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MAR 79 J B FORREST, J M FERRITTO
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A brief review of available analytical computer models dealing with groundwater flow is presented. Those models considered most appropriate for treating the subsurface flow regime beneath a drydock have been selected for application. Attention is also directed toward available soil-structure interaction computer models, and some of their shortcomings or deficiencies are identified.

This report singles out several critical categories for drydock analysis from a geotechnical viewpoint and recommends areas for further endeavor.

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INTRODUCTION

Currently, no clearly defined procedures exist for conducting a reliable evaluation of foundation conditions for drydocks. Naval Sea Systems Command (NAVSEA) requires formal accreditation of both Navy and commercial drydocks to be used for the maintenance and repair of Naval vessels.

This certification must comply with MIL-STD 1625A (Drydocking Facilities Safety Certification Criteria for Docking U. S. Navy Ships), but this document gives negligible recognition to the foundation support and groundwater flow requirements. It deals primarily with design acceptability, operational procedures, and the evaluation of visible structural components. The four major areas of documentation that must be submitted for each drydock facility certification are:

1. Design data
2. Survey of present material condition of the drydock
3. Operating procedures and personnel qualification procedures
4. Safety precautions for a ship while in dock

Experience has shown, however, that because of unknown or misunderstood soil conditions, faulty construction, inaccurate design assumptions, or other factors, drydocks do not always function as intended. Although such structures can accommodate some deviations, serious and costly failures have occurred. Perhaps of equal importance is the uncertainty and lack of assurance of ship safety that can be given to Navy commands responsible for the ships that must remain in drydock for extended periods of time. Foremost among these ships are the nuclear submarines which may be vulnerable during long periods of very sophisticated overhaul in both Navy and commercial docks.

The large buoyant or hydrostatic uplift forces that tend to act upon a dewatered drydock must be opposed either by huge gravity sections (fully hydrostatic), or else the uplift pressures must be reduced. Thus, it is often more economical to design the drydock to either be fully relieved, wherein the hydraulic head beneath the drydock is held at essentially floor level; or partially relieved, that is, drydock stability is attained by a combination of gravity sections and sections (floor) with reduced hydrostatic pressures. It is noted however, that even drydocks that are defined as being totally relieved may have some portions, such as the pumphouse or the entrance sill, which rely upon gravity loads to maintain their stability. Thus, in addition to failures due to loss of foundation support, failures in the subsurface drainage system could lead to such things as floor blow up, uncontrolled flooding, failure of the dock walls, and freezing of the entrance caisson (due to jamming). Subsurface erosion around the periphery of the drydock could also lead to failures of the adjacent slabs or crane rails. These difficulties could be a result of such things as breached flow cut-offs, clogged drainage blankets, improperly functioning pressure relief systems, the presence of erosion cavities or "pipes," or other causes.

Development of a systematic, defined certification routine is necessary in order to minimize the individual judgment factor in ascertaining drydock foundation reliability.

Criteria must be established defining acceptable levels of performance and permanence of drainage blankets, perforated piping, filters, valves, relief wells, cut-off walls. As with other geotechnical problems, it will be necessary to combine both direct and indirect information (such as construction history) to arrive at a reliable evaluation of the situation. This report represents the initial efforts toward this pursuit. It attempts to define the problem, presents some background information on why the problem exists, and suggests directions to be pursued.

Work is being directed primarily along two different avenues. One deals with the identification, adaptation, or development, if necessary, of improved reconnaissance or in situ evaluation technology. Requirements include (1) techniques for locating voids or cavities beneath or adjacent to drydocks and (2) procedures for detecting excessive hydraulic gradients or anomalous seepage pressures.

The second area of concentrated effort is that of developing analytical computer models of drydocks, with particular regard to the groundwater and hydraulic pressure regimes.

DRYDOCK PROBLEMS

This report deals only with that type of stationary drydock often referred to as a graving dock. It, therefore, does not concern itself with such structures as floating drydocks, lift docks, marine railroads, or similar structures. A list of United States drydocks involved is presented in the Appendix.

As noted in the foregoing section, numerous problems can occur during operation of these kinds of docks. The intent in this section is to concentrate on the geotechnical problems; i.e., those related to foundation support and groundwater flow systems. Problems may manifest themselves in terms of: irregular pressure distributions and flow variations due to the development of voids; large settlements, either under ship blocks or under adjacent crane rails; floor blow up; wall or floor slab collapse; entrapment of the caisson gate; uncontrollable water; or soil and water inflows. Such problems may be due to: loss of drainage blanket, loss of subsurface support due to erosion; breached flow cut offs such as split sheeting; disablement of pumping or dewatering system valves or pumps; blockage of underdrains; drainage lines; filters; and drainage blankets.

In the past, a large number of items associated with drydocks have threatened the overall facility. For example, failure of a steel sheet piling wall at Brooklyn Naval Shipyard caused a rapid influx of soil during construction. Faulty Y-connections have led to minor failures and temporarily threatened the integrity of the Trident submarine drydock during the construction phase at Bangor, Wash.

Failure of an unwatering culvert line occurred at Drydock no. 8 at the Norfolk Naval Shipyard; and floor failure in a utility tunnel was experienced at Drydock no. 1 at the Long Beach Naval Shipyard.

Numerous other incidents of drydock or drydock-associated failures have occurred. Subsidence of the surrounding pavement and crane rails is a continuing problem. The subsidence may be long-term deformations, as with Drydock no. 5 in Philadelphia; or dramatic, as in the cases of Pascagoula, or at Drydock no. 2 at Norfolk Naval Shipyard on 16 Nov 1971.

Specific Locations

Pascagoula, Miss. One instance where apparent loss of drainage blanket led to serious consequences was at Pascagoula, Miss., on 22 Feb 1972. Failure occurred at a Foster-type, "semirelieved," graving dock of the Ingalls Shipbuilding Division (Ref 1).

That drydock is constructed with gravity-type, steel sheet pile, cofferdam sidewalls with a pressure-relieved floor slab. The design relies on cutting off the major inflow of groundwater by the sheet pile sidewalls so that a network of drain piping under the pile-supported floor can be used to collect and pump out the remaining inflow and thereby relieve the hydrostatic pressures. Just after construction in Aug 1971, the sand and gravel fill in one cell (cell S-3) suddenly subsided. In Feb 1972, the sand and gravel fill in another cell near the gate abruptly subsided, at which time a major leak developed inside the dock through floor vents near the gate.

