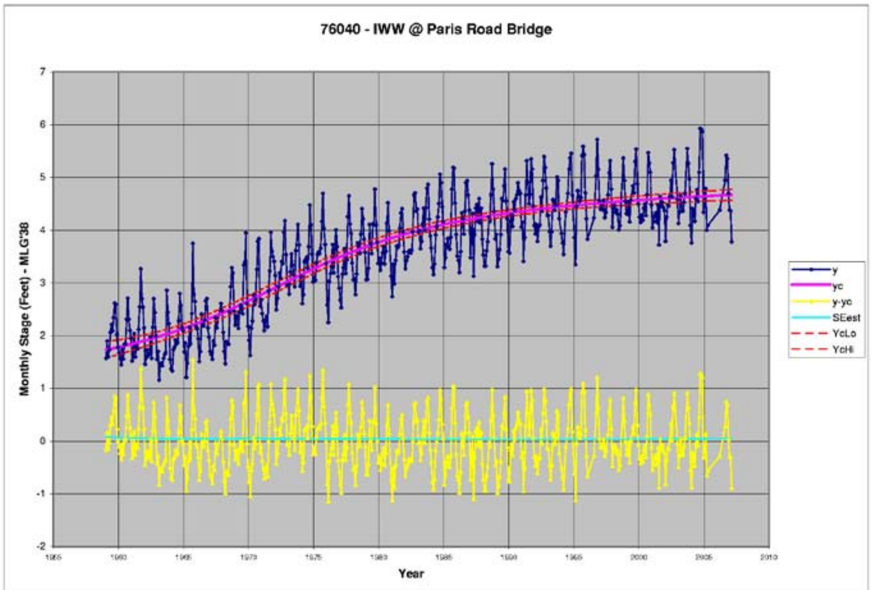
b. Figures L-6 through L-15 generally exhibit the anticipated characteristics (i.e., consistent, smooth, continuous) indicating that all intentional vertical movement of the gage zeros has been accounted for. However, the areas indicated by the red circles suggest that the zero for gage 76040 was disturbed or out of calibration during the latter half of 2001. The areas indicated by the purple circles suggest that the zero for gage 85700 was disturbed or out of calibration during most of 1985. These apparent data problems are noted here but, since their precise cause and magnitude have not been determined and their duration is relatively short lived, the anomalous regions were not removed or altered for subsequent processing. Only the clearly erratic data from mid 2001 forward at gage 85700 has been disregarded in subsequent processing.

L-6. Data Regression. In all of the following discussion, the increase in stage values over time (apparent in Figure L-5 above, for example) is, for simplicities sake, assumed to be attributable solely to subsidence. Although this is almost certainly not the case in reality, it is for the moment, a useful practical assumption. The effects and contribution of eustatic sea level rise will be reintroduced and considered later where necessary. Also, though it may run counter to convention, downward displacement of the land (as reflected by increasing stage readings) is considered positive subsidence (for the purposes of this document, downward displacement of the land and rates of downward displacement will be treated as numerically positive values).

a. A straight-line regression was computed for the 76040 gage data set (see Figure L-5 above). The slope of the line indicates that the average annual apparent subsidence rate over the entire period from 1959 to 2007 is 0.068 feet/year or 21 mm/year, an order of magnitude greater than a commonly accepted estimate for the rate of eustatic sea level rise. This result was essentially duplicated by specialists at NOAA's Center for Operational Oceanographic Products and Services (CO-OPS) using the same normalized data set.

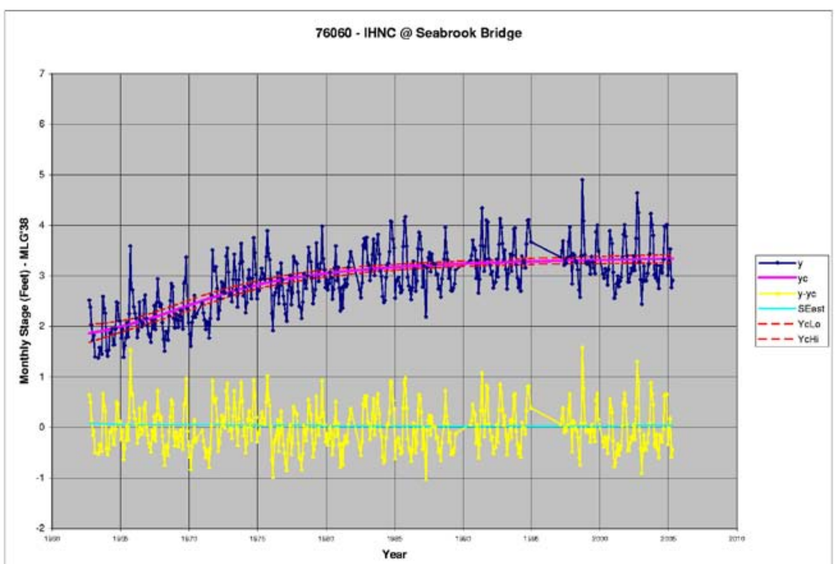
b. It should be noted that CO-OPS has employed basic straight-line regression in order to estimate rates of relative sea level rise at many of its coastal gaging stations, typically using 30 or more years of data (Stolz and Gill, 2005, Zervas 2001). CO-OPS is also responsible for computing National Tidal Datum Epoch (NTDE) mean values for tidal datums (e.g., mean sea level, mean lower low water, mean high water, etc.). The NTDE is the specific 19-year period adopted by NOAA as the official time segment over which tide observations are taken at a given station and reduced to obtain mean values for that station (Gill, et al, 2001). Given the periodic and apparent secular trends in sea level, the NTDE is typically revised every 20 to 25 years. The present NTDE is 1983 through 2001. It should also be noted that while mean sea level is defined as the arithmetic mean of hourly heights observed over the NTDE, a close approximation of mean sea level would be the mid-point of the linear trend (straight line fit via least-squares regression) of monthly means computed from hourly heights observed over the NTDE. The relevance of this point will be made clear subsequently.

c. In Figure L-5, it was apparent that a linear trend line was perhaps not the best model for the USACE gage data sets under consideration. In fact, each gage data set had a distinctly sigmoidal appearance, possibly characterized by a short period of relatively high subsidence preceded and followed by less dramatic rates. For this reason, an attempt was made to fit an arctangent function to the monthly means computed for each of the normalized gage data sets (for months having 20+ days of data). See Figures L-16 through L-20 below.



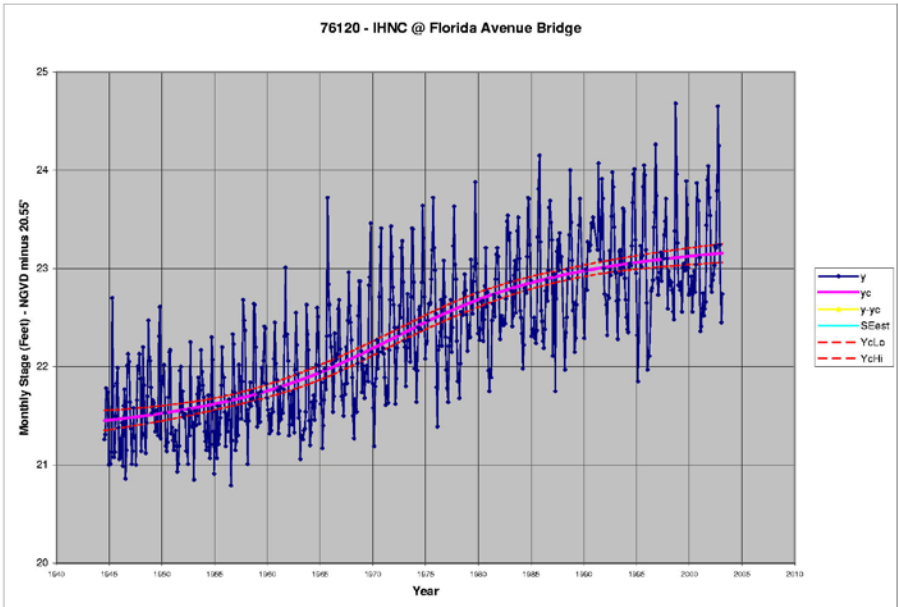
f(x) =	a*ATan(b*(x-c))+d			
a =	1.378155	±0.138453	Corr. Coeff.:	0.88289
b =	0.089980	±0.014565	r^2:	0.779494
c =	1972.207566	±1.119109	RMS Error:	0.512598
d =	2.930489	±0.101659		

Figure L-16. Arctangent function fit to the monthly means for Gage 76040 IWW @ Paris Road Bridge.



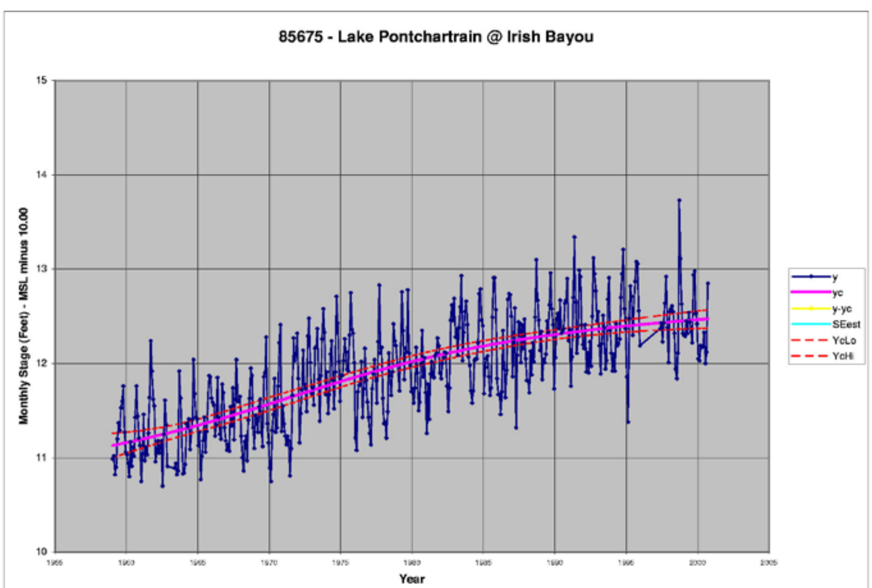
f(x) =	a*ATan(b*(x-c))+d			
a =	0.695612	±0.136575	Corr. Coeff.:	0.720065
b =	0.135400	±0.037931	r^2:	0.518493
c =	1969.704684	±1.896262	RMS Error:	0.433079
d =	2.390207	±0.137857		

Figure L-17. Arctangent function fit to the monthly means Gage 76060 IHNC @ Seabrook Bridge.



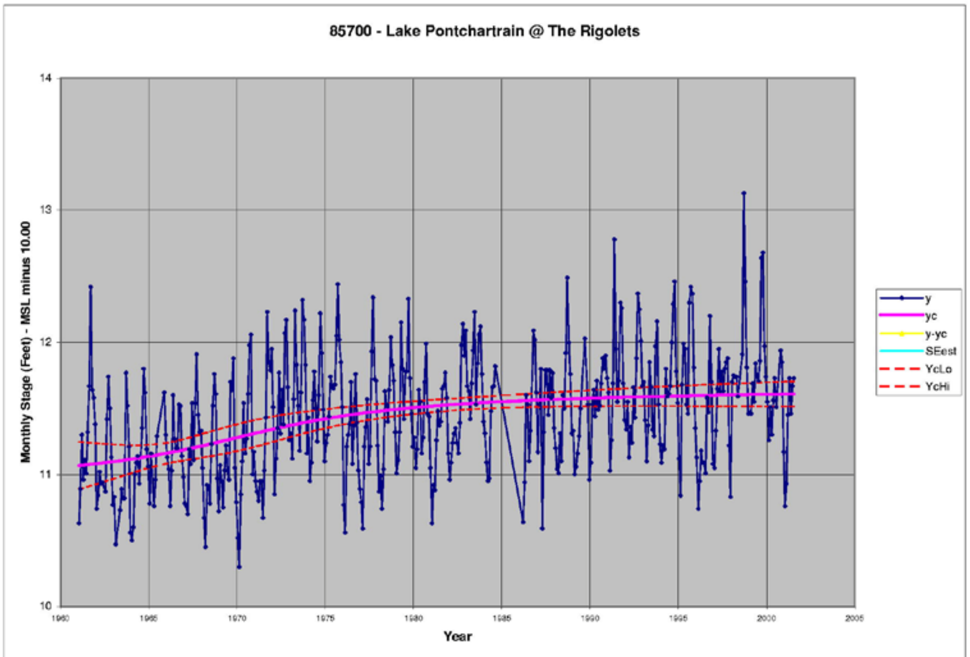
f(x) =	a*ATan(b*(x-c))+d			
a =	0.760559	±0.087261	Corr. Coeff.:	0.777721
b =	0.070699	±0.015234	r^2:	0.60485
c =	1971.786335	±1.298087	RMS Error:	0.482486
d =	22.28285	±0.04282		

Figure L-18. Arctangent function fit to the monthly means for Gage 76120 IHNC @ Florida Avenue Bridge.



f(x) =	a*ATan(b*(x-c))+d			
a =	0.779884	±0.227218	Corr. Coeff.:	0.744277
b =	0.061847	±0.023451	r^2:	0.553948
c =	1971.490714	±3.368383	RMS Error:	0.377163
d =	11.641887	±0.13693		

Figure L-19. Arctangent function fit to the monthly means for Gage 85675 Lake Pontchartrain @ Irish Bayou.

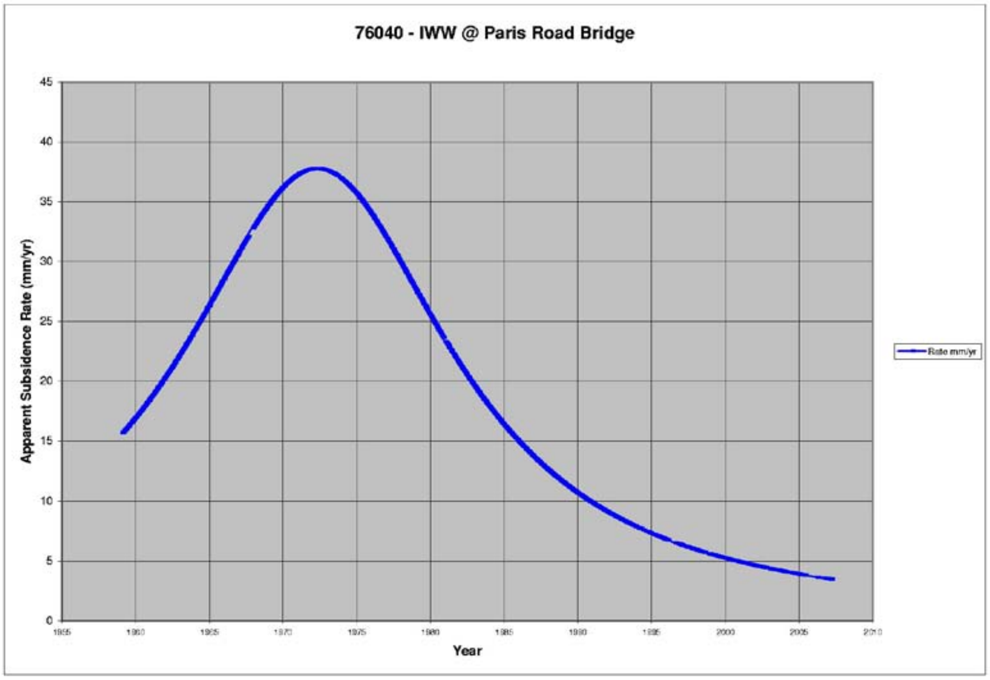


f(x) =	a*ATan(b*(x-c))+d				
a =	0.233812	±0.111057	Corr. Coeff.:	0.387012	
b =	0.144264	±0.09595	r^2:		0.149778
c =	1970.363367	±4.179788	RMS Error:	0.41044	
d =	11.288451	±0.098107			

Figure L-20. Arctangent function fit to the monthly means for Gage 85700 Lake Pontchartrain @ The Rigolets.

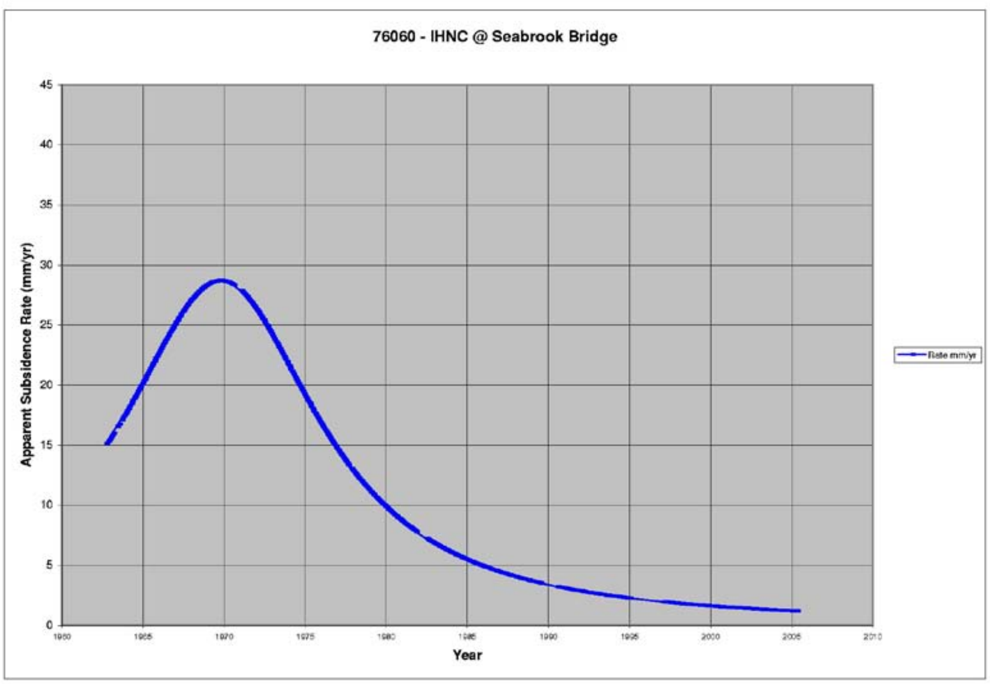
d. In Figures L-16 through L-20, “y” is the observed monthly mean stage computed from the daily 8 A.M. values: “yc” is the estimated mean stage value computed from the arctangent curve fit to the data through iterative, non-linear, least-squares regression; “y-yc” is the residual of observed minus estimated value; “SEst” is the estimate of standard error for each point computed along the curve; “YcHi” and “YcLo” are the upper and lower bounds of the 95% confidence interval of the curve. A notable result of the regression analysis is that the point of inflection, or the point at which the rate of apparent subsidence is at its maximum, occurs at very nearly the same moment in time for each gage site: 1972.2, 1969.7, 1971.8, 1971.5, and 1970.4 for gages 76040, 76060, 76120, 85675, and 85700 respectively. The RMS error, r^2, and Correlation Coefficient were, in all cases, an improvement over the corresponding values produced by straight-line regression. . Note that the graph of mean monthly stages for each gage exhibits (more or less) a seasonal or annually periodic cycle. The phase and amplitude of this apparent seasonal cycle appears to be roughly consistent with estimates for the Average Seasonal Cycle of Monthly Mean Sea Level computed by NOAA for several of their gages along the Louisiana/ Mississippi Gulf Coast (Zervas, 2001).

L-7. Apparent Subsidence Rates. A graph of the estimated instantaneous rate of apparent subsidence, expressed in mm/year rather than ft/year, for each gage was computed from the first derivative of their respective regression models and is shown in Figures L-21 through L-26 below.