The original construction called for placing sheet pile interconnecting cofferdams around the drydock periphery, dewatering the interior, and building the floor. Temporary cofferdam cells at the location of the entrance gate were to be burned off at the sill slab level and the gate installed. Unfortunately, a watertight seal was not obtained between the 18-inch-thick concrete sill slab and the temporary cells. This permitted water (under about 40 feet of differential head, during the unwatered drydock condition) to reach the underside of the drydock floor without being directed around the sheet pile cutoff. It was also discovered that holes drilled in the sill slab immediately inboard of the temporary cells (for dewatering well points during construction) had not been adequately sealed off, permitting another path for water access beneath the floor slab. This, combined with the relatively short lengths of steel sheet piling in some of the cells, permitted high hydraulic gradients. This caused some of the fine subsoil materials to be washed through the coarser filter bed, leaving cavities that eventually resulted in subsidence of the filter itself and destruction of the porous pipe drainage system embedded in it.

This, in turn, led to the massive discharge of water and soil in the vicinity of the drydock gate on 22 Feb 1972. This influx of soil and water entered initially through vents and at the floor-cell intersections and, upon closure of the vents, began to discharge out of the intake sumps connected to the porous underdrain system beneath the floor.

Pressure-reduced drydocks can be particularly vulnerable in that direct communication may exist between vents in the dock chamber and the underside of the dock floor through the filter systems. The filters are commonly subjected to reversal in hydraulic gradients which can damage them to the extent that filter material can be lost into the dock chamber. When this happens, a situation occurs such as that at Pascagoula.

Major drydock problems can result from failure to remove undesirable materials adjacent to or within the structure. The subsidence which occurred in cell S-3 prior to the major Feb 1972 failure at Pascagoula was found to be due to the existence of soft materials between elevations -45 and -47 within the cell. This soft material underwent consolidation, resulting in fill settlement with attendant breaking of the surface pavement, leaning of utility poles, and other damage.

Norfolk, Va. In the Norfolk case, the failure was indirectly caused by an unrecorded opening cut in the sheeting of an earth-retaining structure adjacent to the drydock. The failure was precipitated by a submerged hole cut through the concrete wall of the pumphouse during construction work. Because of the unrecognized opening in the steel sheet piling some distance from the pumphouse, the free water outside the adjacent earth-retaining structure was almost directly connected to the hole in the pumphouse wall. In this case, the very high hydraulic gradients permitted a "blow-in," in which soil and water blew into the pumphouse immediately upon completion of the cut. This blow-in removed foundation support from beneath the slab and crane rails adjacent to the drydock and resulted in a major depression. This fairly rapid subsidence seriously threatened critical vessel servicing equipment.

Long Beach, Calif. A rather unique drydock certification problem exists at Long Beach Naval Shipyard where groundwater depletion associated with oil production has resulted in about 29 feet of overall subsidence, about 15 feet of it directly beneath Drydock no. 1. Increased hydrostatic pressures forced the placement of 16 hydrostatic relief wells in 1946, resulting in a partially relieved dock. Although subsidence ceased during the 1960's, groundwater recharge operations have placed the drydock in danger of hydrostatic uplift once again. Another very serious threat to the Long Beach drydocks is that of earthquake-induced liquefaction.

Drydock no. 1 appears to be located on dense silty sand and clayey silt overlying the dense granular Gaspar aquifer which underlies the Port of Long Beach, Calif. However, the drydock is surrounded by up to 60 feet of loose to very loose silty, sandy material that could be expected to liquefy under relatively moderate (around one tenth the acceleration of gravity) earthquake shaking. Precisely what would happen to Drydock no. 1 upon liquefaction of this loose material is open to conjecture, but the results could be serious. Aside from purely structural considerations such as failure of the dock walls, the effects of liquefaction upon the pressure-relieving system are expected to be very severe. This could lead to flotation of the dock.

Other Factors

Many lesser incidents or difficulties have occurred in drydock operation that can pinpoint areas of potential problems; however, a history of such occurrences is difficult to compile. These problems may also be associated with hydraulic structures other than drydocks but may have modes of failure that are nevertheless relevant to drydock performance. For example, one area of potential problems occurs where earth fills are in contact with concrete structures. Unless special seals are provided, seepage will concentrate along the fill-concrete interface and could result in piping. Such a mechanism was responsible for failure of the cofferdam for the Cannelton Dam on the Ohio River (Ref 2). Where cofferdams are concerned, any failures can generally be expected to occur during construction. However, minor interlock anomalies combined with faulty sheets can, in the situation of excavation or scour of overburden, result in progressive interlock failure. It is important to note the presence of water pipes, conduits, and other structures, which, upon failing or rupturing, could saturate (or scour) an area and exceed design pressures. Weep holes in enclosed cell structures must also be maintained where water level fluctuations are possible.

Drainage Valves. One problem with Foster-type drydocks in particular and all drydocks to some extent is the deterioration of drainage valves in the cofferdam cells. Malfunction of these drainage valves in cofferdam sidewalls often permits loss of fill materials from the cells during dewatering. Should these valves become sealed, on the other hand, then the cofferdam cells tend to "work"; that is, expand and contract with the change in lateral pressure during each dewatering and flooding cycle. This can result in fill surface subsidence and associated paving collapse.

Through-Floor Vents. Experience with pressure-relieved drydocks with through-floor vents suggests that these docks should not be left flooded for extended periods of time. Tidal fluctuations can, in these cases, result in dislodgement of filter materials and eventually lead to erosion of the foundation.

Lack of Adequate Information. Even drydocks that have performed successfully for many years can still provide potential certification difficulties. For example, Drydock no. 1 at Mare Island Naval Shipyard, in Vallejo, Calif., was constructed in 1891, and no original design calculations are available. In cases like this, it is necessary to assume that the structure was designed according to the criteria of that time. Soil reports on tests which might be used for confirming allowable bearing pressures on the clay foundation base are also not available.

This drydock contains an older gravity section whose unreinforced floor could undergo excessive bending stresses in the dewatered condition (Ref 3) under full hydrostatic base pressures. Relief wells placed in 1969 appear to be successfully providing hydrostatic pressure relief, however. Based upon over 75 years of successful operation, it is reasonable to assume that this drydock is adequate. Nevertheless, the dock is allegedly designed for a maximum ship weight of 14,000 dwt, and to date the heaviest ship serviced has been less than 7,000 dwt. Thus, even past history is not really adequate for certifying this dock for its designated capability.

DRYDOCK MEASUREMENTS

The current program for safety certification of drydock facilities as defined by MIL-STD 1625A (SH) calls for a site examination as well as supplementary measurements, but guidelines regarding foundation support measurements are not specifically defined. It is apparent that procedures for measuring subsurface characteristics of drydocks are not yet sufficiently developed, and this represents an area of potential research. This problem is further aggravated in the case of older drydocks where original design calculations are missing. In cases where original soil foundation data are not available, such as with Drydock no. 1 at Mare Island Naval Shipyard, measurements of subgrade stiffness and in situ soil response are necessary before a valid analysis of drydock stresses is possible. In the traditional safety analysis, a structure is defined by the geometry, and the propagation of external and internal forces (i.e., stresses) throughout the structure is considered. From this, the deformational behavior the structure will undergo can be predicted. This procedure is made considerably more difficult in foundation engineering in that most natural geological materials may change with time. These changes are even more dramatic for structures such as drydocks where the hydrodynamic conditions are perpetually changing due to dewatering-filling procedures superimposed, in many cases, upon hydraulic pressure relief operations.

From a safety point of view, evaluating the capability of a structure to perform under its service loads requires only measurement of deformations. However, in order to insure that analysis procedures and individual material response characterizations are valid, load measuring must be resorted to as well. Selection of load measuring devices is generally based upon three criteria:

1. Costs involved
2. Environment in which it is to be used
3. Nature of the application

However, in the case of a drydock, it is presumed that the selection of an instrument would be primarily controlled by environmental factors, including accessibility. This means that with load cells, for example, the vibrating wire type would probably be most suitable because of its stability and durability. Such devices, when used for load measurement, must generally be incorporated into the structure during initial construction. Unfortunately, certification generally deals with drydocks already in service.

Even in cases where extensive field instrumentation has been available, structure failures have occurred because a quantitative description of ground conditions was poor. Thus, before such field instrumentation is installed, the geological picture must be understood. The importance of long-term reliability should be noted; piezometers of the pneumatic type are probably most applicable here. Many recent improvements have been made along these lines. Perhaps the most reliable measuring devices incorporate a mercury manometer.

Although piezometers have been traditionally used to measure water pressure only, more recently the capability of measuring in situ permeabilities has been recognized and utilized. The device used is referred to as a hydraulic piezometer (Ref 4). The principle used is that flow must take place to establish equilibrium between the piezometer pressure and that in the surrounding soil. The time rate of this flow is a direct function of soil permeability. For simple, free-flow piezometers, Hvorslev (Ref 5) has established analytical relationships between time of pressure equilibrium and permeability.

This procedure for deducing permeability from measured flows into or out of piezometer tips under various applied pressures initially assumed a rigid soil skeleton. Thus, with clays or other materials that underwent consolidation or swelling, these initial determinations of permeability were in error. Today some analytical procedures also provide values of the coefficient of consolidation, or swelling (Ref 6). During measurements of permeability (or simply circulating water to de-air piezometers) care has always been taken to keep applied water pressure at the tip less than the overburden pressure (critical pressure), so that a water pocket would not be formed in the soil around the piezometer.

Studies conducted by Vaughan (Ref 7) for remedial work on the clay core of the Balderhead dam showed that by slowly increasing the water pressure at the piezometer tip, the flow into the soil remained low until a critical pressure was reached, when a progressive increase in flow with pressure was observed. With the further increase in pressure, a sudden increase in flow was noted, indicating soil rupture. This critical piezometric pressure could be either less than or more than the apparent overburden pressure, dependent upon effects such as arching or overconsolidation (increased lateral pressures) or other phenomena.

During a subsequent reduction in pressure, a pressure was encountered at which a suddenly reduced outflow signified closing up of the fissure. Most importantly, measurements of permeability before and after the soil rupturing indicated that no permanent damage had been caused by forcing a small quantity of water into the soil. Although the foregoing work applies to compacted clays, it nevertheless suggests an approach not only for measurement of permeabilities in situ, but for possible detection of underground voids even in soils other than clays. For example, in the vicinity of voids, hydraulic piezometers could indicate critical pressures markedly below what might be considered reasonable.

A number of developments in instrumentation have occurred during the past several years involving measurement of not only settlement profiles, but lateral deflection profiles as well (Ref 8). Instrumentation is now commonly used to measure deflection profiles, individual pile loads, soil contact pressures, fluid pressures, and other factors. Also, strain meters having various degrees of sensitivity, e.g., inclinometers may use wheatstone bridges, photographic techniques, or servo accelerometers (such as Digitilt by Slope Indicator Co.). However, only during the past decade have serious attempts been made to record the real behavior of rigid-type retaining structures.

Most of the inadequacies between predicted and actual performance are due to inadequate field measurements. Recent improvements in analytical and instrument technology are placing more emphasis in improved measurements. Because of the major shortcomings in conventional sampling and testing approaches, there is a current trend toward instrumentation of actual structures, not only to assess safety under service conditions but also to gain behaviorial knowledge of the basic material. Nevertheless, it is universally accepted that knowledge of the real behavior of field structures is, in general, inadequate. Consequently, the engineering profession must promote and finance systematic studies into an evaluation, as well as prediction, of behavior of real structures (Ref 9).

ANALYTICAL MODELS

The analysis of a drydock - in order to be complete - must include the various conditions which could cause failure: static and dynamic application of loads, variation in water elevation, failure of drainage devices (pumps, drains, etc.), breach of cutoffs, and deterioration of structure or soil foundation. It becomes obvious that many failure modes and interactions are possible; no single analytical technique can begin to fully and accurately represent the actual structure. Some assumptions and simplifications must be made to attempt to segment and model the problem.

Static Soil-Structure Analysis

Several alternatives can be used to analyze structures to determine foundation interaction. The simplest uses beam column analysis representing the soil as equivalent springs (Ref 10, 11). Nonlinearity of soil behavior may be approximated by parabolic soil stiffness functions. More complex procedures are available such as finite element analysis techniques. The concrete drydocks may be modeled accurately; however, the interface between soil and structure is the key to satisfactory results. Some codes incorporate specific nodal tie elements to allow nonlinear slippage. Static analysis of total stress is available within the present state-of-the-art. New material models are being incorporated which allow for better nonlinear representation of soil elements.

Dynamic Soil-Structure Analysis

Some of the finite element codes used in the static soil-structure analysis are capable of treating total-stress dynamic analysis by utilizing a time history ground acceleration. Coupled with the total stress analysis are supplemental programs which can generate pore pressure histories in an approximate uncoupled manner (Ref 12 through 16). This is most useful in evaluating seismically induced soil liquefaction. Several researchers are developing effective stress material models for implementation into two- and three-dimensional finite element codes. This work is in the research stage and has not been implemented as yet (Ref 17 through 20).