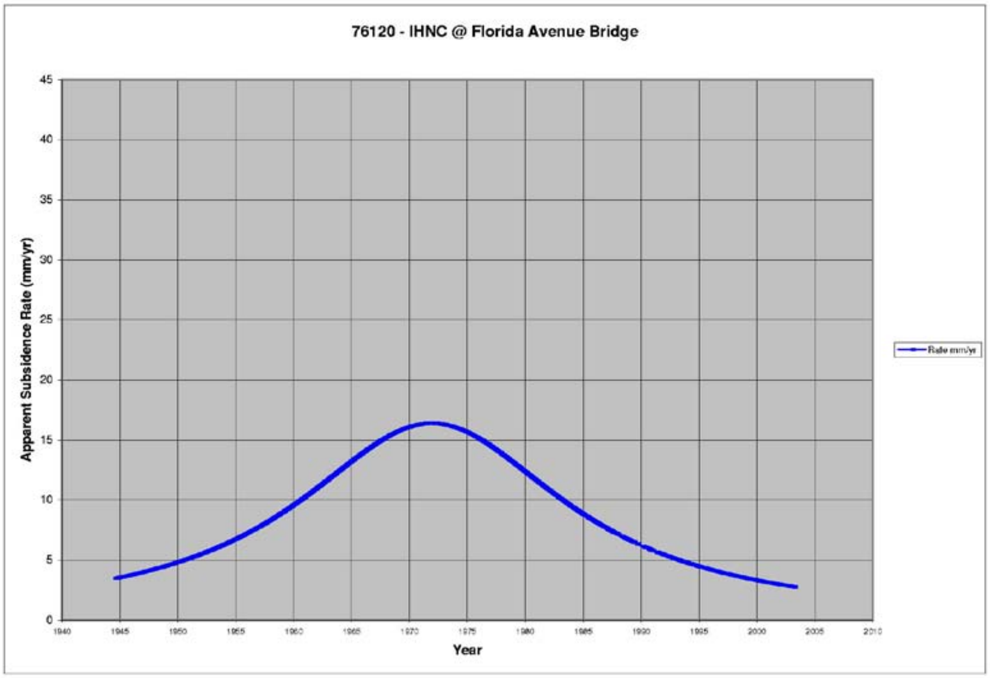
f'(x) = 0.124006/(0.0080964*(x-1972.207566)^2+1)

Figure L-21. Estimated Instantaneous rate of apparent subsidence for Gage 76040 IWW @ Paris Road Bridge.



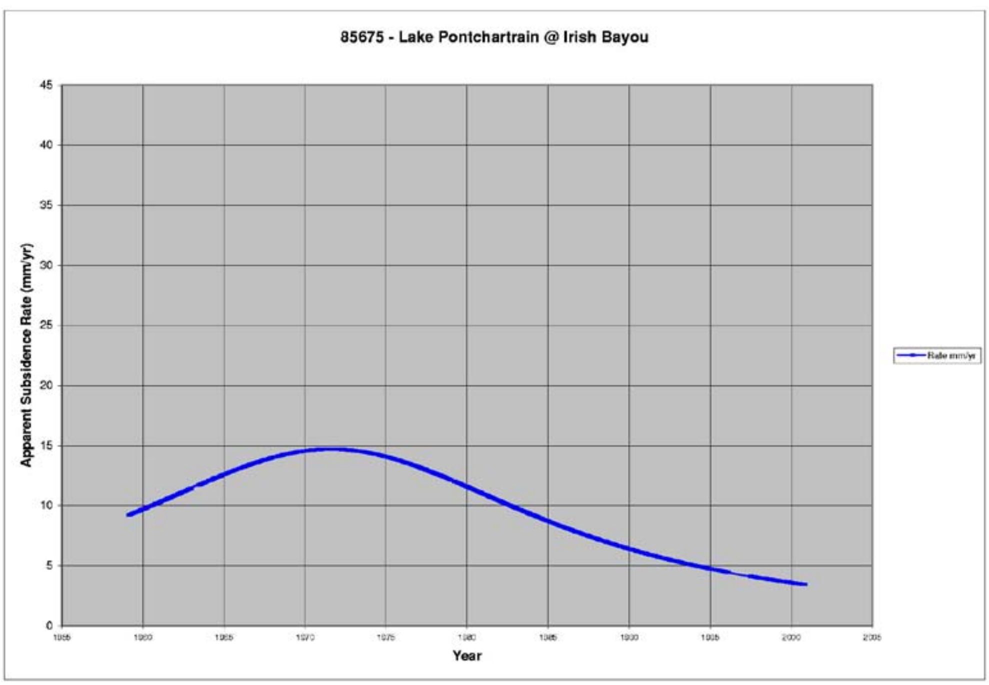
f'(x) = 0.0941859/(0.0183332*(x-1969.704684)^2+1)

Figure L-22. Estimated instantaneous rate of apparent subsidence for Gage 76060 IHNC @ Seabrook Bridge.



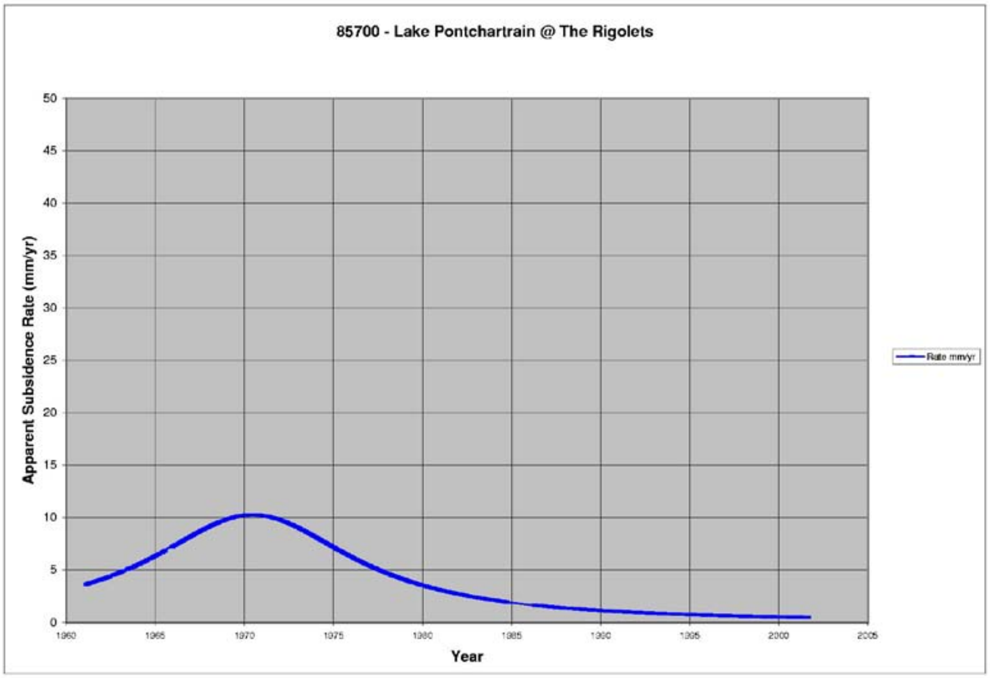
f'(x) = 0.0537708/(0.00499835*(x-1971.786335)^2+1)

Figure L-23. Estimated instantaneous rate of apparent subsidence for Gage 76120 IHNC @ Florida Avenue Bridge.



f'(x) = 0.0482335/(0.00382505*(x-1971.490714)^2+1)

Figure L-24. Estimated instantaneous rate of apparent subsidence for Gage 85675 Lake Pontchartrain @ Irish Bayou.



f'(x) = 0.0337307/(0.0208121*(x-1970.363367)^2+1)

Figure L-25. Estimated instantaneous rate of apparent subsidence for Gage 85700 Lake Pontchartrain @ The Rigolets.

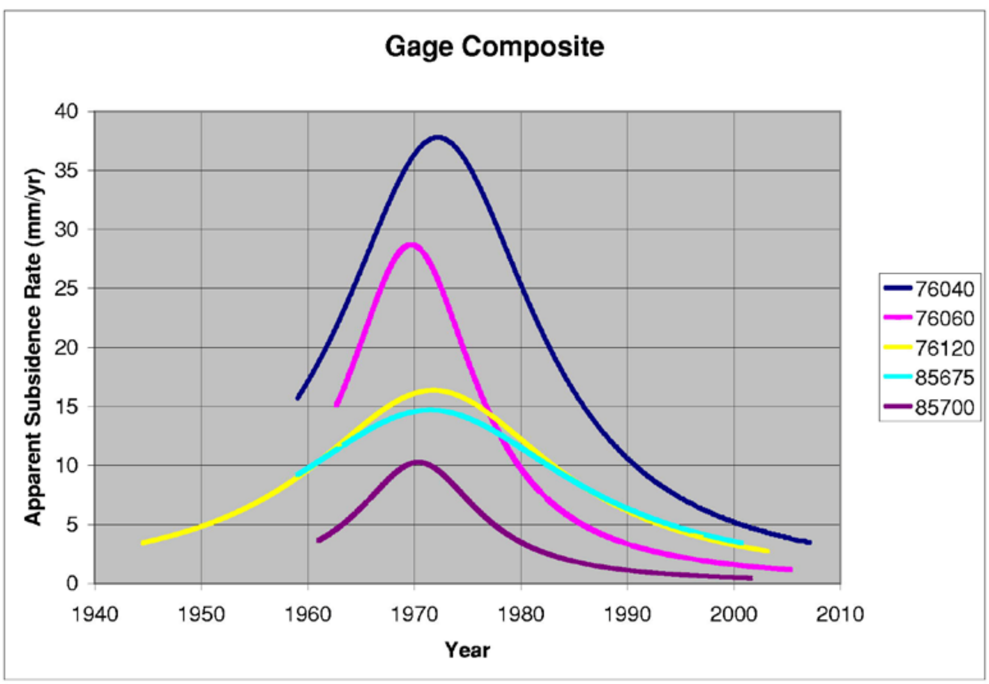


Figure L-26. Composite graph of estimated instantaneous rate of apparent subsidence for all gages.

a. While peak apparent subsidence rates seem to occur nearly simultaneously at all five gage sites, the peak rates themselves vary considerably:

76040 has a peak rate of 37.8 mm/yr at 1972.2
76060 has a peak rate of 28.7 mm/yr at 1969.7
76120 has a peak rate of 16.4 mm/yr at 1971.8
85675 has a peak rate of 14.7 mm/yr at 1971.5
85700 has a peak rate of 10.3 mm/yr at 1970.4

The 76120 rate curve (that is remarkably similar to the 85675 curve) indicates that apparent subsidence at that gage in the 1940s was minimal (± 3 mm/yr) and increased relatively gradually through the 1950s. All of the curves indicate that at some time in the early 1960s rates increased more rapidly, peaking around 1970 to 1972. Thereafter, all of the curves indicate that rates then decreased rapidly over the next ten years and began to flatten out again. All of the curves appear to be converging on a rate of around 2 to 4 mm/yr by 2005.

b. It may be useful to note that the term "apparent subsidence" is simply another way of saying "relative sea level rise." Both are terms used to indicate the combined effect of both *actual* eustatic sea level rise and *actual* subsidence (Stolz and Gill, 2005). Therefore, if all of the above apparent subsidence rate curves are reduced by that portion of the rate thought to be due to eustatic sea level rise (estimated to be 2 mm/yr), the resulting rates should serve as reasonable estimates of *actual subsidence*. The immediate significance here is that the convergence of the apparent subsidence rates in 2005 to 2 to 4 mm/yr suggests that, in fact, the present-day actual subsidence at these gage sites may be negligible (0 to 2 mm/yr).

c. This apparent convergence to zero may be to some extent an artifact of the selected regression function (the arctangent function seeks horizontal asymptotes) and, therefore, overly optimistic. In any event, it would not be wise to use this or any other function fit to the historic stage data as a reliable predictor of future subsidence rates.

L-8. Geodetic – Tidal Datum Comparison. Most of the USACE gage data sets have been connected to one or more nearby permanent bench marks in the national geodetic vertical control network maintained by the NGS. A complete discussion of the history, methods of survey, computation, and adjustment of the elevations of benchmarks in this network is beyond the scope of this work. It is worth noting that a classical vertical datum (e.g., NGVD29 or NAVD88 prior to the advent of Vertical Time Dependant Positioning or VTDP) is, in essence and in the most practical sense for the end user, the physical bench marks and their associated/published elevations.

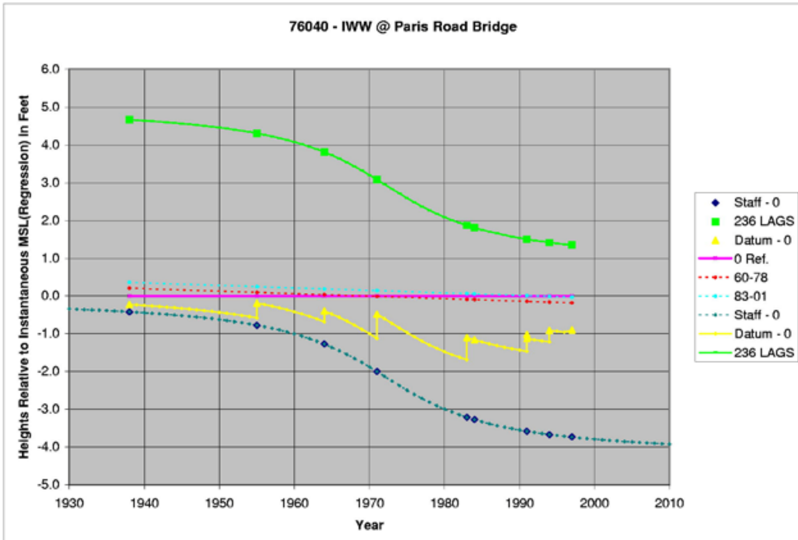
a. Implicit in this statement is the fact that the final adjusted elevations computed for the physical benchmarks - being influenced by a variety of field and computational factors (choice of control constraints, adjustment methods, classical field survey measurement methods and uncertainties, etc.) - define a network-wide "zero" surface that is irregular and does not precisely coincide with any equipotential surface. It is not, in an absolute geodetic sense, truly or permanently "fixed." It is, in a sense, malleable, meaning that, as the benchmarks go, so goes the “zero” surface. As some benchmarks are displaced downward by subsidence and others upward by glacial rebound, for example, so too is the “zero” surface displaced and warped.

b. The “zero” surface is also malleable in the sense that - as new stations and observations are added to the network and other are destroyed or eliminated, and/or as a consequence of the displacements described above – periodic “epoch updates” may be computed for a particular region or portion of an existing vertical datum or a wholesale network-wide re-adjustment may be computed to establish an entirely new vertical datum. An epoch update and/or network-wide datum readjustment essentially results in an entirely new “zero” surface. It may be very similar to the previous “zero” surface in an absolute sense, but it is not precisely coextensive with it (Zilkoski, Richards and Young, 1992).

c. With the advent of GPS, the CORS network,), and more precise geoid models, the problem of defining a truly “fixed” zero reference surface has become more tractable in recent years. However, despite the fact that recent NAVD88 epoch updates in south Louisiana (2004.65 and 2006.81) are based primarily on GPS baseline vector observations, they are still to some degree dependant upon and influenced or otherwise constrained by control point elevations developed conventionally in earlier epoch updates and/or adjustments based on estimated subsidence rates, as well as a geoid model that is subject to further refinement.

d. For these reasons, a means of displaying the relationships among the various geodetic datum epoch “zeros” and the local mean sea level surfaces at each gage location was chosen that may at first seem counter intuitive. The following graphs are based upon the assumption (initially) that the rate of eustatic sea level rise is zero and that the regression curve is the trace of the fixed and unchanging instantaneous mean sea level surface on a gage that is subsiding at the same rate as nearby benchmarks and the land surface to which they and the gage are anchored. By thus defining the regression curve as instantaneous mean sea level (IMSL) or the “fixed” zero reference surface, the vertical displacement of the normalized gage “zero”, the controlling benchmark, and the geodetic datum/epoch “zero” defined by that physical mark may be determined at any moment in time with respect to IMSL.

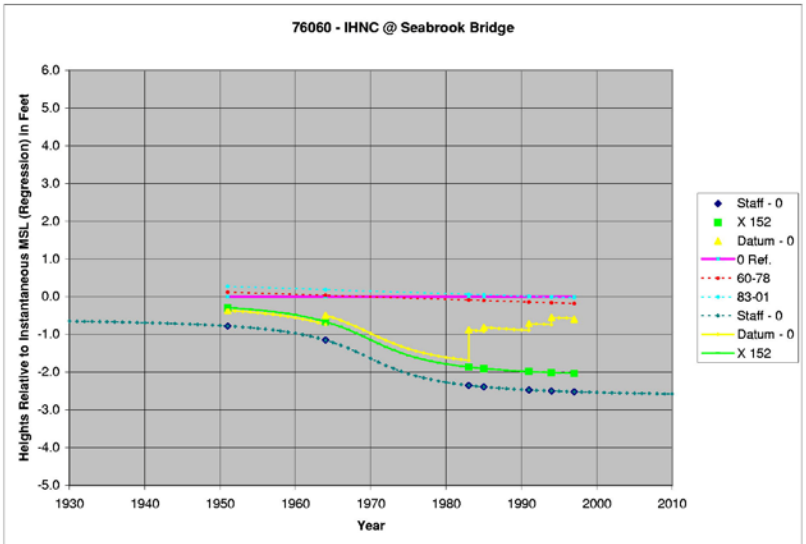
e. Finally, since the rate of eustatic sea level rise is, in fact, estimated to be 2 mm/yr, two lines sloping negatively at the rate of 2 mm/yr (Stolz and Gill, 2005) and intercepting the IMSL zero line at 1969.5 and 1992.5 (these being the respective midpoints of the 1960-1978 and 1983-2001 National Tidal Datum Epochs) have been computed. If the rate estimate for eustatic sea level rise is correct, these lines may be interpreted as the “fixed” tidal datum surfaces corresponding to the height of IMSL at the midpoint of the respective tidal datum epochs (i.e., approximate NTDE MSL). From these lines, estimates of the relationships among the geodetic and tidal datum epochs may be computed (see Figures L-27 through L-30).



NGS Station "236 LAGS"

Year Epoch	IMSL(Reg.) Staff - 0	IMSL(Reg.) 236 LAGS	IMSL(Reg.) Datum - 0	IMSL(Reg.) 0 Ref.	MSL(Reg.) 60-78	MSL(Reg.) 83-01	Stage "MLG'38"	Stage "MSL'38"	NGS Pub Elev.	Vert. Datum
1938	-0.42	4.66	-0.225	0.00	0.207	0.358	1.20	0.42	4.885	"SLD29"
1955	-0.78	4.30	-0.188	0.00	0.095	0.246	1.56	0.78	4.488	"SLD29"
1964	-1.27	3.81	-0.395	0.00	0.036	0.187	2.05	1.27	4.205	"SLD29"
1971	-2.00	3.08	-0.480	0.00	-0.010	0.141	2.78	2.00	3.560	"SLD29"
1983	-3.21	1.87	-1.100	0.00	-0.089	0.062	3.99	3.21	2.970	NGVD29
1984	-3.27	1.81	-1.159	0.00	-0.095	0.056	4.05	3.27	2.969	NGVD29
1991	-3.58	1.50	-1.020	0.00	-0.141	0.010	4.36	3.58	2.520	NGVD29
1991	-3.58	1.50	-1.130	0.00	-0.141	0.010	4.36	3.58	2.630	NAVD88
1994	-3.67	1.41	-0.910	0.00	-0.161	-0.010	4.45	3.67	2.320	NAVD88
1997	-3.73	1.35	-0.900	0.00	-0.180	-0.030	4.51	3.73	2.250	NAVD88

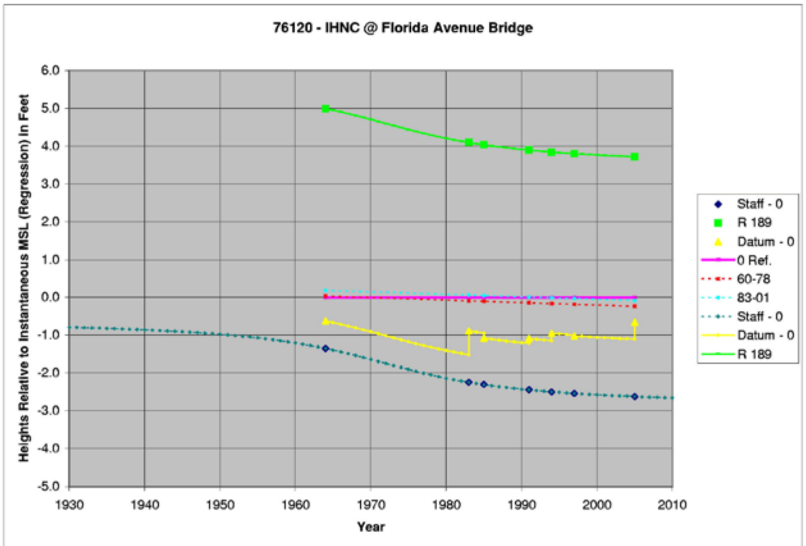
Figure L-27. Estimates of the relationships among the geodetic and tidal datum epochs at Gage 76040 IWW @ Paris Road Bridge.



NGS Station "X 152"

Year Epoch	IMSL(Reg.) Staff - 0	IMSL(Reg.) X 152	IMSL(Reg.) Datum - 0	IMSL(Reg.) 0 Ref.	MSL(Reg.) 60-78	MSL(Reg.) 83-01	Stage "MLG'38"	Stage "MSL'38"	NGS Pub Elev.	Vert. Datum
1951	-0.78	-0.29	-0.363	0.00	0.121	0.272	1.56	0.78	0.072	“SLD29”
1964	-1.15	-0.66	-0.490	0.00	0.036	0.187	1.93	1.15	-0.171	“SLD29”
1983	-2.35	-1.86	-0.880	0.00	-0.089	0.062	3.13	2.35	-0.981	NGVD29
1985	-2.39	-1.90	-0.812	0.00	-0.102	0.049	3.17	2.39	-1.089	NGVD29
1991	-2.47	-1.98	-0.711	0.00	-0.141	0.010	3.25	2.47	-1.270	NGVD29
1994	-2.50	-2.01	-0.551	0.00	-0.161	-0.010	3.28	2.50	-1.460	NAVD88
1997	-2.52	-2.03	-0.591	0.00	-0.180	-0.030	3.30	2.52	-1.440	NAVD88

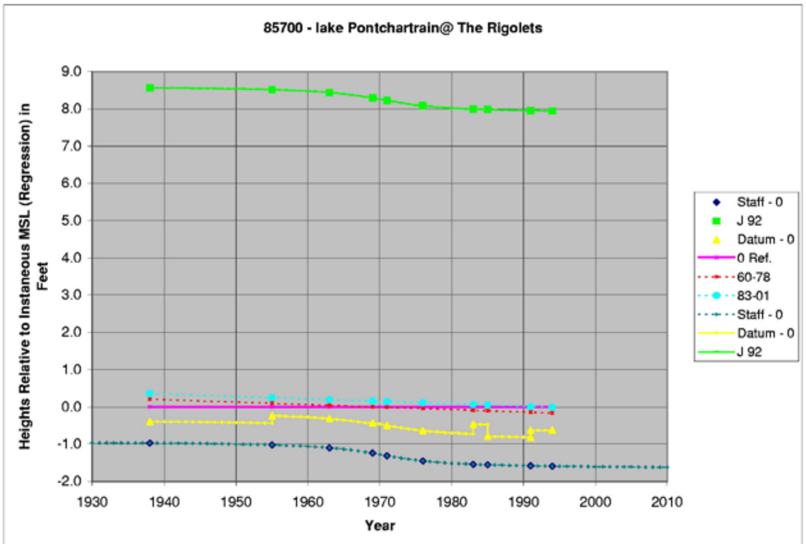
Figure L-28. Estimates of the relationships among the geodetic and tidal datum epochs at Gage 76060 IHNC @ Seabrook Bridge.



NGS Station "R 189"

Year Epoch	IMSL(Reg.) Staff - 0	IMSL(Reg.) R 189	IMSL(Reg.) Datum - 0	IMSL(Reg.) 0 Ref.	MSL(Reg.) 60-78	MSL(Reg.) 83-01	Stage "MSL-20.55"	Stage "MSL"	NGS Pub Elev.	Vert. Datum
1964	-1.35	4.99	-0.620	0.00	0.036	0.187	21.90	1.35	5.610	“SLD29”
1983	-2.24	4.10	-0.883	0.00	-0.089	0.062	22.79	2.24	4.983	NGVD29
1985	-2.30	4.04	-1.075	0.00	-0.102	0.049	22.85	2.30	5.115	NGVD29
1991	-2.44	3.90	-1.090	0.00	-0.141	0.010	22.99	2.44	4.990	NGVD29
1994	-2.50	3.84	-0.950	0.00	-0.161	-0.010	23.05	2.50	4.790	NAVD88
1997	-2.54	3.80	-1.020	0.00	-0.180	-0.030	23.09	2.54	4.820	NAVD88
2005	-2.62	3.72	-0.650	0.00	-0.233	-0.082	23.17	2.62	4.370	NAVD88

Figure L-29. Estimates of the relationships among the geodetic and tidal datum epochs at Gage 76120 IHNC @ Florida Avenue Bridge.



NGS Station "J 92"

Year Epoch	IMSL(Reg.) Staff - 0	IMSL(Reg.) J 92	IMSL(Reg.) Datum - 0	IMSL(Reg.) 0 Ref.	MSL(Reg.) 60-78	MSL(Reg.) 83-01	Stage "MSL-10.00"	Stage "MSL"	NGS Pub Elev.	Vert. Datum
1938	-0.97	8.56	-0.393	0.00	0.207	0.358	10.97	0.97	8.953	“SLD29”
1955	-1.02	8.51	-0.237	0.00	0.095	0.246	11.02	1.02	8.747	“SLD29”
1963	-1.10	8.43	-0.317	0.00	0.043	0.194	11.10	1.10	8.747	“SLD29”
1969	-1.24	8.29	-0.430	0.00	0.003	0.154	11.24	1.24	8.720	“SLD29”
1971	-1.31	8.22	-0.500	0.00	-0.010	0.141	11.31	1.31	8.720	“SLD29”
1976	-1.45	8.08	-0.640	0.00	-0.043	0.108	11.45	1.45	8.720	NGVD29
1983	-1.54	7.99	-0.467	0.00	-0.089	0.062	11.54	1.54	8.457	NGVD29
1985	-1.55	7.98	-0.790	0.00	-0.102	0.049	11.55	1.55	8.770	NGVD29
1991	-1.58	7.95	-0.820	0.00	-0.141	0.010	11.58	1.58	8.770	NGVD29
1991	-1.58	7.95	-0.630	0.00	-0.141	0.010	11.58	1.58	8.580	NAVD88
1994	-1.59	7.94	-0.620	0.00	-0.161	-0.010	11.59	1.59	8.560	NAVD88

Figure L-30. Estimates of the relationships among the geodetic and tidal datum epochs at Gage 85700 Lake Pontchartrain @ The Rigolets.

f. From the perspective of these charts (that again assume that the stage regression lines represent the best approximation of instantaneous mean sea level for any moment in time), one can determine that geodetic datum “zero” for a particular vertical control point at a given gage site has changed over time and at each epoch update and new datum adjustment (i.e., the change from MSL29/ NGVD29 to NAVD88 in 1991) with respect to IMSL and the estimated NTDE MSL values.

g. A chart showing heights relative to IMSL for “85675 – Lake Pontchartrain at Irish Bayou” has not been produced since sufficient documentation of the relationship of the gage zero to a benchmark in the national geodetic vertical network has not yet been obtained. It should also be pointed out that any comparison made to IMSL prior to 1960 for gages 76040, 76060 and 85700 is based upon an extrapolation of the regression curve.

h. As a final effort to clearly convey the apparent relationship among the various geodetic and tidal datum epochs at each gage, graphs that may be more conventional or familiar –

indicating the height differences of IMSL, NTDE'60-'79 MSL and NTDE'83-'01 MSL with respect to the geodetic datum/epoch "zeros" – are shown below (Figures L-31 through L-33, Tables L-2 through L-5).

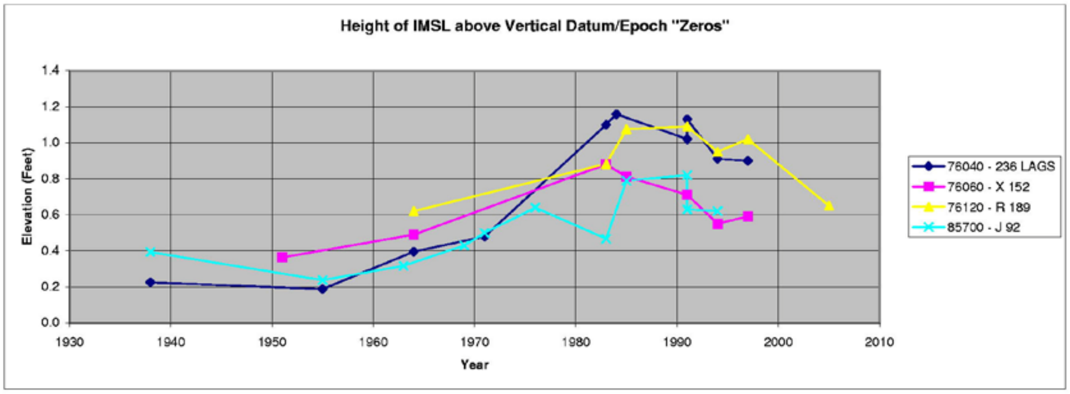


Figure L-31. Elevation of local instantaneous mean sea level with respect to geodetic datum zero as defined by referenced control monuments and associated elevations for given datum and epoch identified in Tables L-2 though L-5.

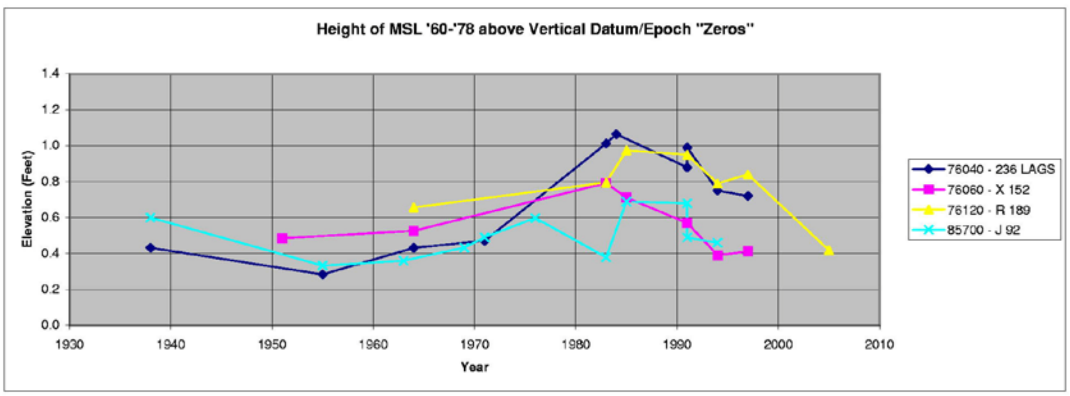


Figure L-32. Elevation of local mean sea level (NTDE 60-78) with respect to geodetic datum zero as defined by referenced control monuments and associated elevations for given datum and epoch identified in Tables L-2 though L-5.

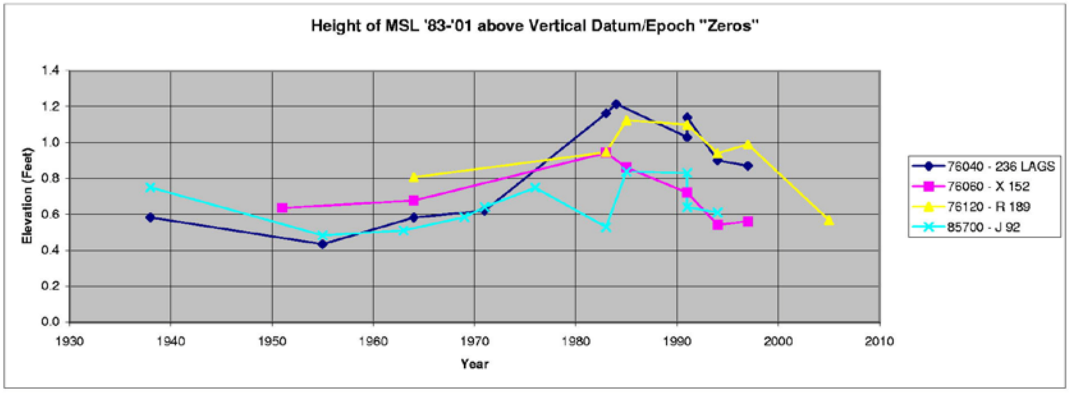


Figure L-33. Elevation of local mean sea level (NTDE 83-01) with respect to geodetic datum zero as defined by referenced control monuments and associated elevations for given datum and epoch identified in Tables L-2 though L-5.

Table L-2. 76040--NGS Station "236 LAGS."

Height (in feet) of IMSL and MSL for 60-78 and 83-01 epochs above various Geodetic Vertical Datum/Epoch "Zeros" (Assumes Eustatic Sea-Level rise of 0.00656 ft/yr or 2 mm/yr)

Vert. Datum	Year Epoch	IMSL(Reg.) 0 Ref.	IMSL(Reg.) 60-78	IMSL(Reg.) 83-01
“SLD29”	1938	0.225	0.432	0.583
“SLD29”	1955	0.188	0.283	0.434
“SLD29”	1964	0.395	0.431	0.582
“SLD29”	1971	0.480	0.470	0.621
NGVD29	1983	1.100	1.011	1.162
NGVD29	1984	1.159	1.064	1.215
NGVD29	1991	1.020	0.879	1.030
NAVD88	1991	1.130	0.989	1.140
NAVD88	1994	0.910	0.749	0.900
NAVD88	1997	0.900	0.720	0.870

Table L-3. 76060--NGS Station "X 152."

Height (in feet) of IMSL and MSL for 60-78 and 83-01 epochs above various Geodetic Vertical Datum/Epoch "Zeros" (Assumes Eustatic Sea-Level rise of 0.00656 ft/yr or 2 mm/yr)

Vert. Datum	Year Epoch	IMSL(Reg.) 0 Ref.	IMSL(Reg.) 60-78	IMSL(Reg.) 83-01
“SLD29”	1951	0.363	0.484	0.635
“SLD29”	1964	0.490	0.526	0.677
NGVD29	1983	0.880	0.791	0.942
NGVD29	1985	0.812	0.710	0.861
NGVD29	1991	0.711	0.570	0.721
NAVD88	1994	0.551	0.390	0.541
NAVD88	1997	0.591	0.411	0.561

Table L-4. 76120--NGS Station "R 189."

Height (in feet) of IMSL and MSL for 60-78 and 83-01 epochs above various Geodetic Vertical Datum/Epoch "Zeros" (Assumes Eustatic Sea-Level rise of 0.00656 ft/yr or 2 mm/yr)

Vert. Datum	Year Epoch	IMSL(Reg.) 0 Ref.	IMSL(Reg.) 60-78	IMSL(Reg.) 83-01
“SLD29”	1964	0.620	0.656	0.807
NGVD29	1983	0.883	0.794	0.945
NGVD29	1985	1.075	0.973	1.124
NGVD29	1991	1.090	0.949	1.100
NAVD88	1994	0.950	0.789	0.940
NAVD88	1997	1.020	0.840	0.990
NAVD88	2005	0.650	0.417	0.568

Table L-5. 85700--NGS Station "J 92."

Height (in feet) of IMSL and MSL for 60-78 and 83-01 epochs above various Geodetic Vertical Datum/Epoch "Zeros"
(Assumes Eustatic Sea-Level rise of 0.00656 ft/yr or 2 mm/yr)

Vert. Datum	Year Epoch	IMSL(Reg.) 0 Ref.	IMSL(Reg.) 60-78	IMSL(Reg.) 83-01
“SLD29”	1938	0.393	0.600	0.751
“SLD29”	1955	0.237	0.332	0.483
“SLD29”	1963	0.317	0.360	0.511
“SLD29”	1969	0.430	0.433	0.584
“SLD29”	1971	0.500	0.490	0.641
NGVD29	1976	0.640	0.597	0.748
NGVD29	1983	0.467	0.378	0.529
NGVD29	1985	0.790	0.688	0.839
NGVD29	1991	0.820	0.679	0.830
NAVD88	1991	0.630	0.489	0.640
NAVD88	1994	0.620	0.459	0.610

L-9. Conclusion. When properly normalized, useful historic subsidence rate data may be extracted from the 8 A.M. daily stages recorded at USACE gages throughout coastal Louisiana. This data may also be helpful in assessing how well previous geodetic vertical datum epoch updates or readjustments by NGS responded to the local effects of subsidence. Finally, south Louisiana appears to be in a region of spatio-temporally varying subsidence. Consequently, straight-line regression applied to stage data recorded at the various USACE gages in this study appears to mask significant subsidence rate variability and captures neither the fact that rates significantly higher than the gage-specific average appear to have occurred during the 1960s and 1970s, nor that these rates appear to have abated to well below the average in recent years.

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APPENDIX M

Uncertainty Model for Orthometric, Tidal, and Hydraulic Datums for use in Risk Assessment Models

M-1. Purpose. This appendix contains excerpted portions from a technical report on datum uncertainty models performed by the Conrad Blucher Institute for Surveying and Science, Texas A&M University (Corpus Christi). The uncertainty study was performed for the USACE Vertical Project Control Delivery Team in 2009. The scope of the study is outlined in paragraph M-2 below. The unedited version of the study report (*Uncertainty Model for Orthometric, Tidal, and Hydraulic Datums for Use in Risk Assessment Models, Phase 2 Final Report, dated 3 Sep 09*) can be obtained at the AGC web site referenced in Chapter 1. References cited in this appendix are listed in Section M-15 (Addendum D to this Appendix).

M-2. Executive Summary. This report is the second in a series requested by the U.S. Army Corps of Engineers (USACE) to assess reference datum accuracy requirements that are currently in place, and to establish whether the USACE is able to perform datum reliable uncertainty analyses to ascertain the risks of project failure. The USACE contracted with the Conrad Blucher Institute for Surveying and Science at Texas A & M University-Corpus Christi to produce these reports under the leadership of Dr. Gary Jeffress, Executive Director.

a. This study phase offers a technical discussion of risk assessment, specifically regarding to relevant orthometric and water level datums and datum conversion for use in protection grade design, and discussion of a suggested approach to integrating vertical uncertainty into future USACE project risk assessments.

b. Findings include an analysis of existing risk assessment guidelines within USACE, as well as a statistical discussion of perceived risk versus actual risk. This statistical discussion goes on to compare and quantify accuracy versus uncertainty. Each datum used by the USACE is analyzed for uncertainty and the accompanying risks, including terrestrial datum and water level datum, and datum conversions, such as converting legacy NGVD29 measurements to NAVD88 elevations.

c. The findings reveal that a very limited analysis of risks associated with converting legacy datums to modern datums has been conducted by the USACE, and that these initial studies reveal the complexities involved in the process, as well as a lack of historical data coverage of significant portions of the United States.

d. Finally, after discussions with engineers in the Corps, the study team created a methodology based upon extended discussions with the USACE and surveyors for investigating and incorporating potential sources of vertical inaccuracy into risk assessment models, and designed a project worksheet and flowcharts to aid in this process. A sample application of this worksheet demonstrating how this could be implemented in future USACE projects is included in this report (see Section M-12--Addendum A to this Appendix). The USACE has adopted a vertical accuracy standard of ± 0.25 feet at the 95% confidence level for the connection of

USACE projects to the National Spatial Reference System (NSRS) maintained by the National Oceanic and Atmospheric Administration. While this standard is very acceptable on a national scale, the standard should be taken under review as changes occur to the NSRS in the future.

M-3. Participants.

Name	Agency	Role
Dr. Gary Jeffress	Texas A&M University-Corpus Christi	Team Leader, Surveyor, and Geodesist
Dr. Edward Jones	Risk Assessment Consultant	Statistics and Risk Assessment
Jennifer Bray	Texas A&M University-Corpus Christi	Technical Writer
Adre Deetlefs	Texas A&M University Corpus Christi	Graduate Research Assistant

M-4. Purpose of Study. From Cooperative Agreement W912HZ-8-0012, "Scope of Work," dated 29 Oct. 2008:

The study team, utilizing statistical data from existing National Oceanic and Atmospheric Administration (NOAA) geospatial databases – Coastal Survey Development Lab/VDatum (CSDL), National Geodetic Survey (NGS)/National Spatial Reference System (NSRS), and Center for Operation Oceanographic Products and Services (CO-OPS) – will develop an uncertainty model for the relevant orthometric, tidal, and hydraulic datums for use in protection grade design and risk assessment models.

Uncertainties in geodetic, topographic, or hydrographic survey measurement systems and dynamic sensor errors are not part of this study since these uncertainties should be available from NOAA, or have been indirectly incorporated into existing products, such as the NSRS.

SECTION 1

A Theoretical Framework for Modeling Elevation Uncertainty

M-5. An Overview of Current Risk Assessment Guidelines within the Corps of Engineers.

a. Overview of EM 1110-2-1619 (Risk-Based Analysis for Flood Damage Reduction Studies). The terms risk and risk assessment are used in many ways. The general public has a definition of risk that can be described as perceived risk. Perceived risk is the public perception of the risk of a specific flooding event. Civil engineers, on the other hand, define risk as a probability determined from flood event related data, such as storm discharge, and from flood risk management measures. The purpose of this report is to integrate expanding knowledge of vertical reference datums and their uncertainty with current engineering approaches to risk assessment.