Seepage

The seepage analytical technique is perhaps the most developed of the several discussed. Full three-dimensional finite element and finite difference codes are available that can model the most complex geometry and determine fill surface location and water flow rates under transient and steady state conditions (Ref 15, 21). Drains, pumps, and cutoffs may all be represented. In this manner, the seepage conditions and possible seepage failure modes under a drydock can be evaluated. Although the technology is available, the types of problems studied have been limited to major structures such as dams. Available groundwater flow models have been studied and the optimum computer model selected for use in drydock analysis. The results of this study are presented in Reference 22.

Consolidation

Programs are available for one-, two-, and three-dimensional consolidation. This procedure analyzes the generation and dissipation of pore water pressure established by a foundation loading or change in water conditions. Finite element techniques are available which form a suitable solution by application of a generalized variational principle to the static equations of equilibrium solving for displacements and pore pressures (Ref 23).

Finite Element Modeling

The earth-structure interaction problem has been of great importance in geotechnical engineering. The interaction of a retaining wall with a frictional soil backfill was studied by Coulomb (Ref 24) in 1776 and Rankine in 1857 (Ref 25). Their work is in use even today. Both theories assume the earth as a rigid-plastic mass material governed by Mohr-Coulomb failure and the structure as rigid. Terzaghi (Ref 26) and others demonstrated limit conditions and the dependence of earth pressure on the model of wall deformation and flexibility. Empirical and semi-empirical techniques are generally used in design of many earth-support systems giving a conservative estimate of loading and deformation. Unfortunately, for structures not so commonly encountered (such as drydocks), the problems are more complex.

The finite element approach offers the ability to simulate in a better manner soil behavior and boundary conditions. As a result, this approach is increasingly becoming a main tool in sizable geotechnical projects. However, idealizations of the problem must be made. These idealizations often entail less than a full three-dimensional analysis and some compromise at boundaries. The retaining wall soil interface is a complex nonlinear junction which must behave differently in tension than in compression; slippage and friction may be paramount to realistic solutions. Numerous material models will simulate some aspects of soil behavior such as nonlinearity, time dependence, dilation, and shear-volume change effects. However, often a material model may represent one phenomenon or type of test data and completely miss another. The choice of material model is critical to a valid solution. If shear induced deformations are of major significance, then a curve-fit model using volumetric stress-strain data will not produce satisfactory results; or, if the problem involved mainly volume change, use of empirical-fit shear modulus parameters might not be suitable. Each model presents certain capabilities and certain complexities. The more test data types capable of being simulated, generally the greater the capability, but also the higher the complexity, cost of use, and increased data input required.

Clough and Duncan (Ref 27) performed an analysis of U-frame locks which are very similar to drydocks. These locks, Port Allen Lock and Old River Lock, were constructed in the early 1960's by the Army Corps of Engineers. Cross sections through the locks and essential dimensions for the locks are shown in Figure 1. The instrumentation for each lock consisted of earth pressure cells, concrete strain meters, heave plugs, settlement reference points, bolts and plates, piezometers, and wall deflection pipes.

The finite element analysis closely followed construction sequence. Figure 2 shows the simulation of the different phases of construction being modeled. The analysis determined settlements during construction (Figure 3); results agreed very closely with observed data. A nonlinear elastic model used for the soils incorporated a separate formulation for the bulk and deviatoric components of the soil stiffness and accounted for the effects of confining pressure, stress history, and shearing stress level. Drained tests were used to evaluate the parameters for the foundation and backfill soils.* One-dimensional elements represented the interface between the soil and the lock, the properties of which were determined in the interface tests conducted with samples of the Port Allen Lock backfill sand and smooth concrete.

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*All of the soils for both locks were silty or sandy, and the time required for construction of the locks gave adequate time for drainage of excess pore pressures.

Figures 4 and 5 give predicted and observed structural deflections and earth pressures. The excellent agreement could only have been obtained by adherence to the exact construction sequence in the simulation.

DEFINITION OF AREAS NECESSARY FOR CERTIFICATION

As suggested previously, any satisfactory procedure for drydock certification will have to encompass directly measurable or recorded design information, as well as indirect information, such as observations made during original construction or during subsequent operations. For example, at Pascagoula, during construction of the Foster-type dock, it was noted that the contractor experienced unusually soft driving conditions in the sheet pile cells. Toward the end of construction, some leakage was also observed from floor vents just inboard of the gate. This knowledge, combined with design information and analysis, provided a preview of some of the subsequent problems experienced. From the geotechnical aspect, several critical items for certification can be singled out. These items might best be treated under the following categories:

1. Applied Loads - Investigate supports for static loads of dock, ship, cranes, and equipment
2. Foundation-Structure Interactions - Assure that sidewalls and adjacent entrance walls are not overstressed by hydrostatic and local soil pressures
3. Hydrostatic Relief and Drainage Systems - Assure that all drainage and pressure relief systems are in sound condition
4. Subsurface Erosion - Detect any subsurface voids or erosion channels adjacent to the dock
5. Deformation-Related Problems - Assure that deformations do not interfere with any drydock operations
6. Earthquake Effects - Assure stability during seismic loading

These categories will be discussed with regard to basic descriptions and engineering design data, field examinations and measurements, and applicable analysis.

1. Applied Loads

Basic data herein would include a description of wall and floor foundation type, floor thickness, spacing, size and type of piling, design floor loading, and design pile loading. A description of the soil layers beneath and adjacent to the dock would be required, including stratum thickness, groundwater levels, and soil characteristics. Elevations of soil and foundation bearing levels and pile tips relative to the soil layers depended upon to carry the loads should be detailed. Loading conditions on the dock and all associated crane tracks and equipment should be defined.

Field inspections should be conducted in conjunction with detailed knowledge of past drydock performance. In addition to inspection of obvious cracking, settling, or yielding of the dock or adjacent pavements and crane rails, all deformations should be precisely measured. This should include, for example, comparisons between measurements at dock-empty and dock-flooded stages. Supplementary field measurements might include current soil explorations and testing programs, which might entail test pits along crane tracks to determine the condition of supporting piling, geophysical testing, and other types of nondestructive soil testing. Sonic, geophysical, or other techniques can also be used to check the soundness or, if necessary, the extent of foundation materials and sheet piling. Estimates must be made of the condition of all these structural items as well as the expected rates of deterioration.

2. Foundation-Structure Interactions

This item is concerned with insuring that the sidewalls, entrance, and other structures are not overstressed by either soil or hydrostatic pressures. Basic engineering requirements include properties of the backfill materials behind the sidewalls and walls or bulkheads adjacent to the dock entrance and within cells of cofferdams. It is necessary to denote the location, dimensions, and properties of wall drainage systems, filters, and piping. The location, dimensions, and characteristics of special hydrostatic control wells or drains must be stated. This information permits analysis of the soil and hydrostatic loading on walls and bulkheads, of the stability of cells, and of other characteristics.

Care must be taken during inspection to observe such items as wall tilting, free flowing leakage, openings in or bulging of cell walls.

Entrance bulkheads must be given special attention to detect problems such as excessive erosion or loss of backfill material. Infiltration rates should be estimated and the proper functioning of cell drainage systems assured. Additional testing can include penetrometer probing of filled areas and measurements of piezometric and soil pressure gradients behind walls and cells. Again, precise measurement and analysis of wall displacements between flooded and empty dock stages can provide valuable insight toward drydock condition.

3. Hydrostatic Relief and Drainage Systems

The locations, dimensions, and engineering properties of all sub-drainage features must be defined, including subfloor blankets, filters and pipe drains, wall backfill drainage, and any special hydrostatic control or relief wells. Tip elevations of cut-off walls, cofferdam, or bulkhead sheet piles must be identified in relation to the soil profiles.

All manholes, pipes, and drainage passages must be clean and properly aligned. Evidence of any gradual reduction in seepage from wall or floor drains must be noted, since this could result from clogging or some other form of malfunction.

Hydrostatic heads and pressure gradients beneath the floor and behind walls should be determined, using piezometers, for both dock-flooded and dock-empty conditions. Measurements made during pumping from particular drainage wells can be compared with analytical results of the flow regime beneath the dock. In this way, any anomalies in pressure gradients under the sidewalls, the entrance sill, or other areas can be recognized. All records must be maintained for comparison to permit estimates of rate of drainage system deterioriation.

4. Subsurface Erosion

The danger posed to a hydraulic structure (such as a drydock) by underground erosion or piping warrants special consideration. It is critical that no incipient flow channels or voids develop beneath or around a drydock. This could lead to uncontrolled flooding or complete loss of support for the dock and for adjacent work areas. Construction records must be studied in detail with regard to compaction control, design and gradation of filters, filter thicknesses, size of pipe perforations, and other information. All this information must be considered in relation to the grain size and gradation of the local soils and backfill materials. It must be determined whether or not all construction dewatering systems have been removed or sealed up.

Attention must be directed toward any evidence of sediment accumulating in drainage and filling tunnels or seepage collector pipes or of sediment moving through cracks. Analysis of inflowing water for suspended or dissolved solids can be a valuable indicator of drydock condition. Any progressive increase in the quantity of seepage must be noted. Localized settlements of paved areas around the dock can be an indicator of soil loss or voids. Examination of the harbor bottom near the drydock entrance for evidence of holes, craters, heaves, or other anomalies is valuable. Voids beneath the floor may be detected through the vent holes, but generally nondestructive detection of voids beneath the substantial concrete thickness of a drydock floor is an unresolved problem.

Field measurements may include borehole permeability tests and pumping tests in selected areas with measurements of flow rates. Use of dyes, isotopes, or measurement of chloride content can be used to detect sources of seepage flows. Bathymetric surveys of the entrance bottom and adjacent areas can suggest any deteriorating conditions. Pressure head measurements and pumping tests can be used together with an analysis of the flow regime around and beneath a dock to detect any anomalies in flow paths or hydraulic gradients.

5. Deformation-Related Problems

It is important to insure that settlements and earth movements adjacent to the drydock do not adversely affect the safety of equipment operation. Basic information here includes surface and subsurface soil profiles, data on possible negative skin friction acting on piles and walls, and estimates of possible subsidence or distortion levels. The causes and nature of any soil movements must be determined and their significance evaluated. This aspect of drydock certification entails placement and monitoring of bench marks for both vertical and lateral movements. Use of inclinometers and deformation gages are valuable in this regard. Piezometers and soil stress measurement combined with theoretical analysis can be valuable in evaluating structure response.

6. Earthquake Effects

Because many drydocks are located in seismic regions, such as the West Coast, the central Atlantic coast, or even the northeastern Great Lakes area, it is necessary to assure that the dock and associated crane rail foundations and other structures can withstand anticipated earthquake effects. Basic information required here is knowledge of adjacent native soils and fills and the geometry, stiffness, and mass of involved structures.

A major factor for consideration is earthquake-induced soil liquefaction which could destroy not only pressure relief and drainage systems but also the dock itself. The stress conditions around a drydock during seismic loading can be complex, making it difficult to predict liquefaction potential in all but the more extreme cases. Where limited liquefaction can be expected to occur at depth, the results on the dock and upon the adjacent facilities are also difficult to predict. Even in cases where liquefaction may not be a problem, such as beneath the drydocks at Mare Island which are allegedly founded on a stiff clay, problems regarding dynamic lateral pressures arise. Current analysis procedures for handling dynamic soil pressures are empirical quasi-static approaches. Although sophisticated dynamic computer code analysis is possible, there is reason to suspect the reliability of these codes with respect to the soil-structure interfaces (Ref 28).

Thus, the ability to analyze the resistance of the dock walls, entrance walls, floor support piling, and other structures to ground acceleration forces must be dramatically improved. This problem also applies to flooding tunnels, major buried conduit and utility connections, and their supports.

One possible source of information on this aspect of drydock certification would be reviews of historical records. This may provide some correlations between facility response and previously noted ground motions and other earthquake effects.

The research needs can be treated in terms of two major areas: (a) reliable, expedient field procedures to evaluate the condition of the soil with regard to its behavior under seismic excitation and (b) validated, analytical techniques for predicting the response under any specified level of ground motion.

FUTURE WORK

Immediate plans for continuation of this work during Fiscal Year 1979 are primarily directed toward two areas. The first includes an in-depth study of graving drydock features. A detailed survey of graving docks will be carried out for two reasons: (1) to determine the relative prominence of various drydock features and (2) to provide a better perspective on functional problems. This survey will permit formulation of a tentative outline for establishing acceptance criteria and a certification format. This survey will serve as a basis for selecting the most appropriate graving drydock for further detailed analysis. It will provide an enhanced definition of those aspects of drydocks most often leading to problems, and hence aid in defining acceptance limits.

The second area of study deals with analytical treatment of the groundwater flow regime. This will comprise development of a generalized subsurface flow model for application to graving drydock analysis and adaptation of this model to a specific situation; i.e., the drydock selected earlier for detailed analysis. This will permit an in-depth study of a specific graving drydock, and this defined drydock can then be used for validation of any proposed certification techniques or analytical procedures.