(1) A formal, or civil engineering, definition of risk is based upon probability and scientific analysis of factors contributing to the flooding event as well as the efficacy of flood risk management measures. Bulletin #17B (1981) *Guidelines for Determining Flood Flow Frequency*, published by the Federal Hydrology Subcommittee states:

> "*…risk is defined as the probability that one or more events will exceed a given flood magnitude within a specified period of years.*"

(2) In EM 1110-2-1619 (1996) *Risk-Based Analysis for Flood Damage Reduction Studies* the USACE defines long-term risk in a similar way:

> "*The probability of capacity exceedance during a specified period. For example, 30-year risk refers to the probability of one or more exceedances of the capacity of a measure during a 30-year period*."

(3) This report formalizes this definition using the following notation. If E is a defined flooding event and Y is a specified time period, the risk of one or more flooding event during this period is defined as the conditional probability: $P_Y = P(E > 0 \mid Y)$. If $P_1 = P(E \mid 1)$ is the risk of flooding for a single year and if this is assumed to be independent and unvarying from year to year then it follows that $P_Y = 1 - (1 - P_1)^Y$. Independence of flooding from year to year is typically assumed when little historical data is available.

(4) The same engineering manual goes on to describe the traditional approach to dealing with uncertainty in design by incorporating somewhat arbitrary safety factors into flood risk management plans:

(5) EM 1110-2-1619 also offers a more quantitative approach to assessing risk using quantifiable uncertainty:

"Quantitative risk analysis describes the uncertainties, and permits evaluation of their impact."

b. Statistical Considerations. As described above, there are two definitions of risk: perceived versus actual. This perception is formed based upon recent reporting of the same or similar events. Although perceived risk can have little relationship to more formal risk assessments, it can play an important role in public support of flood risk management.

(1) Actual risk, as defined by engineers, is a probability calculation P_Y. There are many factors that can cause perceived and actual risks to diverge. One potential cause is stating risk as the probability of a flood in a single year, P_1. For example if the probability of a defined flooding event is 1/100, 1 in 100 years, then the probability of one or more floods during a 50-year period is:

$$P_{50} = P(E > 0 \mid 50) = 1 - (1 - 0.01)^{50} = 0.39$$

If this were a flood risk management project with a life expectancy of 50 years, we would say that there is a 39% chance of seeing one or more of these flood events during the life expectancy of this project. Stating that there is only a 1% chance of a flood each year is correct, but understates the risk over the project lifespan.

(2) The relationship between risk and project lifespan is illustrated in Figure M-1 for P_1= 1/100:

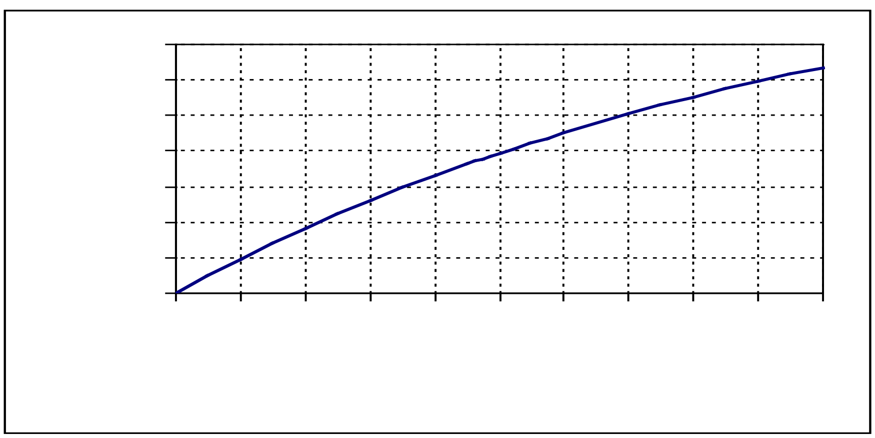

Figure M-1. Risk vs. Project Lifespan for P_1 = 1/100.

This calculation of risk versus project lifespan using the annual probability of flooding can be reversed. The probability of flooding in a single year, P_1, is determined from the distribution of flood events together with the probability that the flood risk management measures control these events. Sometimes it is easier to design a structure using the annual probability of flooding.

(3) The question is what value of P_1, the annual probability of flooding, is needed to provide a stated risk, R over the project lifespan, Y. This is answered by solving for P_1 in the equation $R = 1-(1-P_1)^Y$, yielding the relationship:

$$P_1 = 1 - e^{\left(\ln(1-R)/Y\right)}$$

For example, if we required a risk of less than ten percent for a 50-year project lifespan, the flood risk management project would need to be designed to reduce the probability of flooding in a single year to 2/1000:

$$P_1 = 1 - e^{\left(\ln(1-0.1)/50\right)} = 0.002$$

For small values of R and large values of Y, $R \leq 1/10$ and $Y \geq 50$, this calculation is approximately equal to the ratio R/Y.

c. Quantifying Uncertainty - Accuracy vs. Precision. Uncertainty in vertical measurements arises because of several factors, some involving measurement error and others arising from bias or calibration issues. Measurement error, also referred to as measurement precision, occurs from factors that cause variation in vertical measurements that cannot be entirely controlled. Field surveys, for example, are expected to have some small measurement errors due to instrument inaccuracies as well as small variations in field procedures and environments. Measurements from tidal gages exhibit short term variation from predicted tides due to meteorological changes.

(1) Bias or calibration errors are systematic deviations of the true elevation from the measured elevation. A bench mark, for example, may have been subject to land subsidence making the actual height of the bench mark lower than the bench mark's stated height. Although survey measures using this bench mark might contain only small measurement errors, they will yield heights on newly surveyed points that will all be systematically computed with values higher than their actual height. Similarly, if the realization of a vertical datum in an area is systematically lower or higher than truth, then this causes bias in other marks.

(2) The accuracy of a vertical or horizontal measurement is a function of both measurement bias and precision. Formally, accuracy is defined as the expectation of the square of the difference between the measurement M and the true value T:

$$EMSE = E(M-T)^2 = (E(M)-T)^2 + \sigma^2 = Bias^2 + Var(M).$$

In this statistical formulation, $E(M)$ is the expected value for M, the measurement. The expected deviation of the measurement from its true value, $E(M-T)^2$, is commonly referred to as the expected mean squared error, or EMSE, for the measurement. The squared bias, $(E(M)-T)^2$, is a consequence of calibration issues and systematic differences in the datum realization. The variance, σ^2, is the measurement error or precision associated with the measurement.

(3) In many cases, the variance or precision of a vertical measurement is much smaller than the bias. In practice, the systematic deviation of a measurement from the reference system is several times larger than the measurement precision. However, risk assessments for some flood risk management projects should include all sources contributing to vertical measurement accuracy over the life expectancy of the project.

M-6. Modeling Elevation Uncertainty--Major Factors Contributing to Elevation Uncertainty. The risk of a flooding event depends upon both the probability of a flood event and the probability that it exceeds the flood risk management measures. Bulletin 17B (1981) sets general guidelines for modeling the flood event distribution using historical gage data. In most flood risk management projects, the probability that a flood event exceeds the flood risk management measures depends upon several vertical measurements. For most flood reduction projects, the accuracy of these measurements is critical. Figure M-2 illustrates a general process for investigating and assessing the impacts of elevation measurement inaccuracy in risk modeling. The major sources of inaccuracy can be organized into three major categories: (1) Terrestrial or Orthometric Elevations, (2) Water Level Elevations, both tidal and non-tidal, and (3) Datum Realizations and Conversions from legacy datum to current project datum.

Figure M-2. Major Sources of Elevation Inaccuracies.

In some flood risk reduction projects, it is necessary to evaluate the elevation inaccuracies arising from all three areas. Other projects may require attention to only one area, such as datum conversion. Any inaccuracy concerns can be further described in terms of a bias and variance factor, as described above. In most cases, the variance is solely considered, such as for the inaccuracies associated with datum conversions. In some cases, such as land subsidence and rebound, the estimated bias, or offset, over the project lifespan becomes important. Once these inaccuracies are summarized in terms of bias and variance errors, the overall inaccuracy in elevations can be expressed using the expected mean squared error, *EMSE* calculation described above. This in turn should be incorporated into elevations used in engineering risk modeling for a project.

a. Terrestrial Elevations. A terrestrial datum (zero elevation) establishes the standard used for terrestrial leveling such as North American Vertical Datum of 1988 (NAVD88) and its earlier cousin National Geodetic Vertical Datum of 1929 (NGVD29). Onsite vertical measurements are made relative to a fixed point (height of the primary tidal benchmark at Father Point/Rimouski, Quebec, Canada for NAVD88) and tied to local elevation networks as needed. Vertical measurements are subject to measurement errors and bias. In some cases these errors are a result of well established and measured factors, such as leveling measurement precision defined by the quality of the instrument and its correct use, and in others they result from known and difficult to quantify factors such as insufficient coverage of known elevations in the project area.

(1) Subsidence and Rebound. Land subsidence is common along the Gulf coast of the United States, whereas land rebound is common along the coast of Alaska. The combined effects of land subsidence, or isostatic rebound, coupled with sea level changes are referred to as relative sea level rise or fall. This can be a large source of elevation errors. Worse still, relative sea level changes results in elevation bias, bias in which most assumed project elevations are too high or too low. In some cases it is possible to separate land subsidence or rebound from changing sea levels. However, in many cases there are insufficient data to separate the two effects. In engineering risk assessments, subsidence and rebound cause elevations to change relative to the intended datum over the project lifespan. This can be a critical consideration. In many areas, the effects of relative sea level change can be estimated over the life expectancy of the project. In Galveston, Texas, for example, historical data at the Galveston Pier 21 tide gage indicated an average relative sea level rise of approximately 0.21 feet every decade. For a project with a life expectancy of 50 years, this results in a relative sea level rise of approximately one foot.

(2) Elevation Measurements. Elevation measurements can consist of both bias and measurement error. Bias in leveling can occur when established surveying standards are not followed, instruments are not properly calibrated, or when measurements are not properly adjusted for ambient conditions. When chart or topographic elevations are used as project elevations, a bias can occur when the charts are out-of-date. The result is a set of measurements that can be precise, but which are all shifted too high or too low.

(a) Measurement errors in leveling can generally be controlled to small tolerances when established surveying standards are followed. In fact it is common practice to record measurement deviations in closed-loop surveys in the survey log. Closed-loop survey deviations are required to be smaller than the values shown in Table M-1 to ensure the precision of the measurements are within those tolerances. The a priori standard errors of 1 km of single-run leveling for first-, second-, and third-order leveling used by NGS to incorporate data into the NSRS follow in Table M-1:

Table M-1. Allowable Leveling Misclosures. (from Zilkoski, et al (1992)).

first-order, class 0	*= 0.7 mm*
first-order, class I	*= 1.1 mm*
first-order, class II	*= 1.4 mm*
second-order, class I	*= 2.1 mm*
second-order, class II	*= 2.8 mm*
second-order, class 0	*= 3.0 mm*
third-order	*= 4.2 mm*

The estimates of these standard errors were empirically determined in the late 1970s using the limited amount of data available in computer-readable form at the time the analysis was performed.

(b) It should be noted, however, that survey tolerances used in closed-loop surveys only control measurement error, not measurement bias. In practice, leveling bias is assumed to be small enough to ignore since it would arise from unobserved errors such as relying upon only one faulty monument to adjust the survey observations. If a faulty monument is used, it will bias all recorded vertical elevations by the amount its assumed elevation deviates from its true elevation. This bias is overcome by the USACE requirement to have a minimum of three control monuments for each project, although this requirement will not alleviate broadly occurring subsidence that affects all three monuments.

(c) In most projects, following established surveying standards ensures that measurement error and bias in leveling elevations are small enough to be ignored. Good leveling quality control measures (such as archiving and checking survey log computations) are used to validate these assumptions.

(3) Datum Coverage. Errors in conversion can occur for inadequate or insufficient coverage by one or more of the reference datums. This can result in datum conversion bias on the order of several feet in some extreme cases, for example in some projects involving old legacy datums. In instances when understanding risk and uncertainty in elevation is critical, direct measurements should be used to ensure accurate datum conversion. Conversion from one datum in the project area to another is common. For example this can occur when older elevations, such as NGVD29, are converted to NAVD88 and used in project construction. The accuracy of datum conversions relies on assumed datum accuracy in the project area, and although one datum may be accurate the other may not. Software such as VERTCON and CORPSCON readily supply a conversion from NGVD29 to NAVD88 and vice versa.

(4) National Spatial Reference System Adjustment. In order to ensure consistency between various vertical measurement surveys, the USACE requires all project vertical networks be tied

to the NSRS. This ensures vertical measurement consistency over larger areas and allows planners and engineers to answer larger area flood event questions. These might occur, for example, from unexpected large-scale disasters, such as a dam failure or other, larger than expected, flood events. Conversion of elevations on existing project datums to the current NSRS datum can be inaccurate. Datum conversions, especially vertical datums, are based on models, which may not be reliable. While care is taken to use properly qualified monuments in this conversion, larger projects may have coverage gaps resulting in inaccurate conversions.

b. Water Level Datum. Water level datums can be tidal or non-tidal. A tidal datum consists of lunar-solar tide-affected water level measurements at tide gages such as mean sea level (MSL) or Mean Lower Low Water (MLLW). A non-tidal datum consists of local water level measurements (on bodies of water no measurably affected by lunar-solar tides, like lakes or rivers) such as a low water reference plane (LWRP). Uncertainties in tidal and non-tidal datum realizations can occur from several sources, including gage accuracy and placement. These should also be considered in the total elevation uncertainty budget.

(1) Tidal Elevations. Tidal datums stem from measurements of sea level, and in the case of the Great Lakes, lake levels, at long term tidal gages. Measurements at these gages are averaged over an epoch, a 19 year period, to obtain local mean sea level (LMSL) and other tidal datum at a particular location (see Section M-13--Addendum B to this Appendix). The accuracy of these datums is important to projects near the coasts. The accuracy is influenced by several factors, including gage construction and age, the location of the gage relative to the project area, the historical record from that gage, and any relative sea level change in that area. For short-term station observations (few months to a few years), NOAA uses a method of simultaneous comparison with a National Water Level Observation Network (NWLON) station to determine equivalent 19 National Tidal Datum Epoch (NTDE) values (accuracies are discussed later. See Addendum B to this Appendix at Section M-13.

(a) Sea Level Change. Relative sea level changes are conceptually easy to monitor and estimate. However, data on *absolute* sea level rise or fall are less common. If relative sea level change is the only data available, then its total effect cannot be easily split between absolute sea level change and land subsidence and rebound. However, for assessing the flood damage risks in coastal areas, only the total *relative* sea level change matters. If historical data for relative sea level change is available it should be documented and incorporated into total elevation inaccuracy expressed as a maximum bias and a variance expected over the project lifespan. The bias is an estimate of the total sea level change. The impact of sea level rise is similar to that of land subsidence. The estimated relative sea level rise should be calculated for the project lifespan and entered as a negative bias (-). Any sea level decrease would be expressed as a positive bias (+). The variance of sea level change is an estimate of the uncertainty in the sea level data and the assumption that future changes in sea levels will continue as seen in the historical data. This variance is the variation of sea levels around the sea level trend, and normally would not be expected to change from one epoch to the next. USACE policy for incorporating sea-level change in project planning and design is contained in USACE EC -1165-2-211, 1 July 2009, *Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil Works Programs.*

(b) Gage Uncertainty. Gage uncertainty has been reduced with the installation of newer digital tide gages by NOS. NOAA's National Ocean Service currently uses a digital acoustic Next Generation Water Level Measurement System (NGWLMS). The NGWLMS water level sensors have an accuracy of about 1.0 cm for each sample (*Schultz, et al., 1998*). However, many gages used by the Corps of Engineers are very old and rely on older technology, which is less reliable and less accurate than newer NOS gages. For projects using a tidal datum, it is important to determine the age of the gage and its accuracy. The age of the gage is normally within the records. Identifying the technology used by the gage and the resulting accuracy might take additional investigation. This information can be used to determine the accuracy of datums used from these historic gages. The IPET study revealed several daily staff reading records from USACE tide staff gages near New Orleans with record lengths of several decades. However the metadata records from which to put the readings precisely onto a common vertical reference datum were not located, thus limiting the accuracy for sea level trends analyses. When tracking historical records using NGVD29 as a reference, the time period of the NGVD29 value was missing; knowing the time period and the NGVD29 adjustment date is often critical to application for sea level trends and datum relationships.

(c) Proximity of Gage. The location of the gage is also important. Ideally it should be relatively close to the project area. If not, the heights relative to the tidal datum can be expected to be less accurate in the project area.

(d) Epoch Adjustment. Typically tidal water levels are computed by averaging observations over a 19 year period, referred to as an epoch. Averages computed in this manner represent the water levels at mid-epoch. If water levels are changing over time, either from land subsidence or rebound, or from sea level rise or fall, then current water levels and levels at the end of a project lifespan must be adjusted for this trend. This adjustment is achieved simply by determining the expected relative sea level change over the project lifespan. Sea levels such as MSL, mean sea level, are adjusted for this trend by adding or subtracting the maximum bias expected for the project lifespan. This can significantly impact assumed vertical measurements. In the presence of relative sea level changes, this can introduce significant bias in vertical measurements. According to Zervas, 2001: "*The standard errors of the calculated trends are found to be inversely related to the time span of the data available. An inverse power relationship was derived empirically by fitting a least squares line to a log-log plot of the standard errors for each station versus the year range of the data. This relationship gives an estimated requirement of 50 to 60 years of data for obtaining linear MSL trends having a 1 mm/yr precision with a 95% statistical confidence.*"

(e) Gage Historical Record. The historical record for a gage is important. Ideally, a complete record over the epoch period is needed to quantify any trend in relative sea level changes. Incomplete records can cause significant error in trend determination relative to current sea levels. Such historical gaps can arise from unexpected events such vessel collisions and significant storm events.

(2) Non-Tidal Elevations. Non-tidal datums stem from measurements at long term stream gages located in rivers, streams, and reservoirs. The accuracy of these datums is important to projects near these locations. The accuracy is influenced by several factors, including gage

construction and age, the location of the gage relative to the project area, the historical record from that gage and any subsidence or rebound in that area.

(a) Low Water Reference Plane. Low water reference planes (LWRP) are established at local stream gages located in rivers, streams, and reservoirs. Like their coastal cousins, these stream gages can be subject to errors and land subsidence or rebound. These should be considered in the calculating an overall adjustment to elevation uncertainty. Errors in the low water reference plane consist of two components: bias and variance. The bias, or systematic errors, only occurs if the gages used to determine the LWRP are subject to land subsidence or rebound, or if the absolute height of the body of water is raised or lowered by design or some non-random affect in the gage design is biasing all of the measurements. The variance, on the other hand, represents the variation of the LWRP extrapolated into the body of water. If gages in a project area, for example, are located a mile apart then the variation of the LWRP over that mile is represented as a variance, which typically would be small in most cases.

(b) Stream Gage Accuracy. The accuracy of the stream gage, its ability to measure flow rates and stage elevation, used to measure the reference plane should also be considered. In some cases the gage may be older or less well maintained. This uncertainty would be represented as an additional variance.

(c) Stream Gage Proximity. The proximity of the stream gage to the project area should be considered for determining the uncertainty of heights in the project. River or stream flow gages that are critical for a project should be located within the project area.

(d) Pool Elevation Record Length. The historical record for a stream gage is important. Incomplete or inaccurate records can cause significant bias in vertical measurements, relative to current water levels. These too should be indicated with a larger variance component in water level elevations.

c. Datum Conversion. Vertical inaccuracy can be caused by datum confusion and conversion problems. Datum confusion occurs when different datums are used on a project without adequate conversions. For example, an engineer might specify a design elevation of 10 feet using NAVD88. During construction, engineers may confuse datums and, for example, refer to an older datum resulting in a construction height of 10 feet using NGVD29. Different datums, such as NAVD88 and NGVD29 can deviate significantly in assumed elevations in many regions of the country. Deviations of 2-3 feet are not unusual. In some areas of the Gulf coast, for example, FEMA requires construction heights above 10 feet to qualify for federal flood insurance. FEMA does not, however, specify which datum should be used in making these measurements. As a result, most surveyors report elevations using NGVD29, which can be numerically greater than the equivalent NAVD88 elevations, giving the illusion of a higher level of protection. See *Elevation Data for Floodplain Mapping,* 2007 published by the Committee on Floodplain Mapping Technologies. Datum conversions can also cause vertical inaccuracy. Typically this results from insufficient coverage for one or more of the reference datums used in the project. Examples of datum conversion issues are described in the following section. Datum conversions are not exact but rely upon unproven models. This can introduce inaccuracy when converting from one datum to another.