Work will also be continued on two other items but at a reduced level, with only enough effort expended to keep informed of developments in the respective fields. These areas are analytical soil-structure interaction computer codes, and technology for detection of subsurface voids.

In the computer code area, attention will be directed toward obtaining the most recent developments in finite element stress analysis and in exercising these codes at CEL to evaluate their applicability and reliability.

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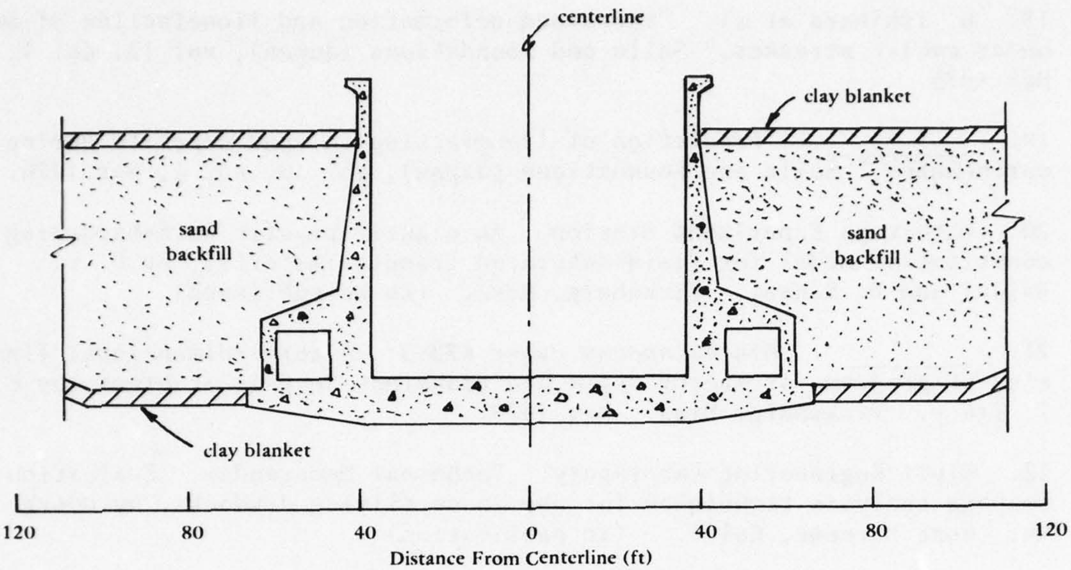
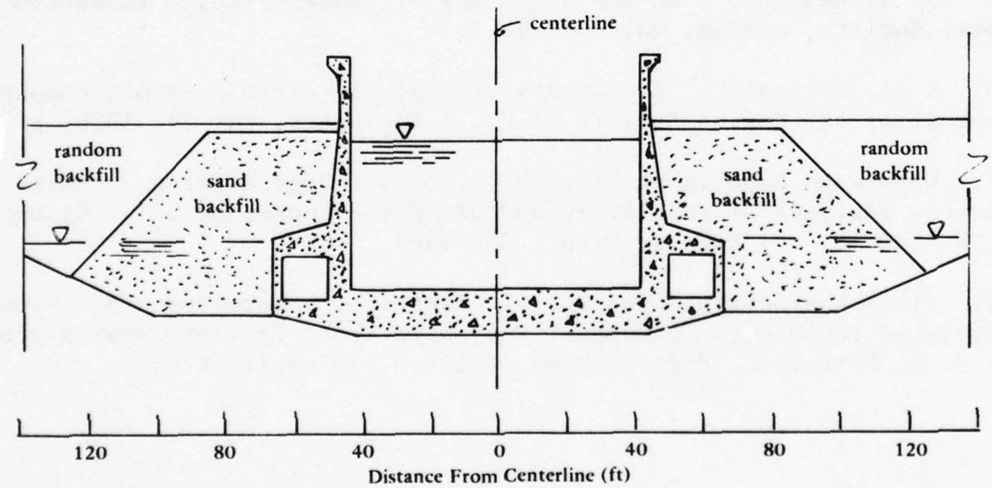


a. Cross section of Old River Lock.



b. Cross section of Port Allen Lock.

Figure 1. Cross sections of Old River and Port Allen Locks.

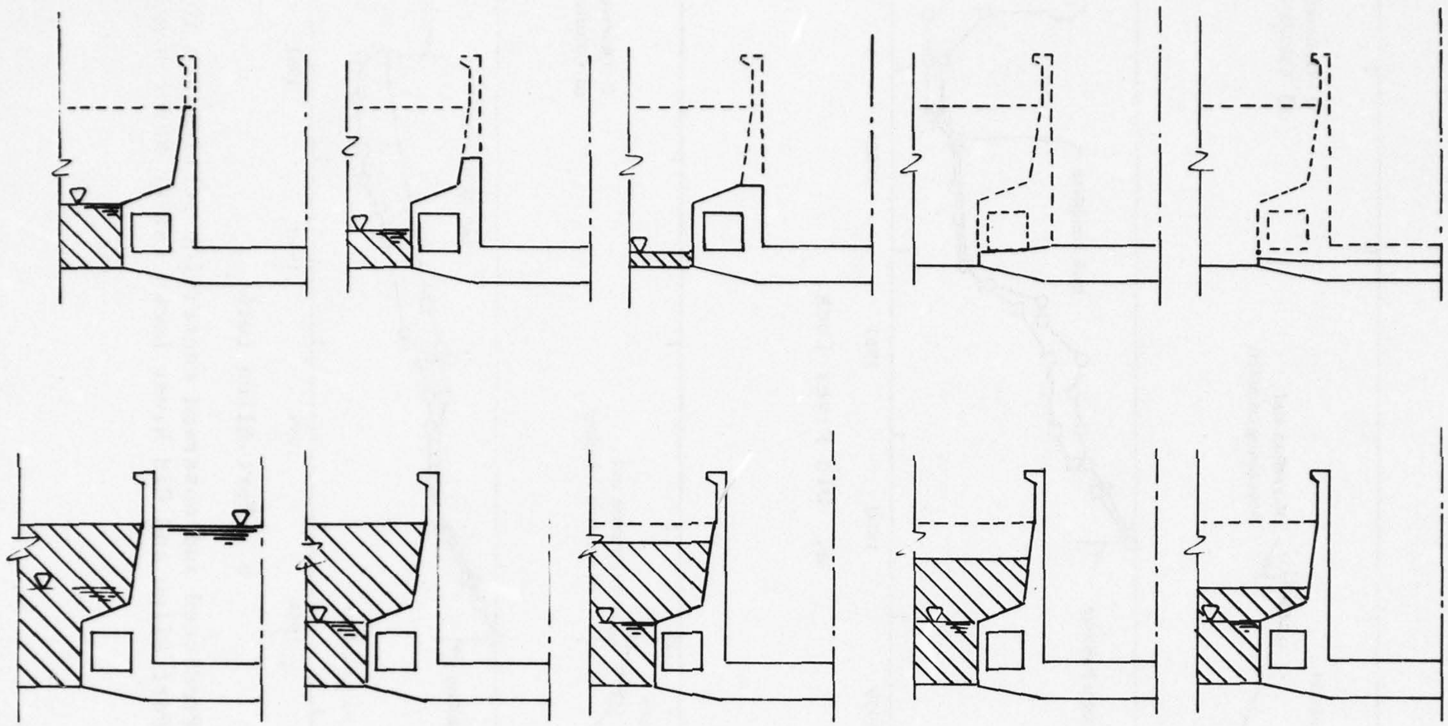
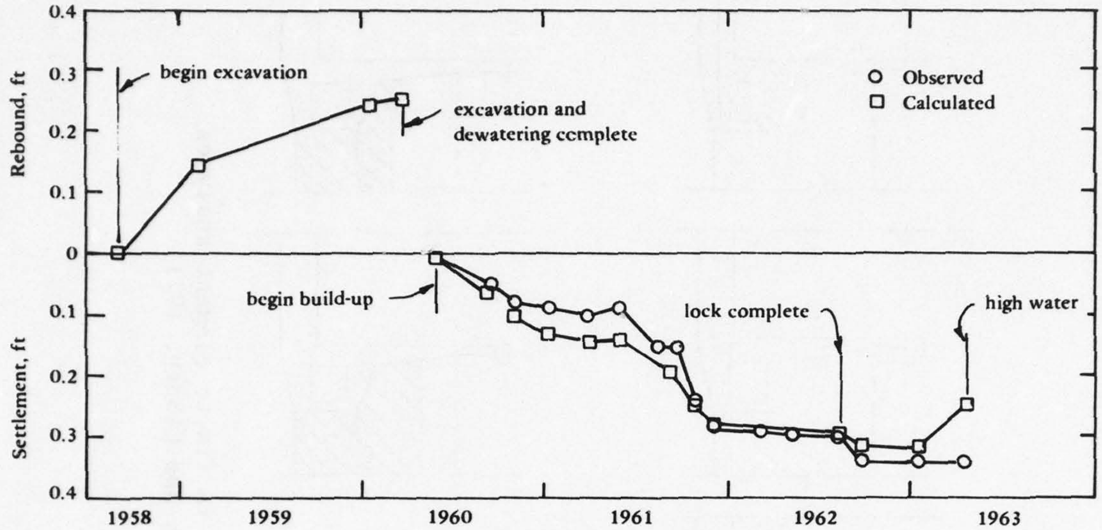
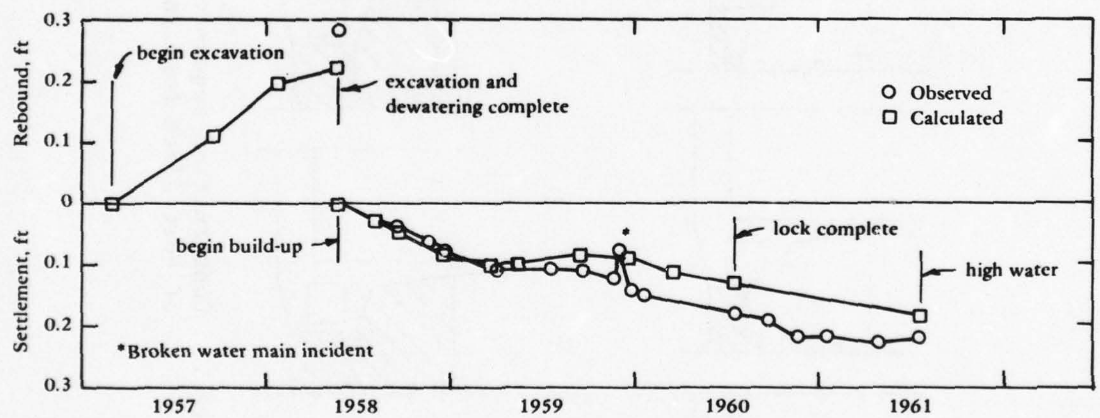


Figure 2. Construction sequence simulated in finite element analyses of Port Allen Lock (after Duncan and Clough, 1971).