SECTION 2

Practical Implications and Examples

M-7. <u>Overview</u>. In the United States, continental geodetic, tidal, and non-tidal datums (including hydraulic) have been around for about two centuries with our greatest knowledge of these datums appearing towards the end of the 20th century. Much of this knowledge came with the advent of computers, precise measurement technologies, and satellite based measurements. While very few engineering projects cover an entire continent (with the exception of transportation and communication systems), most engineering works need homogeneous spatial vertical reference control to manage the flow of stored water, excessive amounts of water, and general surface water runoff in a local and limited geographic area in concert with the natural drainage system.

a. Transportation systems led the need for more expansive homogeneous leveling networks. First was the need for leveling the tidal regime in New York Bay and Hudson River in 1856-57 for maritime purposes. This was followed by the need for leveling to support the construction of railroads, and later roads, both of which covered the entire North American continent. The first comprehensive vertical control network general adjustment resulted in the Sea Level Datum of 1929, which was renamed the National Geodetic Vertical Datum of 1929 (NGVD29) in 1973. This adjustment was constrained to MSL observed at 26 tide gages (21 in the US and five in Canada). It should be noted that the island state and territories of the United States, including American Samoa, Guam, Hawaii, Northern Marianas, Puerto Rico, and the Virgin Islands were never included in NGVD29 or NAVD88. These areas (except Hawaii) have their own local geodetic reference datums. These include, American Samoa Vertical Datum of 2002 (ASVD02), Guam Vertical Datum of 2004 (GUVD04), Northern Marianas Vertical Datum of 2003 (NMVD03), Puerto Rico Vertical Datum of 2002 (PRVD02) and the Virgin Islands Vertical Datum of 2009 (VIVD09).

b. Knowledge obtained since 1929 showed that the deviation of Local Mean Sea Level from the geoid (e.g. "sea surface topography) at the 26 tide gages used as constraints, introduced distortions in the 1929 general adjustment. Combined with other issues (lack of coverage, lack of good surface gravity data, etc.) it became clear that NGVD29 would need to be superseded eventually. Subsequently a re-adjustment of the vertical network, with vastly increased data sets, was undertaken in 1988 and published in 1991 as the North American Vertical Datum 1988 (NAVD88) (Zilkoski, Richards, and Young, 1992). The new NAVD88 adjustment was constrained to LMSL at one tide gage at Father Point/Rimouski, Quebec, Canada. This new vertical datum remains the current official terrestrial vertical datum used by all civil federal mapping authorities throughout the conterminous United States and Alaska. Over 200 modern digital tide gages operated by CO-OPS as part of the National Water Level Observation Network (NWLON), which computes tidal datums now based on the 1983-2001 National Tidal Datum Epoch (NTDE), complement the NAVD88 vertical datum by supplying local tidal datums and local relative sea level trends tied to NAVD88 through leveling. Complementing the NAVD88 and the 200 tide gages is a vertical transformation software tool called VDatum. VDatum translates geospatial data between 36 different vertical reference systems and removes the most serious impediments to data sharing allowing for the easy transformation of elevation data from one vertical datum to another (NOAA, 2009). VDatum allows for the transformation between onshore terrestrial vertical datums, tidal datums, topographic vertical data, and hydrographic depth data. The datum relationships among tidal and geodetic datums are designed to match the relationships at tide stations. Using hydrodynamic tidal models and other interpolation software, VDatum is particularly useful in interpolating between tide stations along the shore and interpolating across the water surfaces of bays and estuaries.

M-8. Examples of Risk Resulting from Elevation/Datum Uncertainty.

a. Hurricane Katrina - New Orleans, 2005. In October 2005 Lieutenant General Carl A. Strock, USACE Chief of Engineers formed the Interagency Performance Evaluation Task Force (IPET) to determine the facts concerning the performance of the New Orleans Hurricane Protection System (HPS) during Hurricane Katrina. The details of the study are reported in nine volumes (USACE, 2008). Risk and uncertainty are included in the scope of this study. The Executive Summary of IPET Volume II – Geodetic Vertical and Water Level Datums --dated March 2007 specifically addresses water level and datums:

> *"A spatial and temporal variation was found to exist between the geodetic datums and the water level reference datums used to define elevations for regional hydrodynamic conditions. This 0.2- to 3.0-ft variation is critical in relating measurements of wave heights and water level elevations, high-resolution hydrodynamic conditions, water elevations of hydrostatic forces and loadings at levees and floodwalls, elevations of pump station inverts, and related elevations of flood inundation models deriving drainage volumes or first-floor elevations in residential areas. Flood control structures in this region were authorized, designed, and numerically modeled relative to a water level reference datum (e.g., mean sea level). However, these structures were constructed relative to a geodetic vertical datum that was incorrectly assumed as being equivalent to, or constantly offset from, a water level datum. These varied datums, coupled with redefinitions and periodic readjustments to account for the high subsidence and sea level variations in this region, significantly complicated the process of obtaining a basic reference elevation for hydrodynamic modeling, risk assessment, and design, construction, and maintenance of flood control and hurricane protection systems."*

b. Risk, research, and change are addressed in Volume I, “Executive Summary and Overview” of the IPET study dated June 2008:

> *"RISK: Understanding risk is a powerful tool in helping both individuals and government agencies to make consistent and conscientious decisions concerning natural hazard risk management. The ability to quantify risk for large geographical areas and complex engineered systems is just emerging through the work in New Orleans and central California. Risk provides a much richer body of knowledge to understand and manage vulnerability to hazards as well as providing a clear common picture of the situation to all. Risk methods for regional infrastructure, if fully developed, will not only allow assessment of multiple hazards, but also allow collective consideration of life safety, direct and indirect economics, and social-cultural issues, enabling customization of solutions to*

situations. But the evolution and application of risk to support decision making must be enabled by policy which currently does not exist.

RESEARCH: There is much more to learn. If we as a nation hope to manage risk from the most severe hazards, we need to learn how to work with rather than control nature. Research is needed to better define the actual role of natural environments in managing surge and waves; rules of thumb are just too inaccurate. Given the challenges of continued sea level rise and subsidence and the potential for more intense storms, the art of building and sustaining natural environment is especially important. The vulnerability of natural features to large storms is a particular challenge if we are to rely on them for long-term risk reduction. The fact that there are not enough high quality natural materials to build traditional structures demands that we seek innovative alternatives. The ability to routinely monitor conditions and residual risk on a system- wide and regional basis will require much more effective sensing and analysis, particularly concerning geotechnical issues.

CHANGE: Our current policy and practice does not deal well with change. We must be more anticipatory and adaptive as changes occur in the hazard, the system, or the potential consequences. All of these factors changed dramatically over the life of the hurricane protection projects in New Orleans with little capability for appropriate response. This is another symptom of short term rather than long term sustainable strategies, policies and practices for addressing a major life safety need."

c. Hurricane Ike - East Texas, 2008. Devastating flooding also occurred during Hurricane Ike around Galveston Bay and coastal east Texas in September of 2008. One particular subdivision in LaBelle, Texas suffered about four feet of flooding in houses that have been constructed in the past two decades (see Figure M-3). Most homeowners possessed Elevation Certificates (required by FEMA for flood insurance purposes) showing elevations above the flood stage. A post storm investigation revealed that the original elevation leveling survey (conducted in 1982) to establish elevation control for the subdivision was based on an NGS bench mark with a published elevation that was determined in 1959. A Texas Water Commission study in the 1970s of the area revealed that the area where the NGS bench mark was located was subject to approximately three feet of subsidence. Recent surveys of adjoining bench marks shows up to four feet in subsidence in the same area indicating that the area has experienced another foot of subsidence since the Texas Water Commission report.

Figure M-3. Subdivision flooding during Hurricane Ike in 2008; the house floor level location was measured based on connections to inaccurate survey elevation control due to inaccurate survey elevation control. (The house has since been demolished.)
(Photo courtesy of Caroline Miller.)

M-9. Expected Magnitude and Range of Elevation Uncertainty.

a. Terrestrial Datums.

(1) Elevation Determination. Typically surveying measurement estimation precision, like most scientific measuring techniques, relies on repeatability at a 95 percent confidence level. These tolerances are the result of repeated measurements over time with statistical analysis based on a least squares adjustment methodology, whereas accuracy is an estimation of how close the measurement is to the true value. Table M-2 is from USACE Circular 1110-2-6070 [1], 1 July 2009, "*COMPREHENSIVE EVALUATION OF PROJECT DATUMS - Guidance for a Comprehensive Evaluation of Vertical Datums on Flood risk management, Shore Protection, Hurricane Protection, and Navigation Projects*." These recommended standards show allowable error in establishing survey control in the vertical and horizontal and their respective datums.

[1] Interim EC 1110-2-6070 was superseded by and incorporated into this manual.

Table M-2. Nominal or Target Accuracy Standards for Connecting USACE Flood Control Projects to the NSRS Network – Primary Project Control Points.

	NSRS Global Accuracy (95%)	Reference Datum
Vertical	± 0.25 ft (± 8 cm)	NAVD88
Horizontal	± 2 ft (± 60 cm)	NAD83

Accuracies stated are at the 95% confidence level relative to points published by NOAA on the NSRS.

(a) It should be noted that these accuracies are easily achievable with modern surveying equipment operated by educated and experienced field staff. This includes geodetic quality GPS equipment, which offers the additional assurance of supplying much tighter horizontal accuracy. As well, the accuracies stated in Table M-2 refer to tying USACE projects to the NSRS. There is every likelihood that vertical precision within a project (i.e., relative accuracies) will be at a much higher standard.

(b) From discussions with USACE engineers and surveyors, these accuracy standards are suitable for the purpose of tying USACE projects to the NSRS. These accuracies are in line with the desire to standardize USACE projects to a common geodetic vertical datum on a national scale. USACE should re-assess and update these accuracy standards as hydrological and hydraulic modelers adopt or require higher accuracies in the future.

(2) Terrestrial Datums. Since 1929, and hence for most of the modern projects undertaken by the USACE, there have been only two official terrestrial vertical datums: NGVD29 and NAVD88. While there are no data available on the accuracy of these adjustments, there have been extensive comparisons of the two datums, with the assumption that the NAVD88 datum is void of distortions introduced by the 1929 adjustment, which were introduced by constraining the adjustment to fit 26 tide gages, as well as from using less rigorous leveling corrections than used in NAVD88.

(3) The NGVD29 datum, while incorporating distortions on a national scale, was precise in a relative sense over regional areas. As such, the NGVD29 served USACE well for the vast majority of its projects. The move to NAVD88, as directed by Lt. General Carl A. Strock, is encouraged, as the NAVD88 datum represents the most accurate terrestrial orthometric datum available. Figure M-4 shows the differences between NAVD88 and NGVD29 (Zilkoski et al, 1992).

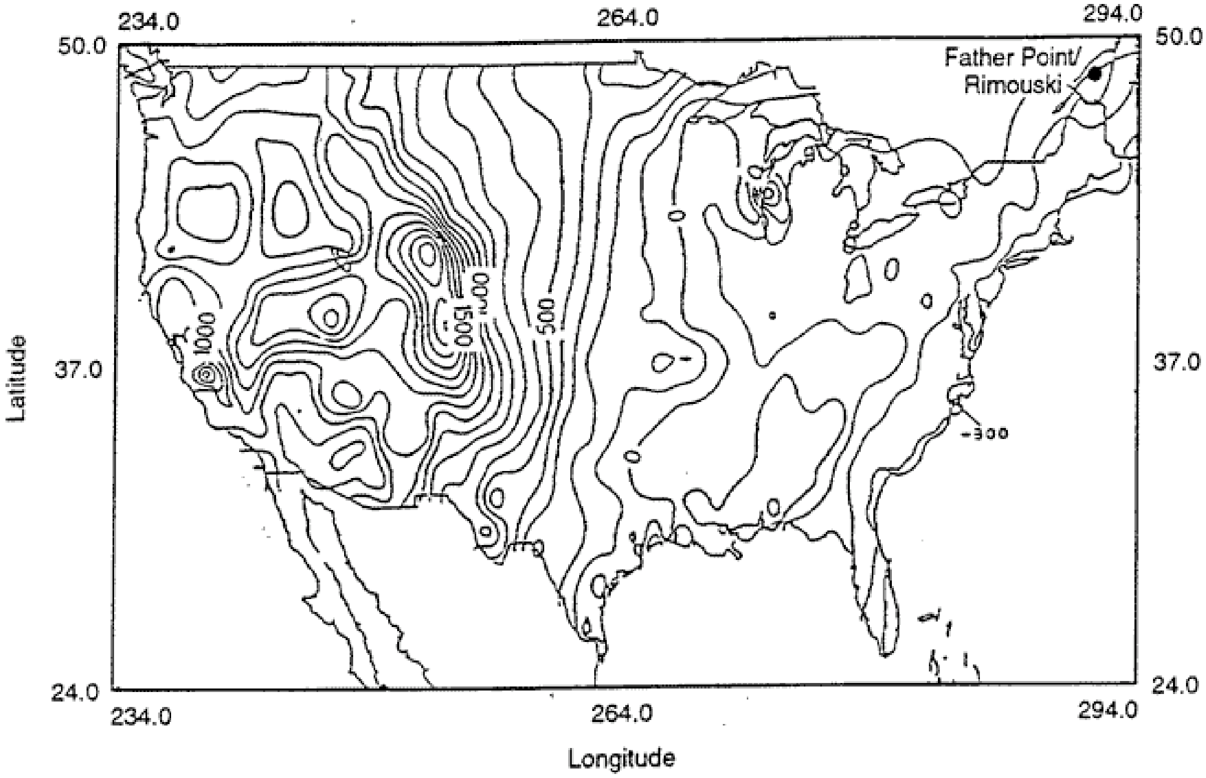

Figure M-4. Contour map depicting height differences between NAVD88 and NGVD29. (units = mm)

(4) Land Subsidence & Isostatic Rebound and Relative Sea Level Change. Surveyors and engineers should pay close attention when determining elevations in areas that are prone to subsidence or isostatic rebound. Examples of each of these geotechnical phenomena can be seen in tide gage records from CO-OPS (see http://www.co-ops.nos.noaa.gov/sltrends/). Figure M-5 shows the relative sea level change due to a combination of rapid land surface rising around Juneau Alaska (due to isostatic rebound) and less rapid global sea level rise. As such, relative to sea level is actually falling in this region at a rate of nearly 13 millimeters per year. In contrast, Figure M-6 shows the relative sea level rate around Galveston, Texas, which is a combination of land sinking (due to subsidence) and global sea level rise. As such, in this region relative sea level rises at the rate of over 6 millimeters per year. Quite often, in practice, NGVD29 and NAVD88 are assumed to be equivalent to present day local MSL. This is an incorrect assumption that often leads to poor planning. The relationships need to be precisely established and used appropriately. More precise rates of vertical land movement are being determined from establishment of the Continuously Operating Reference System (CORS) by NOAA/NGS. A CORS co-located at a tide station, or an episodically re-checked GPS survey at the tide station, will provide more precise magnitude of the local vertical land movement to separate this signal from the rate of local relative sea level change.

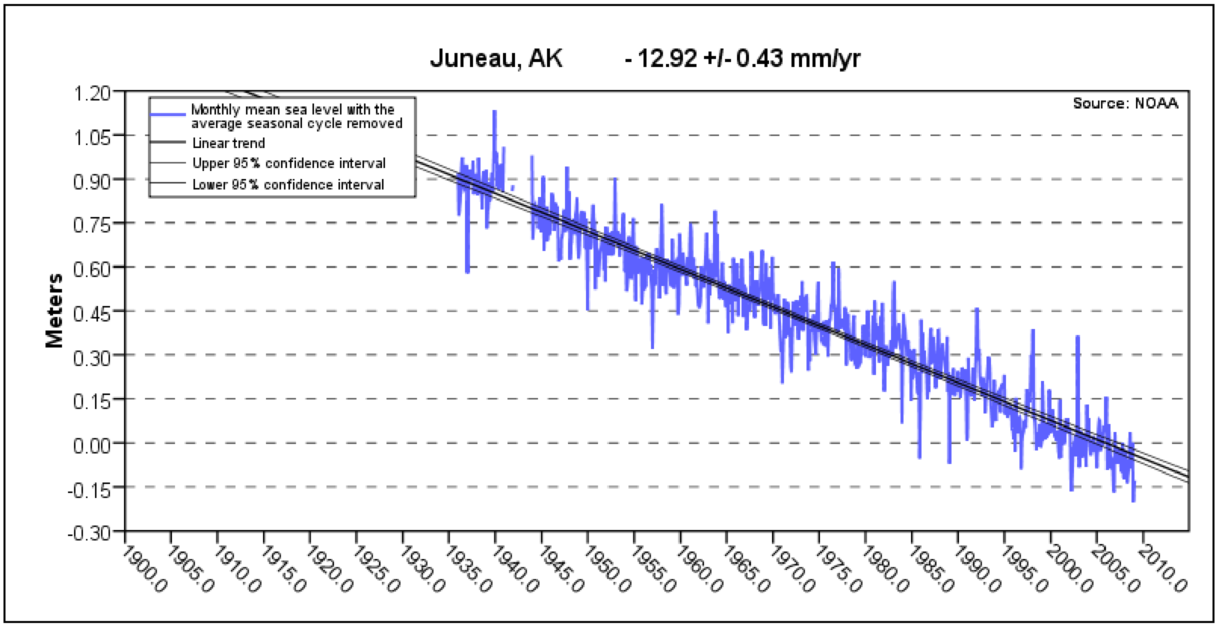

Figure M-5. The mean sea level trend for Juneau, Alaska is -12.92 millimeters/year with a 95% confidence interval of +/- 0.43 mm/yr based on monthly mean sea level data from 1936 to 2006, which is equivalent to a change of -4.24 feet in 100 years.

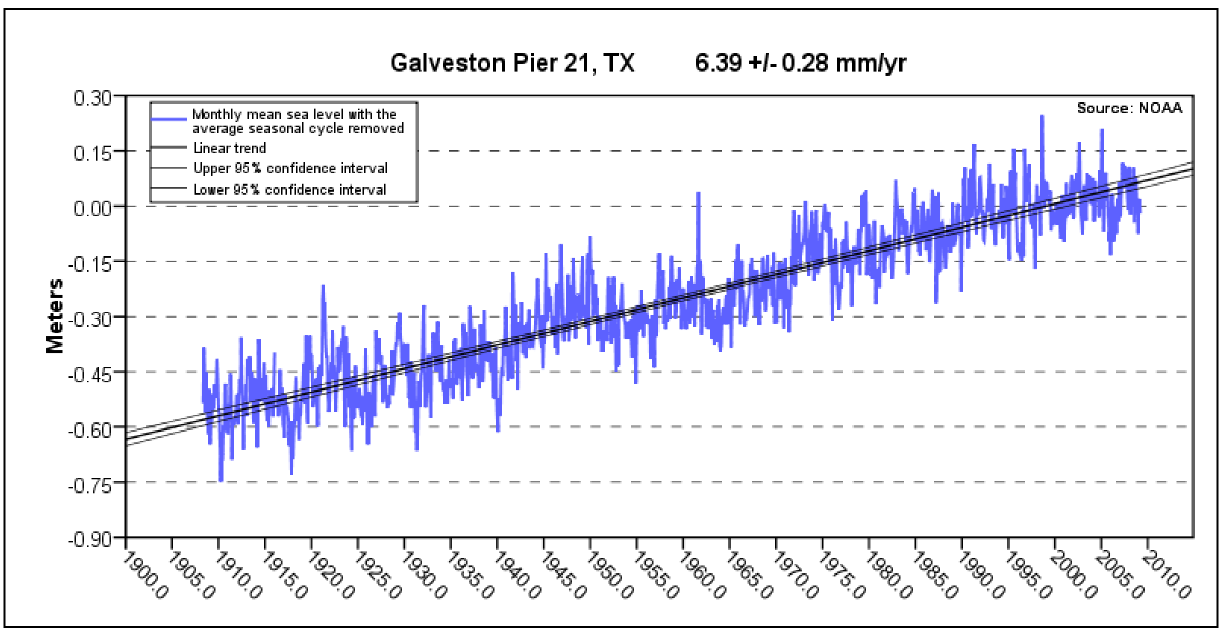

Figure M-6. The mean sea level trend for Galveston, Texas is 6.39 millimeters/year with a 95% confidence interval of +/- 0.28 mm/yr based on monthly mean sea level data from 1908 to 2006, which is equivalent to a change of 2.10 feet in 100 years.

(5) Insufficient or Non-Existing Coverage. Elevation ties to the NAVD88 elevation network should take into account the distance to the nearest recoverable bench marks. In many areas of the continental United States this may prove to be difficult. Figure M-7 shows the national vertical control used in the NAVD88 adjustment and Figure M-8 shows the available first order control in Texas. The maps show an impressive network, however many areas are still without easily accessible bench marks. Tying to the elevation network in many locations can be problematic and/ or expensive.

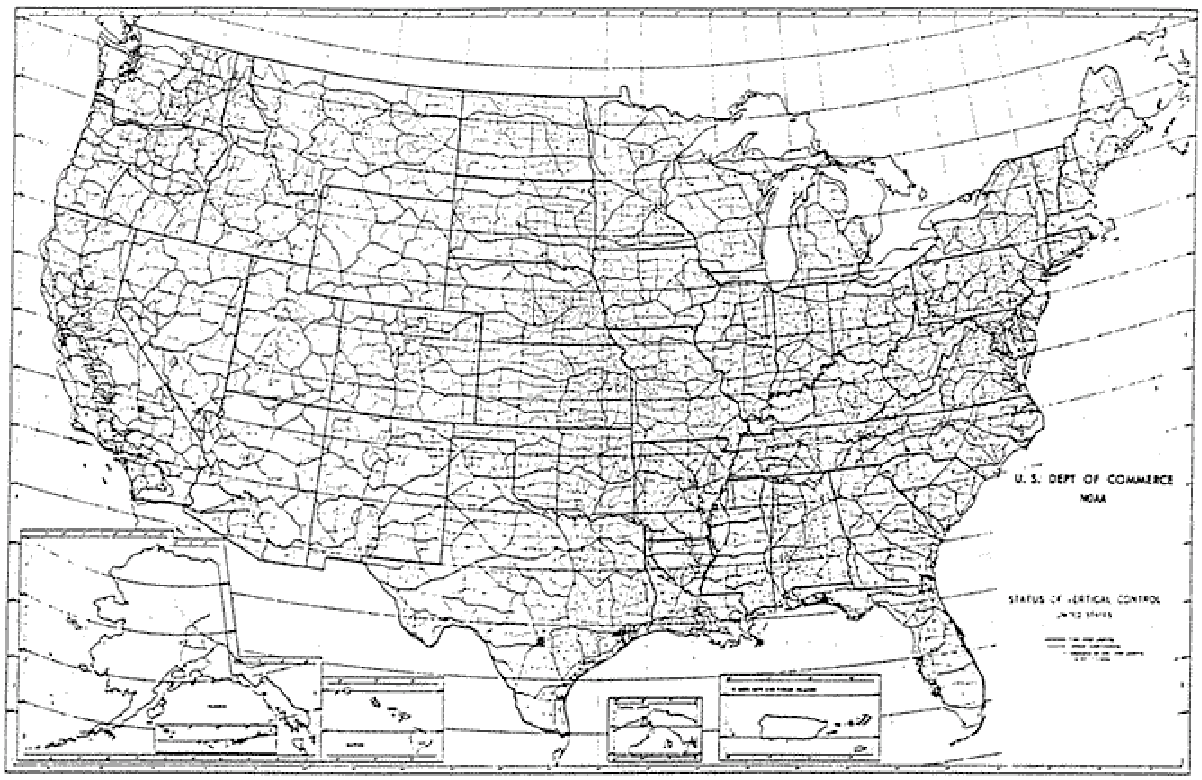

Figure M-7. Vertical control used in 1988 adjustment.

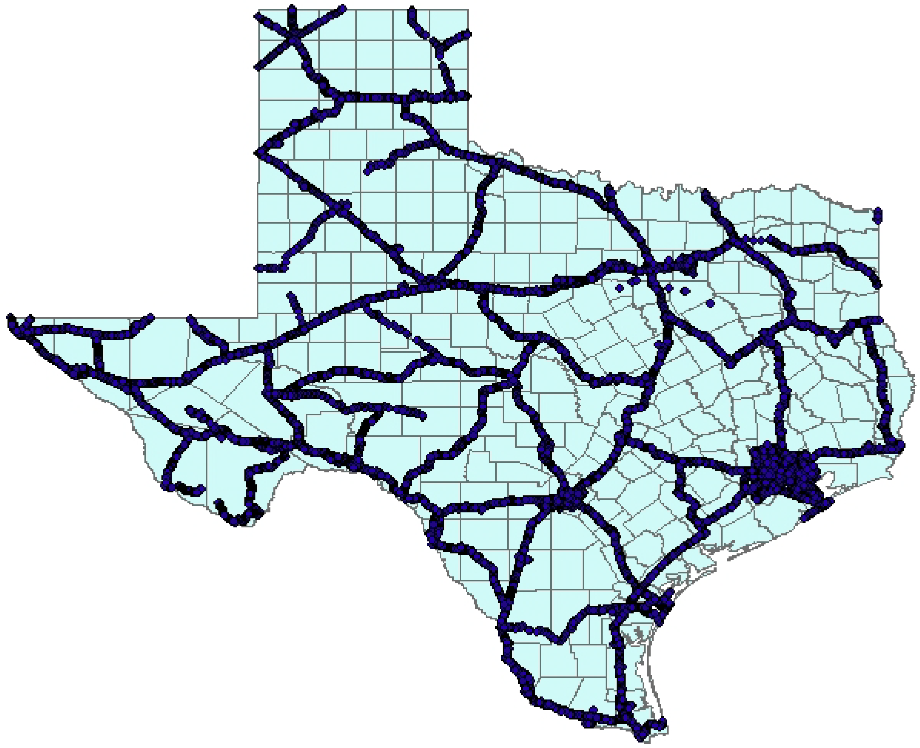

Figure M-8. NGS Database of First-Order Bench Marks in Texas.
(Note the number of counties that show no bench marks.)

b. Water Level Datum.

(1) Tidal Elevations—Gage Uncertainty: Accuracy of a Tidal-Geoid Model of a Navigation Project. The following Table M-3 is from USACE Circular 1110-2-6070, 1 July 2009, "*COMPREHENSIVE EVALUATION OF PROJECT DATUMS - Guidance for a Comprehensive Evaluation of Vertical Datums on Flood Control, Shore Protection, Hurricane Protection, and Navigation Projects*." This table represents the USACE desired accuracy of a navigation project model, considering both the MLLW datum and the geoid.

Table M-3. Recommended Accuracies for Reference Datums on Navigation Project Tidal Models.

	Accuracy (95%)	Reference Datum
Absolute accuracy of tidal-geoid model	± 0.25 ft (± 8 cm)	MLLW
Relative accuracy of tidal-geoid model	± 0.1 ft (± 3 cm)	MLLW
Tidal-geoid model resolution	0.01 ft	
Linear density along navigation channel	100 to 500 ft (varies with magnitude of tidal range)	
Geoid model	use latest available at time of study (currently Geoid03)	
Accuracy of predicted geoid model	< 5 cm	
Accuracy of predicted MLLW datums In offshore entrance channels	< 5 cm	
Tidal-geoid model format	1D or 2D (typically 1D for linear navigation channels)	

NOTE: The above standards are believed representative for most CONUS navigation projects. Exceptions may exist in extreme tide ranges or in parts of Alaska.

(2) From discussions with USACE engineers and surveyors, these proposed accuracy standards are suitable for the purpose of tying USACE projects to the NSRS and the NOS tidal datums computed for the latest tidal epoch. These accuracies are in line with the desire to standardize USACE projects to a common geodetic vertical datum on a national scale. USACE should re-assess and update these accuracy standards as NOAA adopts changes to the NSRS in the future.

(3) Tidal Elevations—Proximity of Gage. Presently the NWLON comprises some 200 tide gages. While most of these gages are located adjacent to major population centers and ports, there are coastal areas that are not covered adequately to supply accurate water level and tidal datums. Recently, CO-OPS conducted a comprehensive gap analysis of adequate tidal data coverage. This study is now published as NOAA Technical Memorandum NOS CO-OPS 0048

entitled, "*A Network Gaps Analysis for the National Water Level Observation Network*" by Stephen K. Gill and Kathleen M. Fisher, March 2008. NOS states:

> *"A deterministic approach to estimating the areas of NWLON coverage for datum determination at nearby subordinate tide stations has been developed. The approach uses the basic error analyses of Swanson (1974) and the regression error analyses of Bodnar (1981) to estimate regions of coverage for each individual NWLON station. Using GIS tools, the information is displayed on maps of coverage polygons. The GIS output is then used to identify geographic areas that represent gaps in the NWLON. The datum error polygons can be used for multiple purposes for short-term and long-term management of the NWLP. This analysis is being used for strategic planning and prioritization of locations to establish new NWLON stations as the network grows towards the optimum number of stations. It is being used to make decisions regarding utilization of resources for the importance of bringing an NWLON station back on line immediately or if a nearby station can be used effectively as a back-up until reconstruction can take place. The analysis results identified approximately 113 gaps in NWLON coverage beyond the 200 station deployed as of FY2007. Forty-three (43) gaps are located along the east coast, 33 in the gulf coast, 6 gaps on the west coast, 29 gaps in Alaska, and 2 in Hawaii."*

(4) Figure M-9 shows the water level observation gap analysis for the Gulf of Mexico along the Texas coast.

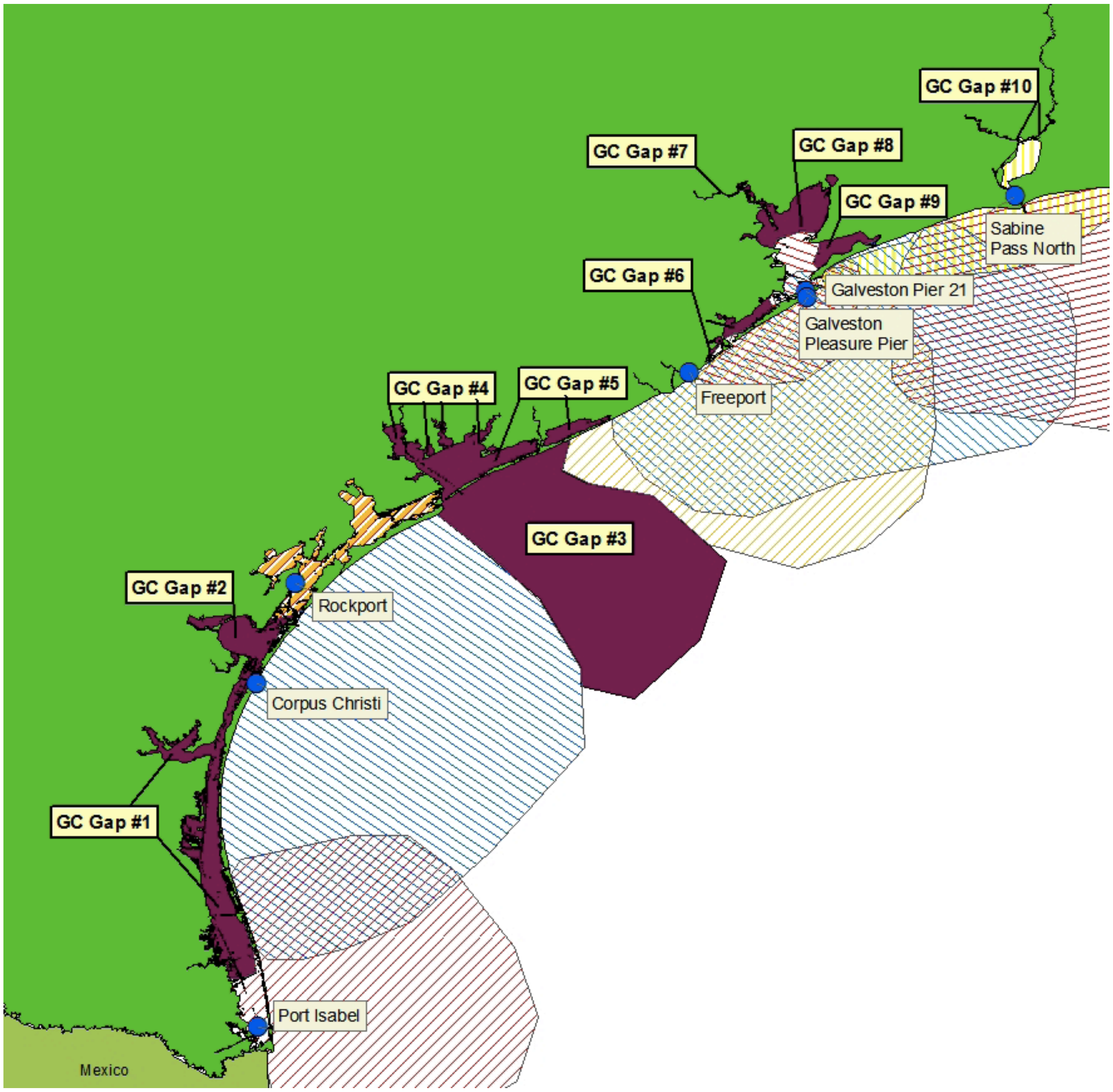

Figure M-9. NWLON gaps analysis for Texas.

(5) Tidal Elevations—Gage Historical Record (time span). In order to compute tidal datums, a water level measurement station needs to collect data for at least 19 years and preferably during the 19 years selected for the National Tidal Datum Epoch (NTDE). The longer the water level station collects data, the greater reliability of observing long-term sea level changes. Tidal datums are computed using monthly means for high and low tides over the 19-year epoch. Monthly means cannot be computed if a more than 72 hour gap exists in the record. Therefore, it is imperative that a continuous record for the data series be maintained for the duration of the epoch and well beyond the 19 years if long-term trends are to be computed. The following is taken from NOAA Special Publication NOS CO-OPS 1, "*Tidal Datums and Their Applications*," Silver Spring, Maryland, June 2000. This special publication was prepared under the editorship of Stephen K. Gill and John R. Schultz; Contributors were Wolfgang Scherer, William M. Stoney, Thomas N. Mero, Michael O'Hargan, William Michael Gibson, James R.

Hubbard, Michael I. Weiss, Ole Varmer, Brenda Via, Daphne M. Frilot, and Kristen A. Tronvig. The following excerpts from this publication describe the definitions of the NTDE and the tidal datums computed:

"First-Reduction Tidal Datum Computations.

The National Tidal Datum Epoch (NTDE) is defined as the specific 19-year cycle adopted by the NOS as the official time segment over which water level observations are taken and reduced to obtain mean values (e.g., mean lower low water) for tidal datums. Adoption of the NTDE averages out long-term seasonal meteorological, hydrologic, and oceanographic fluctuations. It provides a nationally consistent tidal datum network by accounting for seasonal and secular trends in sea level that affect the adequacy of the tidal datums (Marmer, 1951). NOS operates the NWLON to provide the data required to maintain the epoch and to make primary and secondary determinations of the tidal datums. The present NTDE is 1983 through 2001. It is reviewed for revision at least every 20 to 25 years and implementation of a new NTDE is currently under consideration by NOS.

A vertical datum is called a tidal datum when it is defined by a certain phase of the tide. Tidal datums are local datums and should not be extended into areas, which have differing hydrographic characteristics without substantiating measurements. In order that they may be recovered when needed, such datums are referenced to fixed points known as benchmarks.

A primary determination of a tidal datum is based directly on the average of observations over a 19-year period. For example, a primary determination of mean high water is based directly on the average of the high waters over a 19-year period. Tidal datums must be specified with regard to the NTDE (Marmer, 1951).

Mean Higher High Water (MHHW) is defined as the arithmetic mean of the higher high water heights of the tide observed over a specific 19-year Metonic cycle (the NTDE). Only the higher high water of each pair of high waters of a tidal day is included in the mean. For stations with shorter series, simultaneous observational comparisons are made with a control tide station in order to derive the equivalent of a 19-year value (Marmer, 1951).

Mean High Water (MHW) is defined as the arithmetic mean of the high water heights observed over a specific 19-year Metonic cycle (the NTDE). For stations with shorter series, simultaneous observational comparisons are made with a control tide station in order to derive the equivalent of a 19-year value (Marmer, 1951) Use of the synonymous term, mean high tide, is discouraged.

Mean Low Water (MLW) is defined as the arithmetic mean of the low water heights observed over a specific 19-year Metonic cycle (the NTDE). For stations with shorter series, simultaneous observational comparisons are made with a control tide station in order to derive the equivalent of a 19-year value (Marmer, 1951). Use of the synonymous term, mean low tide, is discouraged.

Mean Lower Low Water (MLLW) is defined as the arithmetic mean of the lower low water heights of the tide observed over a specific 19-year Metonic cycle (the NTDE). Only the lower low water of each pair of low waters of a tidal day is included in the mean. For stations with shorter series, simultaneous observational comparisons are made with a control tide station in order to derive the equivalent of a 19-year value (Marmer, 1951).

In addition, Mean Tide Level (MTL), Mean Range (Mn), Diurnal High Water Inequality (DHQ), Diurnal Low Water Inequality (DLQ), Great Diurnal Range (Gt), Diurnal Tide Level (DTL), and Mean Sea Level (MSL) have the following definitions: MTL is the average of MHW and MLW; Mn is the difference between MHW and MLW; DHQ is the difference between MHHW and MHW; DLQ is the difference between MLW and MLLW; Gt is the difference between MHHW and MLLW; DTL is a tidal datum which defines the midpoint between MHHW and MLLW; and MSL is defined as the arithmetic mean of hourly heights observed over a specific 19-year Metonic 40 cycle (the NTDE). Shorter series, such as monthly mean sea level and yearly mean sea level, are specified in the name (Marmer, 1951; Hicks, 1985). The Glossary of this document contains the definitions of additional tidal datums.

Equivalent Short-Term Datums.

Due to time and resource constraints, primary determinations of tidal datums are not practical at every location along the entire coast where tidal datums are required. At intermediate locations, a secondary determination of tidal datums can usually be made by means of observations covering much shorter periods than 19 years if the results are corrected to an equivalent mean value by comparison with a suitable primary control tide station (Marmer, 1951). A primary control station is one (Marmer, 1951) at which continuous observations have been made over a minimum of 19 years spanning the NTDE. The data series from this station serves as a primary control for the reduction of relatively short series from subordinate stations through the method of comparison of simultaneous observations and for monitoring long-period sea level trends and variations.

A secondary control tide station is a subordinate tide station at which continuous observations have been made over a minimum of one year but less than 19 years. The data series is reduced to equivalent 19-year tidal datums by comparison with simultaneous observations from a suitable primary control observation.

A tertiary control tide station is a subordinate tide station at which continuous observations have been made over a minimum of 30 days but less than 1 year. The data series is reduced to equivalent 19 year tidal datums by comparison with simultaneous observations from a suitable secondary or primary control tide station. NOS uses the following methods to perform comparisons of simultaneous observations for secondary (i.e., short-term) determinations of tidal datums. "

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(6) Tidal Elevations—Tidal Datum Accuracy. Tidal datums for each water level observation station vary in their accuracies depending on the type of tide (diurnal, semi-diurnal,

or mixed); the range of tide; the coastal hydraulics; and the weather effects of wind speed, wind direction, and barometric pressure. The standard deviation of the computed datums is computed and is available from the NOS. The accuracy of tidal datums is discussed in the following, which is taken from NOAA Special Publication NOS CO-OPS 1:

"Accuracy. Generalized accuracies (Swanson, 1974) for datums computed at secondary or tertiary stations based on the standard deviation error for the length of the record are summarized below. These values were calculated using control stations in the NWLON. The accuracies of the secondary and tertiary datums can be interpreted as known to within plus or minus the appropriate value in the table. That is, the values in the table are the confidence intervals for the tidal datums based on the standard deviation.

Generalized accuracy of tidal datums for East, Gulf, and West Coasts when determined from short series of record and based on +/- sigma. From NOS CO-OPS 1, Swanson (1974).

Series Length (months)	*East Coast (cm)*	*(ft.)*	*Gulf Coast (cm)*	*(ft.)*	*West Coast (cm)*	*(ft.)*
1	*4.26*	*0.13*	*5.91*	*0.18*	*4.26*	*0.13*
3	*3.28*	*0.10*	*4.92*	*0.15*	*3.61*	*0.11*
6	*2.30*	*0.07*	*3.94*	*0.12*	*2.62*	*0.08*
12	*1.64*	*0.05*	*2.95*	*0.09*	*1.97*	*0.06*

The uncertainty in the value of the tidal datum translates into a horizontal uncertainty of the location of a marine boundary when the tidal datum line is surveyed to the land (Demarcating and Mapping Tidal Boundaries, 1970). The table below expresses the uncertainty in the marine boundary as a function of the slope of the land. A slope of 1% means that the land rises 1 meter for every 100 meters of horizontal distance. In this table, the error is defined as (0.03 m) x [cotangent(slope)]. The greatest errors in the determination of the marine boundary occur for relatively flat terrain, which is characteristic of broad sections of the Atlantic and Gulf Coasts.

Error in position of marine boundary as a function of the slope of the land (From NOS CO-OPS 1).

% of Slope	*Degree of Slope (degrees)*	*Error (meters)*
0.1	*0.05*	*32.3*
0.2	*0.1*	*14.9*
0.5	*0.3*	*6.1*
1.0	*0.6*	*3.0*
2.0	*1*	*1.5*
5.0	*3*	*0.61*
10.0	*6*	*0.30*
15.0	*9*	*0.18*
20.0	*11*	*0.15*
30.0	*17*	*0.09*
50.0	*27*	*0.06*
100.0	*45*	*0.03*

(7) NOAA has recently updated the above information. See "*Water Level Station Specifications and Deliverables for Shoreline Mapping Projects*," May 2009; Center for Operational Oceanographic Products and Services; National Ocean Service; National Oceanic and Atmospheric Administration.

(8) In practice, NOAA considers the accuracies found in the above table to be maximum values, as they are based on comparisons of NWLON station pairs. Most of which are a considerable distance from each other (both in miles and in tidal differences). NOAA now uses a set of equations developed through a multiple correlation analysis of the Swanson results by Bodnar (1981) to determine higher resolution accuracy estimates for individual stations rather than a regional approach. Errors in the estimation of 19 NTDE datums at short term stations are a function of the geographic distance from the short-term station to the control station; the difference in the time of tide between the stations; and, the ratio of the range of tide between the short-term and control. Having a stronger, denser NWLON control network is one way NOAA is minimizing these dependencies. The other obvious way to minimize datum errors, in practice, is to leave short-term stations in operation for at least one-year for a more accurate datum computation (the standard deviations for the errors in the above table for a one-year station are only ½ of what they are for a one-month station).

(9) Non-Tidal Elevations: Low Water Reference Plane. Low Water Reference Plane (LWRP) is a hydraulic reference plane, established from long-term observations of the river's stage, discharge rates, and flow duration periods. The low water profile is developed about the 97-percent flow duration line (USACE EM 1110-1-1005). For example, a low water profile was taken throughout the Memphis district from August 15 through 23 in 2005. This profile was used to shape the proposed 2007 LWRP between the five key stations. The August 2005 profile is considered representative of the low water slope that would occur as a result of the 97 percent

exceedance discharge. The stages that occurred on the survey dates were within 0.0-0.7 feet of the computed LWRP stages at the five key gage locations; stages for survey data at four of the five locations were within 0.5 feet (from Memphis District, USACE, 2007).

(10) Non-Tidal Elevations: Stream Gage Accuracy. Stream gages are designed to give hydraulics engineers data on the discharge flow of water at that point in a stream. The discharge flow is the product of the velocity and the area (i.e., *discharge = area × velocity*). The stream gage measures the stage of the river, which is then used as an observable on a stage vs. discharge logarithmic graph. As such, the water surface stage elevation is a component of the final discharge value. Olean and Norris report the accuracy of stream gages used by the US Geological Survey as "S*tream gages operated by the US Geological Survey provide stage measurements that are accurate to the nearest 0.01 foot, or 0.2 percent of stage, whichever is greater*" (Olson and Norris, 2006). Most U.S. Geological Survey stream gages used by the USACE are tied to NGVD29 datum. Updating these values to NAVD88 will assist the modeling of stream and river hydraulics as more sophisticated models allow the management of entire river catchment areas.

c. VDatum.

(1) The tidal hydraulic forcing of the VDatum model relies on existing water level observation stations. The model interprets water levels over time between these existing stations. The uncertainty with these models increases with distance from the water level observation stations and areas that are more remote from tide stations, such as South Bay in San Francisco, show higher variances in the estimation of tidal datums. VDatum is a useful tool for converting elevations from one datum to another (and along the coast this would include tidal datums); however, the software is interpretive of various data sets and models and does have uncertainty depending on the quality and quantity of the underlying data sets. Additionally, VDatum is still under development as of this writing. Changes and adjustments to the algorithms are ongoing. The following timeline is an indication of the current rate of change to the software taken from http://vdatum.noaa.gov/about/currentevents.html:

March 27, 2009
VDatum version 2.2.4 Released!

August 07, 2008
VDatum version 2.2.3 Released!
- Added GEOID 06 model: supporting elevation conversions in Alaska with the newest Geoid transformation grids.

July 11, 2008
VDatum version 2.1.3 Released!
- Added transformation filters for conversion from tidal datums to IGLD 85 and vice versa. The conversions between tidal datums and IGLD85 are prohibited.

July 2, 2008
VDatum version 2.1.2 Released!
- Fixed sign problem with sounding data

June 23, 2008
VDatum version 2.1.1 Released
- Added HPGN transformation grids which supporting horizontal transformation from NAD 27 to NAD83 (HARN). Fixed problem with IGLD85 grids. Fixed problem with 3-D transformation based on HPGN v2.9 model. Input and output time are set to January 1st of the current year. Minor tweaks and GUI improvements.

October 01, 2007
VDatum version 2.0.0 Release!
VDatum 2.0.0 has been released. This version contains significant enhancements and new features, including a new user-friendly interface and productivity features.

(2) The NOAA VDatum team is conducting a detailed error analysis on the use of VDatum for each of its regional applications. Some of the analyses completed for certain regional can currently be found on the VDatum website. This error analysis reveals how complex a task assessing the accuracies associated with VDatum conversions can be. These complexities, coupled with the previously mentioned ongoing development of VDatum, present challenges in trying to assess the risk and uncertainty of using VDatum transformation values.

SECTION 3

Recommendations & Conclusions

M-10. Datum Uncertainties. Most risk assessments for flooding depend upon accurate vertical measurements and the accuracy of these measurements depends upon several factors. These can be divided into two categories: factors related to measurement errors, such as leveling, and factors related to measurement bias, such as the use of a faulty monument. It is important to review these factors with surveyors and others who are knowledgeable about the local issues and conditions that impact vertical measurement accuracy.

a. The terrestrial datum can be subject to problems related to leveling control, datum confusion, datum coverage, and relative sea level changes. In addition they should be tied into the NSRS to allow for comparisons among projects and for analysis of larger flooding events and unexpected disasters. Engineers involved in hydrological risk assessments should discuss the effects of these issues with local experts knowledgeable in these factors. In some cases, this might require resurveying elevations in the project area, as well as a detailed analysis of various datums used in that area.

b. Surveyors and others responsible for establishing and maintaining reference elevations for a project should ensure that standard surveying practices are followed when establishing elevations. This should include good quality control of surveys. Quality control, at a minimum, should include keeping long-term archives of survey logs and checking individual logs to assess leveling precision to determine whether or not any bias was introduced from using insufficient or inaccurate monuments.

c. Projects on the coast that rely on tidal datums are subject to possible inaccuracies due to variations of the tidal gages in the project area, proximity of tidal gages to the project site, and incomplete historical records or tidal gage readings. In addition, relative sea level changes in the area should be assessed, including both the general rate of change and the length of time from the middle of the epoch used to establish the tidal datum. Again, it is generally useful to discuss these issues with local experts, surveyors and others responsible for maintaining these gages and to review a trend analysis for these data to plan for elevation changes during the estimated project life expectancy.

d. A general approach to incorporating these datum issues into risk assessments is outlined in Table M-4. This table also appears as a worksheet in Addendum B to this Appendix, together with supporting flowcharts summarizing how the elevation uncertainty worksheet is used to compute an elevation uncertainty budget. Estimates of potential vertical measurement bias and variance should be evaluated in conjunction with local experts familiar with these measurements, surveyors and mapping experts for the project region. This table provides general groupings for sources of vertical uncertainty that includes terrestrial, water levels, and datum conversion issues.

Table M-4. Worksheet for Determining Elevation Uncertainty for Project Risk Assessments.

SOURCES OF VERTICAL UNCERTAINTY	Average Bias(ft)	Average Variance(ft^2)
TERRESTRIAL		
Subsidence(-)/Rebound(+)		
Measurement Uncertainty		
Datum Coverage		
National Spatial Reference System		
Other Sources of Uncertainty		
Total Terrestrial Adjustment		
WATER LEVELS		
TIDAL		
Sea Level Change		
Gage Accuracy		
Gage Proximity		
Project Epoch Adjustment		
Gage Historical Record		
Other Sources of Vertical Uncertainty		
Total Tidal Adjustment		
NON-TIDAL		
Low Water Reference Plane		
Gage Accuracy		
Gage Proximity		
Pool Elevation Record (length)		
Other		
Total Non-Tidal Adjustment		
DATUM CONVERSIONS		
NGVD29 to NAVD88		
NAD83 Ellipsoid to NAVD88		
Geoid Model Uncertainty		
Legacy Datum to NAVD88		
VDatum conversions		
Other		
Total Datum Conversion Adjustment		
OVERALL UNCERTAINTY		

(1) The first column contains the potential sources described in this report organized by terrestrial elevations, water levels, and datum conversions. Depending upon the project, one or all of these categories would be used to calculate the elevation uncertainty. The elevations assumed in risk modeling should be raised or lowered by the *overall uncertainty*, the last line of

Table M-4. Overall uncertainty is added to the assumed elevations. To account for a 95% one-sided confidence interval in variances, 1.645σ is also added or subtracted from the assumed elevations. This calculation is added in cases where higher elevations produce higher risks, and is subtracted in cases where lower elevations produce higher risks.

(2) The second column is used to record the magnitude of potential bias or deviation between the project elevation measurements at the time the project was designed and true elevations over the design life of the project. In some cases the sign of the bias (+ or -) may be known. Historical records in Galveston, for example, indicate that relative sea level change is rising, creating a negative bias. At present, relative sea level change in Galveston is approximately 2.1 ft/100 years.

(3) Measurement errors, expressed as a variance, are recorded in the third column. In some cases these can be expressed precisely. Leveling variance, for example, is often controlled to a very small tolerance, typically $\sigma^2 = 0.01^2$ for a project. On the other hand, although information is generally available for estimating the variances associated with datum conversions and coverage, they are generally less precise.

e. In an ideal state in which elevations are known exactly and are not expected to change over the project lifespan, the elevation budget, or adjustment, would be zero. Recorded elevations would be used as input into the risk assessment calculations relating flood stage to the probability of flooding. For most projects, however, elevation adjustments are necessary. This worksheet is designed as an aide in that process.

f. If elevations are not expected to change over time, and if measured elevations are unbiased, then only variances are recorded in the worksheet. These are recorded by source as a variance with units (ft^2) since the final adjustment for individual variances is calculated using the formula for the total standard deviation (σ_T):

$$\sigma_T = \sqrt{\sigma_1^2 + \sigma_2^2 + \cdots + \sigma_{1k}^2}$$

where σ_1^2, σ_2^2,…, σ_{1k}^2 are the k non-zero variances recorded in the Elevation Uncertainty Worksheet, assuming they are all uncorrelated.

g. If elevations might change over the project lifespan, or if epoch adjustments are necessary, then the total bias is calculated by simply summing the individual bias adjustments expected in the elevations.

h. Some bias values may be expressed as plus and minus, +/-. This can occur because of spatial variation and uncertainty of historical data. The worst case scenario for a bias value will be either positive or negative depending upon the nature of the project. A conservative approach would dictate that the overall bias calculation is computed using only the worst case scenarios for a particular project.

i. For example, levee height is directly related to the risk of a flooding event causing a levee breach. In this case, a negative bias is worse than positive since a negative bias would result in true levee height being lower than the design height needed for that risk level. Dredging projects, on the other hand, might be concerned about a vertical bias in either direction. If a positive bias is not accounted for the design depth, then channels would be dredged deeper than required, and a negative bias would result in channels that are shallower than required.

M-11. Example Computation for a Coastal Protection Project. As an example, consider a coastal protection project such as levees in Galveston with a 50-year design life. The project levees are designed using LMSL wherein some of the measurements require conversion of historical elevations measured in NGVD29 and the final values are expressed using NAVD88. Table M-5 contains representative entries for this hypothetical example.

a. Leveling is conducted using accepted standard surveying practices. This requires all measurements be closed-loop measurements to ensure the measurement variance is no larger than 0.01^2 ft. Bias for these measurements is zero if these measurements are tied to two or more reliable monuments. In addition, quality control practice would archive project log books and check the closed-loop calculations to ensure the measurement variance meets measurement standards.

Table M-5. An Example Application of Vertical Inaccuracy Analysis for a Hypothetical Coastal Protection Project in Galveston, TX.

SOURCES OF VERTICAL UNCERTAINTY	Average Bias(ft)	Average Variance(ft^2)
TERRESTRIAL		
Subsidence(-)/Rebound(+)	-0.70	0.01^2
Measurement Uncertainty	na	0.01^2
Datum Coverage	na	0.00
National Spatial Reference System	na	0.20^2
Other Sources of Uncertainty		
Total Terrestrial Adjustment	-0.70	0.0402
WATER LEVELS		
TIDAL		
Sea Level Change	-0.35	0.01^2
Gage Accuracy	na	0.01^2
Gage Proximity	na	0.01^2
Project Epoch Adjustment	-0.06	0.00
Gage Historical Record	na	0.00
Other Sources of Vertical Uncertainty	na	
Total Tidal Adjustment	-0.41	0.0003
NON-TIDAL		
Low Water Reference Plane		
Gage Accuracy	na	
Gage Proximity	na	
Pool Elevation Record (length)	na	
Other		
Total Non-Tidal Adjustment		
DATUM CONVERSIONS		
NGVD29 to NAVD88	na	0.01^2
Ellipsoid to NAVD88	na	
Ellipsoid to Geoid	na	
Legacy Datum to NAVD88	na	
VDatum conversions	na	
Other	na	
Total Datum Conversion Adjustment	na	0.0001
OVERALL UNCERTAINTY	-1.11 (ft)	0.20 (ft)

Table M-5 (Continued). An Example Application of Vertical Inaccuracy Analysis for a Hypothetical Coastal Protection Project in Galveston, TX.

Notes on Table:
Bias Calculations: Since heights lower than designed are a concern, the total adjustment for worst case bias is: -0.70 –0.41 = -1.11 ft for the combined effects of terrestrial, and water level elevation bias.

Variance Calculations: The total adjustment for variance is the variance for the terrestrial elevations, the tidal datum, and the datum conversions. The overall standard deviation is $\sigma_T = \sqrt{0.0406} = 0.20'$.

b. Since some of the measurements will rely upon NGVD29 datum, some uncertainty can be expected from converting these values to NAVD88 datum required for the design heights. In the region covered by this project, this conversion is assumed to have a variance of 0.01^2 (ft^2), which is added to the overall variance used to compute the overall standard deviation.

c. The largest potential source of vertical inaccuracy in this project is relative sea level change. In Galveston, the tidal gage used for this project indicates a relative sea level increase of 2.1 ft/100 years, or 0.021 ft/yr. Since the assumed design life of this project is 50 years, the total bias expected over the life expectancy of this project is minus (-) 1.11ft.

d. The final potential source of vertical inaccuracy for the terrestrial datum in this project is conversion to the NSRS. Discussion with local experts indicates that this conversion has a variance of 0.20^2 (ft^2).

e. As previously described, risk assessments are currently conducted following the model and process described EM 1110-2-1619, "*Risk Based Analysis for Flood Damage Reduction Studies*" (August 1996). This is a general process that can be tailored to accommodate most flood risk management projects designed by the Corps of Engineers. However, the assumptions and inputs to this assessment are critical to obtaining accurate risk estimates. Vertical inaccuracy is typically a key input to a risk assessment. The risk of a flooding event is typically directly tied to the design height of flood risk management measures.

f. The overall bias and standard error of measurement can be used to set conservative design heights or depths in a project. The concern is that the design height be conservative enough to provide adequate protection over the project lifespan. If D represents the design height, the goal is to ensure that $D \geq D_{min}$, where D_{min} is the minimum design height needed to ensure the risk of flooding is less than a targeted level. If *B* is the overall bias calculated in Table M-5, which can be negative or positive, then in order to ensure that $D \geq D_{min}$ over the project lifespan with 95% confidence, calculate D using $D = D_{\min} + B - 1.645\sigma_T$, where σ_T is the total standard deviation from Table M-5.

g. For example, in the Galveston coastal protection project described above, the overall bias and standard deviation were -1.11' and 0.20', respectively. Hence risk assessment should be calculated using heights calculated by $D = D_{\min} - 1.21 - 1.645 \cdot 0.20 = D_{\min} - 1.54'$. In other words, at the end of design life of this hypothetical project, the terrestrial heights are estimated to be as much as 1.54' lower than originally designed. Estimated risks could be reported as a range using the value determined at the design height and a higher risk calculated using heights 1.54' lower.

h. Dredging and other projects where the goal is to ensure that the depth D is equal to or greater than a target depth are treated similarly but with an important change. The dredging depths need to be adjusted over time to account for trends in subsidence, rebound, or sea level change. The goal is still to ensure that $D_T \geq D_{min}$, i.e. where D_T is the dredge depth at time T and D_{min} is the minimum target depth. In this case, Table M-5 is calculated relative to time=T. The bias corrections for subsidence and rebound as well as the sea level change are calculated for time=T rather than using the minimum or maximum bias expected over the project lifespan.

i. In summary, heights are not static and in some cases can be inaccurate. Over the design life of a project relative sea level changes can increase or decrease the original design heights or depths for a project. Measurement error can also arise from datum conversions and inadequate coverage. These factors combine to cause uncertainty in vertical measurements assumed for an engineering project.

j. It is important to consider all factors and to attempt to estimate the magnitude of these factors by consulting local surveying and mapping experts. In some regions, this may require additional resources to accurately establish the magnitude of these effects. Existing reference monuments may need to be resurveyed, and in some cases new monuments in the project area may be needed.

M-12. Addendum A: Uncertainty Worksheet and Terms.

a. Project Risk Assessment: Vertical Uncertainty Worksheet. The following worksheet may be used for investigating and incorporating potential sources of vertical uncertainty in project risk assessments.

SOURCES OF VERTICAL UNCERTAINTY	Average Bias(ft)	Average Variance(ft^2)
TERRESTRIAL		
Subsidence(-)/Rebound(+)		
Measurement Uncertainty	na	
Datum Coverage	na	
National Spatial Reference System	na	
Other Sources of Uncertainty		
Total Terrestrial Adjustment		
WATER LEVELS		
TIDAL		
Sea Level Change		
Gage Accuracy	na	
Gage Proximity	na	
Project Epoch Adjustment		
Gage Historical Record	na	
Other Sources of Vertical Uncertainty	na	
Total Tidal Adjustment		
NON-TIDAL		
Low Water Reference Plane		
Gage Accuracy	na	
Gage Proximity	na	
Pool Elevation Record (length)	na	
Other		
Total Non-Tidal Adjustment		
DATUM CONVERSIONS		
NGVD29 to NAVD88	na	
NAD83 Ellipsoid to NAVD88	na	
Geoid Model Uncertainty	na	
Legacy Datum to NAVD88	na	
VDatum conversions	na	
Other	na	
Total Datum Conversion Adjustment	na	
OVERALL ADJUSTMENT		

b. Terms Defined.

(1) Subsidence (-)/Rebound (+). Measurements of subsidence and rebound can be determined using long-term accurate GPS observations as undertaken by the National Geodetic Survey's Continuous Operated Reference Stations (CORS) or through episodic re-surveying of a mark through Global Navigation Satellite System (GNSS).

(2) Measurement Uncertainty. The actual elevation measurement variance is known from long-term use of various leveling technologies. These include observations made using optical levels, digital levels, trigonometric techniques, and precise geodetic GPS observations. These include both orthometric elevation and ellipsoid elevation observations.

(3) Datum Coverage. Proximity to established NSRS control can affect the accuracy of elevation measurements. The distance between NSRS control, be it a benchmark or a CORS site, can add to the uncertainty of the observation.

(4) National Spatial Reference System. The current NSRS vertical datum is the NAVD88 datum. The adjustment that produced the published elevations of benchmarks included in the network vary across the United States. It is generally accepted that the adjustment had a vertical accuracy of five centimeters at the time of the adjustment.

(5) Other Sources of Inaccuracy. Unique field measurements may cause additional sources of uncertainty, which can be accounted for within this component of the computation.

(6) Sea Level Change. Sea level change is measured using long-term coastal water level observations and long-term satellite altimeter observations. Current data estimates the rate of change to be + 2-3 millimeters per year on a global basis.

(7) Gage Accuracy. Modern water level gages have a single observation resolution of 0.01 ft. Older float gages will have lower resolutions depending on age and model. Water level observations are designed to reduce the effects of high frequency wave energy and the result of the mean of a series of observations with a computed standard deviation. These observed variances can be used to compute overall uncertainty.

(8) Gage Proximity. Water level observations at a gage are accurate for the specific location of the gage. Using a gage to establish water levels some distance from the gage will introduce uncertainty. Uncertainty from gage proximity to the project should be ascertained.

(9) Project Epoch Adjustment. Tidal water level datums are computed using water level data observed over a 19-year epoch. Approximately every 25 years the National Ocean Service is instructed by Congress to update the tidal datums to a new epoch. Changes in datum elevation caused by updated epoch computation should be noted.

(10) Gage Historical Record. Gages used to compute tidal datums ideally use a complete record series of data. Tidal datums can be computed for gages with an incomplete data record by

using a nearby gage with similar tidal characteristics as a control. Gages with incomplete records will have unique variances, which should be noted.

(11) Low Water Reference Plane (LWRP). Non-tidal LWRP's are associated with stream gaging, the computation of stream profiles, stream stage, and discharge rates. The combination of vertical measurements for projects requiring vertical positioning relative to stream staging should be estimated using this component.

(12) Gage Accuracy. Stream gages vary with age and technology. A suitable accuracy for each stream gage should be assessed for this component.

(13) Gage Proximity. The proximity of a stream gage relative to a project site will dictate a higher uncertainty for determining elevations relative to the stream stage. For major projects, establishing a modern stream gage at the site of the project can reduce this uncertainty.

(14) Pool Elevation Record Length. Water storage reservoir, lakes, and pools, which fluctuate due to rainfall and seasonal effects, are subject to the length of record series. The computation of average stage elevation or similar may have uncertainty based on the length of the records.

(15) NGVD29 to NAVD88. Conversion of elevation from NGVD29 to NAVD88 is usually carried out using conversion software, namely VERTCON or CORPSCON or VDATUM (which incorporates VERTCON). These software programs are based on a national generalized model based on comparisons of known points in the NSRS. Uncertainty arises when a conversion is performed at a distant location, which is not in the adjacent vicinity to known points. These uncertainties should be assessed.

(16) NAD83 Ellipsoid to NAVD88. The following is from "*Converting GPS Height into NAVD88 Elevation with the GEOID96 Geoid Height Model*" by Dennis G. Milbert, Ph.D. and Dru A. Smith, Ph.D., National Geodetic Survey: "*Gravimetric geoid models show systematic departures from NAD83 GPS derived ellipsoidal heights at leveled benchmarks with NAVD88 orthometric heights. These departures are dominated by datum definition and datum realization problems. It is possible to fit these departures into a very smooth conversion surface, and add this surface to a gravimetric geoid model. For example, the GEOID96 geoid height model, which incorporates such a conversion surface, displays about 2.5 cm of accuracy (one sigma) between points spaced at 50 km or greater. GPS ellipsoidal height error of about 6 cm was observed after the computations.*" (Milbert and Smith, 1996)

(17) Geoid Model Uncertainty. The geoid is the equipotential surface of the Earth's gravity field, which best fits, in a least squares sense, global mean sea level. Several versions of the Geoid have been computed over the last three decades. Estimates of the accuracy of the Geoid vary due to the source and density of gravity data used to compute the Geoid. Current estimates show the Geoid accuracy to be approximately 20-30 centimeters for worst cases areas derived from sparse data sets. For details see http://www.ngs.noaa.gov/GEOID/geolib.html

(18) Legacy Datum to NAVD88. Due to the age and lack of documentation of legacy datums used by USACE, a conservative estimate should be used for this estimate of uncertainty. Local knowledge or professional judgment may be used to ascertain an appropriate level of uncertainty for the conversion of legacy datums.

(19) VDatum Conversions. See NOAA VDatum web site.

c. Flow Charts for Use in Assessing Risk Uncertainty.

(1) Flowchart for identifying terrestrial elevation inaccuracies.

(2) Flowchart for identifying tidal datum inaccuracies.

(3) Flowchart for identifying non-tidal datum inaccuracies.

(4) Flowchart for identifying datum conversion inaccuracies.

M-13. Addendum B: Definitions of Tidal Datums, Geodetic Vertical Datums, and the Relationship between Tidal and Geodetic Vertical Datums.
(From http://tidesandcurrents.noaa.gov/datum_options.html).

a. Tidal datums.

In general, a datum is a base elevation used as a reference from which to reckon heights or depths. A tidal datum is a standard elevation defined by a certain phase of the tide. Tidal datums are used as references to measure local water levels and should not be extended into areas having differing oceanographic characteristics without substantiating measurements. In order that they may be recovered when needed, such datums are referenced to fixed points known as benchmarks. Tidal datums are also the basis for establishing privately owned land, state owned land, territorial sea, exclusive economic zone, and high seas boundaries. Below are definitions of tidal datums maintained by the Center for Operational Oceanographic Products and Services.

MHHW* Mean Higher High Water	The average of the higher high water height of each tidal day observed over the National Tidal Datum Epoch. For stations with shorter series, comparison of simultaneous observations with a control tide station is made in order to derive the equivalent datum of the National Tidal Datum Epoch.
MHW Mean High Water	The average of all the high water heights observed over the National Tidal Datum Epoch. For stations with shorter series, comparison of simultaneous observations with a control tide station is made in order to derive the equivalent datum of the National Tidal Datum Epoch.
DTL Diurnal Tide Level	The arithmetic mean of mean higher high water and mean lower low water.
MTL Mean Tide Level	The arithmetic mean of mean high water and mean low water.
MSL Mean Sea Level	The arithmetic mean of hourly heights observed over the National Tidal Datum Epoch. Shorter series are specified in the name; e.g. monthly mean sea level and yearly mean sea level.
MLW Mean Low Water	The average of all the low water heights observed over the National Tidal Datum Epoch. For stations with shorter series, comparison of simultaneous observations with a control tide station is made in order to derive the equivalent datum of the National Tidal Datum Epoch.
MLLW* Mean Lower Low Water	The average of the lower low water height of each tidal day observed over the National

	Tidal Datum Epoch. For stations with shorter series, comparison of simultaneous observations with a control tide station is made in order to derive the equivalent datum of the National Tidal Datum Epoch.
GT Great Diurnal Range	The difference in height between mean higher high water and mean lower low water.
MN Mean Range of Tide	The difference in height between mean high water and mean low water.
DHQ Mean Diurnal High Water Inequality	The difference in height of the two high waters of each tidal day for a mixed or semidiurnal tide.
DLQ Mean Diurnal Low Water Inequality	The difference in height of the two low waters of each tidal day for a mixed or semidiurnal tide.
HWI Greenwich High Water Interval	The average interval (in hours) between the moon's transit over the Greenwich meridian and the following high water at a location.
LWI Greenwich Low Water Interval	The average interval (in hours) between the moon's transit over the Greenwich meridian and the following low water at a location.
Station Datum	A fixed base elevation at a tide station to which all water level measurements are referred. The datum is unique to each station and is established at a lower elevation than the water is ever expected to reach. It is referenced to the primary benchmark at the station and is held constant regardless of changes to the water level gage or tide staff. The datum of tabulation is most often at the zero of the first tide staff installed.
National Tidal Datum Epoch	The specific 19-year period adopted by the National Ocean Service as the official time segment over which tide observations are taken and reduced to obtain mean values (e.g., mean lower low water, etc.) for tidal datums. It is necessary for standardization because of periodic and apparent secular trends in sea level. The present NTDE is 1983 through 2001 and is actively considered for revision every 20-25 years. Tidal datums in certain regions with anomalous sea level changes (Alaska, Gulf of Mexico) are calculated on a Modified 5-Year Epoch.

*NOTE: Some locations have diurnal tides--one high tide and one low tide per day. At most locations, there are semidiurnal tides--the tide cycles through a high and low twice each day, with one of the two high tides being higher than the other and one of the two low tides being lower than the other.

b. Geodetic vertical datums.

The National Geodetic Survey (NGS) defines a geodetic datum as: 1. "A set of constants used for calculating the coordinates of points on the Earth." Generally a datum is a reference from which measurements are made. In surveying and geodesy a datum is a reference point on the earth's surface against which position measurements are made and an associated model of the shape of the earth for computing positions. Horizontal datums are used for describing a point on the earth's surface in latitude and longitude. Vertical datums are used to measure elevations or underwater depths.	
North American Vertical Datum of 1988 (NAVD88)	A fixed reference for elevations determined by geodetic leveling. The datum was derived from a general adjustment of the first-order terrestrial leveling nets of the United States, Canada, and Mexico. In the adjustment, only the height of the primary tidal bench mark, referenced to the International Great Lakes Datum of 1985 (IGLD 85) local mean sea level height value, at Father Point, Rimouski, Quebec, Canada was held fixed, thus providing minimum constraint. NAVD88 and IGLD 85 are identical. However, NAVD88 benchmark values are given in Helmert orthometric height units while IGLD 85 values are in dynamic heights. See International Great Lakes Datum of 1985, National Geodetic Vertical Datum of 1929, and geopotential difference. NAVD88 should not be used as Mean Sea Level.
National Geodetic Vertical Datum of 1929 (NGVD29)	A fixed reference adopted as a standard geodetic datum for elevations determined by leveling. The datum was derived for surveys from a general adjustment of the first-order leveling nets of both the United States and Canada. In the adjustment, mean sea level was held fixed as observed at 21 tide stations in the United States and 5 in Canada. The year indicates the time of the general adjustment. A synonym for Sea-level Datum of 1929. The geodetic datum is fixed and does not take into account the changing stands of sea level. Because there are many variables affecting sea level, and because the geodetic datum represents a best fit over a broad area, the relationship between the geodetic datum and local mean sea level is not consistent from one location to another in either time or space. For this reason, the National Geodetic Vertical Datum should not be confused with mean sea level. See North American Vertical Datum of 1988 (NAVD88). NGVD29 should not be used as Mean Sea Level. NGVD29 is no longer supported by NGS.

M-14. Addendum C: Estimation of Vertical Uncertainties in VDatum.

[This report section has been withdrawn from this Appendix M. It is a copy of a NOAA report (*Estimation of Vertical Uncertainties in VDatum*) found at http://vdatum.noaa.gov/docs/est_uncertainties.html, which was created in March 2009 and last revised in July 2009. This report section may also be viewed at the AGC web site http://www.agc.army.mil/ndsp--(*Uncertainty Model for Orthometric, Tidal, and Hydraulic Datums for Use in Risk Assessment Models, Phase 2 Final Report, dated 3 Sep 09*)]

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GLOSSARY

Abbreviations and Acronyms

1D .... One Dimensional
2D .... Two-dimensional
3D .... Three-dimensional
A-E.............................. Architect-Engineer
ACSM......................... American Congress on Surveying and Mapping
ADCIRC..................... ADvanced CIRCulation model
AGC............................ Army Geospatial Center
ARP .... Antenna Reference Point
ASCE.......................... American Society of Civil Engineers
ASPRS........................ American Society for Photogrammetry and Remote Sensing
BFE............................. Base Flood Elevation
BIH .... Bureau International Heure
c/c................................ Center to Center
CADD......................... Computer Aided Drafting and Design
CELW......................... Corps of Engineers Low Water
CEPD.......................... Comprehensive Evaluation of Project Datums
DFIRM .... Digital Flood Insurance Rate Map
DFO .... Department of Fisheries and Oceans
CFR............................. Code of Federal Regulations
CHL .... Coastal and Hydraulics Laboratory
CHS .... Canadian Hydrographic Service
cm .... Centimeter
CMAS......................... Circular Map Accuracy Standard
COEMLW.................. Corps Of Engineers Mean Low Water
COGO......................... Coordinate Geometry
CONUS. .... CONtinental United States
CO-OPS...................... Center for Operational Oceanographic Products and Services
CORPSCON............... CORPS CONvert
CORS.......................... Continuously Operating Reference Stations
COTS.......................... Commercial Off-The-Shelf
CSDL .... Coast Survey Development Laboratory
C&SF.......................... Central and Southern Florida [Flood Control Project]
CPT............................. Cone Penetration Test
CTP............................. Conventional Terrestrial Pole
CUBE .... Combined Uncertainty and Bathymetry Estimator
DA............................... Department of the Army
DEP............................. Department of Environmental Protection
DGPS.......................... Differential Global Positioning System
DOD............................ Department of Defense
DOT............................ Department of Transportation
EC .... Engineer Circular

ECEF ....Earth Centered Earth Fixed
ECL.............................Erosion Control Line
EDM ....Electronic Distance Measurement
EEZ ....Exclusive Economic Zone
EM ....Engineer Manual
EPA.............................Environmental Protection Agency
ER ....Engineer Regulation
ERDC ....Engineer Research and Development Center
ETL ....Engineer Technical Letter
FBN ....Federal Base Network
FEMA.........................Federal Emergency Management Agency
FGCC..........................Federal Geodetic Control Committee
FGCS ....Federal Geodetic Control Subcommittee
FGDC ....Federal Geographic Data Committee
FIRM ....Flood Insurance Rate Map
ft ....Foot
GEOREF ....Geographic Reference
GIS..............................Geographic Information System
GLONASS ....GLObal Navigation Satellite System
GNSS..........................Global Navigation Satellite System
GPS.............................Global Positioning System
GRS80 ....Geodetic Reference System of 1980
GUVD04 ....GUam Vertical Datum 2004
GZD............................Grid Zone Designation
HAT............................Highest Astronomical Tide
HC...............................Hydraulic Corrector
H&H ....Hydraulics and Hydrology
HARN ....High Accuracy Reference Network
HEC ....Hydrologic Engineering Center
HI. ....Height of Instrument
HPGN ....High Precision Geodetic Network
HQUSACE.................Headquarters, US Army Corps of Engineers
HSPP...........................Hurricane and Shore Protection Project
HTL ....High Tide Line
HWI ....Greenwich High Water Interval
HWM..........................High Water Mark
IERS............................International Earth Rotation Service
IGLD55 ....International Great Lakes Datum of 1955
IGLD85 ....International Great Lakes Datum of 1985
IHNC ....Inner Harbor Navigation Canal
IHO ....International Hydrographic Organization
IJC...............................International Joint Commission
IMSL...........................Instantaneous Meal Seal Level
IMU ....Inertial Measurement Unit
IPET............................Interagency Performance Evaluation Taskforce
ITRF............................International Terrestrial Reference Frame

ITRS............................International Terrestrial Reference System
IWW ....Intracoastal WaterWay
KTD ....Kinematic Tidal Datum
LCC ....Lambert Conformal Conic
LIDAR........................LIght Detection And Ranging
LMSL ....Local Mean Sea Level
LPCP...........................Local Project Control Point
LST ....Local Standard Time
LWD ....Low Water Datum
LWI.............................(Greenwich) Low Water Interval
LWRP ....Low Water Reference Plane
m ....Meter
MACOM ....Major Army Command
MDEQ ....Michigan Department of Environmental Quality
MGL ....Mean Gulf Level
MGRS.........................Military Grid-Reference System
MLG ....Mean Low Gulf
MHHW.......................Mean Higher High Water
MHHWL ....Mean Higher High Water Line
MHT ....Mean High Tide
MHW..........................Mean High Water
MHWL ....Mean High Water Line
MHWS ....Mean High Water Springs
MLLW........................Mean Lower Low Water
MLT............................Mean Low Tide
MLW ....Mean Low Water
MLWL........................Mean Low Water Line
MLWS ....Mean Low Water Springs
Mn...............................Mean Tide Range
MSC............................Major Subordinate Command
MSL............................Mean Sea Level
MTL............................Mean Tide Level
MVD...........................Mississippi Valley Division
MVN...........................Mississippi Valley Division, New Orleans District
MVS............................Mississippi Valley Division, St. Louis District
MWL ....Mean Water Level
NAD27 ....North American Datum of 1927
NAD83 ....North American Datum of 1983
NADCON ....North American Datum CONversion
NAVD88 ....North American Vertical Datum of 1988
NAVOCEANO ....US NAVal OCEANographic Office
NCD............................New Cairo Datum
NFIP............................National Flood Insurance Program
NGRS ....National Geodetic Reference System
NGS ....National Geodetic Survey
NGVD29 ....National Geodetic Vertical Datum 1929

NLD .... National Levee Database
NMAS......................... National Map Accuracy Standard
NOAA .... National Oceanic and Atmospheric Administration
NOS .... National Ocean Service
NRC............................ National Research Council
NSDI........................... National Spatial Data Infrastructure
NSRS .... National Spatial Reference System
NSSDA....................... National Standard for Spatial Data Accuracy
NTDE .... National Tidal Datum Epoch
NTE .... Not To Exceed
NWLON .... National Water Level Observation Network
NWLP......................... National Water Level Program
OCONUS .... Outside the Continental United States
OHWM....................... Ordinary High Water Mark
OHWP .... Ordinary High Water Profile
O&M........................... Operations and Maintenance
OPUS .... On-Line Positioning User Service
ORD............................ Ohio River Datum
PAGES .... Program for the Adjustment of GPS Ephemeredes
PASPCS...................... PennsylvaniA State Plane Coordinate System
PBM............................ Permanent Bench Mark
PC .... Point of Curve
PDF............................. Portable Document Format
PED .... Preconstruction Engineering and Design
PI................................. Point of Intersection
PID.............................. Position Identification
POB .... Point of Beginning
POS/MV .... Positioning and Orientation System—Marine Vessels (Applanix Corp.)
PPCP........................... Primary Project Control Point
PPK............................. Post-Processed Kinematic
ppm .... Parts per Million
PRVD02 .... Puerto Rico Vertical Datum 2002
P&S............................. Plans and Specifications
PT................................ Point of Tangent
QA............................... Quality Assurance
QC............................... Quality Control
RMS............................ Root mean Square
RMSE .... Root Mean Square Error
RTK .... Real Time Kinematic
RTN .... Real Time Network
SEMMS...................... Survey Engineering Monumentation Management System
SHWT .... Spring High Water Tide
SLD29......................... Sea Level Datum of 1929
SPCS .... State Plane Coordinate System
TBM .... Temporary Bench Mark
TCARI .... Tidal Constituent And Residual Interpolation

TEC............................Topographic Engineering Center
TGO...........................Trimble Geomatics Office
TIN.............................Triangular Irregular Network
TM ....Transverse Mercator
TPU............................Total Propagated Uncertainty
US ....United States
USACE ....US Army Corps of Engineers
U.S.C. ....United States Code
USC&GS ....US Coast & Geodetic Survey
USCG ....US Coast Guard
USED.........................US Engineer Datum
USGS.........................US Geological Survey
U-SMART ....USACE Survey Monumentation Archival and Retrieval Tool
USNG ....US National Grid
UT ....Universal Time
UTM ....Universal Transverse Mercator
VDatum ....(National) Vertical Datum
VDOP ....Vertical Dilution of Position
VERTCON ....VERTical CONversion
VIVD09......................Virgin Islands Vertical Datum 2009
VLBI...........................Very-Long-Baseline-Interferometry
VMAS.........................Vertical Map Accuracy Standard
VRN............................Virtual Reference Network
VRS ....Virtual Reference Station
VTDP..........................Vertical Time Dependent Positioning
WAVE........................Weighted Ambiguity and Vector Estimator
WGS 84 ....World Geodetic System of 1984
WRDA........................Water Resources Development Act
WS ....Water Surface (elevation)