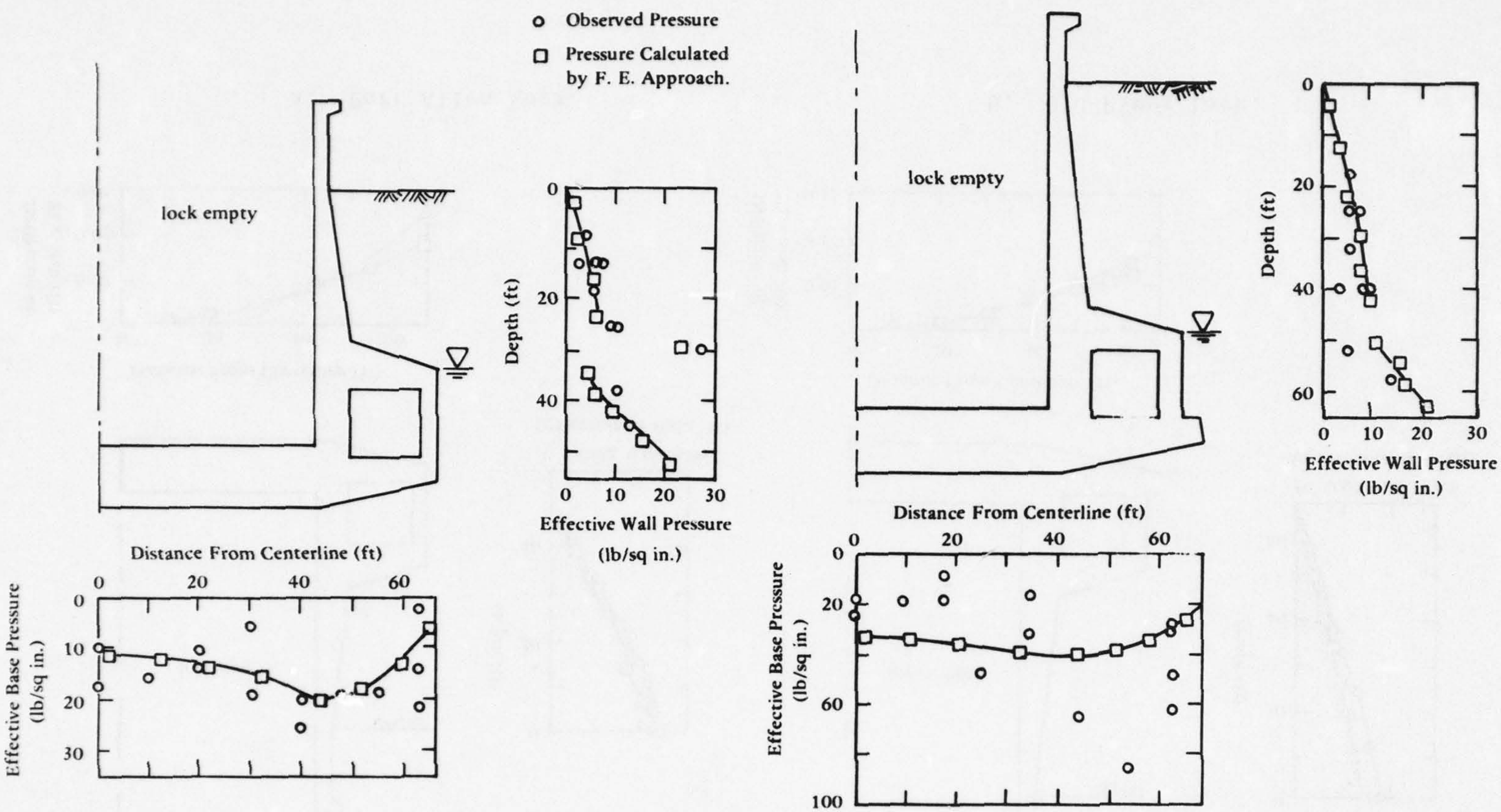


a. Old River Lock.



b. Port Allen Lock.

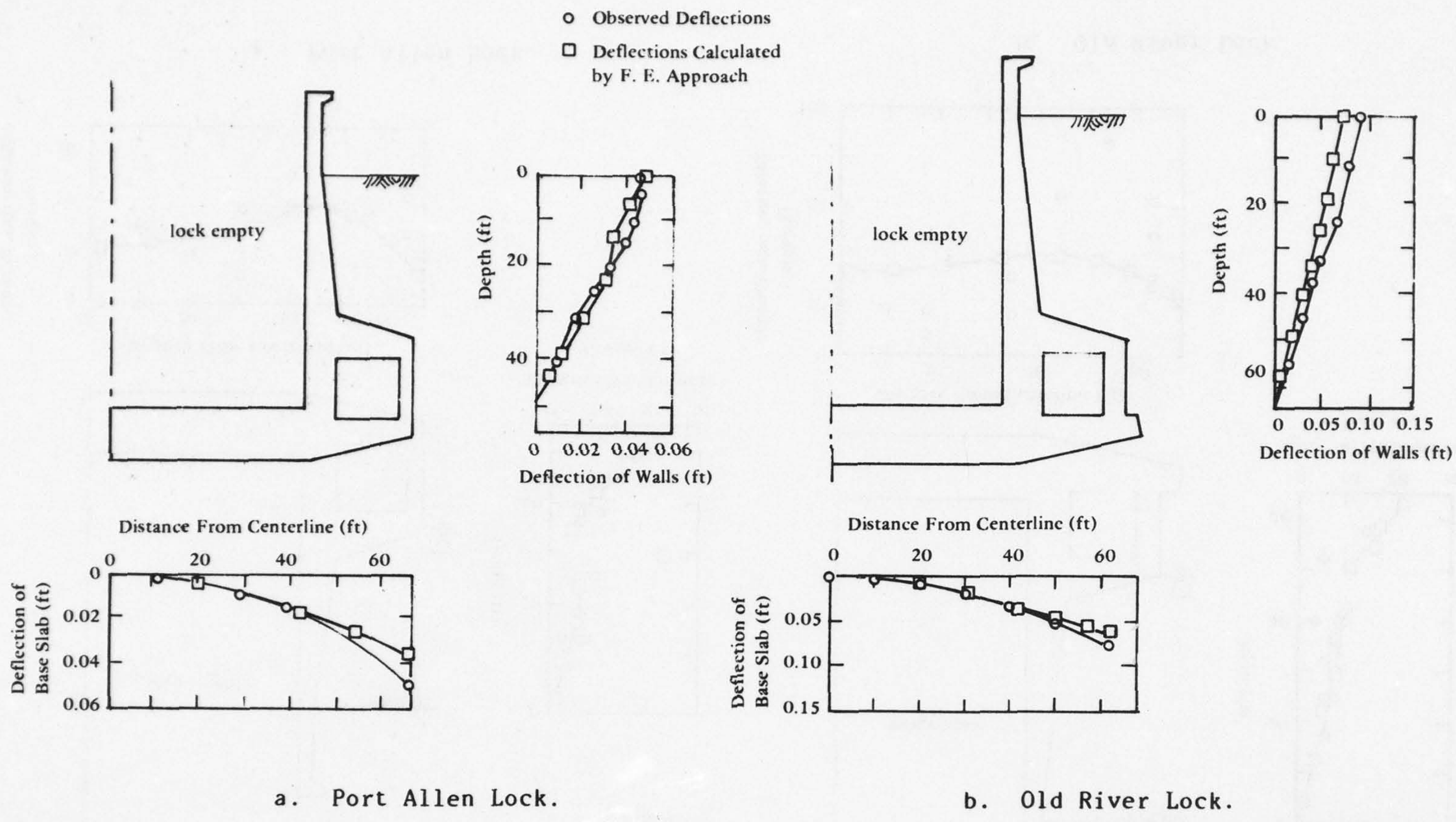
Figure 3. Predicted and observed centerline settlements for Port Allen and Old River Locks (after Reference 27).



a. Port Allen Lock.

b. Old River Lock.

Figure 4. Predicted and observed effective earth pressures for Port Allen and Old River Locks (after Reference 27).



a. Port Allen Lock.

b. Old River Lock.

Figure 5. Predicted and observed structural deflections for Port Allen and Old River Locks (after Reference 27).

Appendix

LIST OF UNITED STATES GRAVING DRYDOCKS

List of United States Graving Drydocks

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
109	Baltimore, Md.	DD No. 3 (GO3)	SUPSHIPS Portsmouth, Va.	Bethlehem Steel Corp. Key Highway Yard	590	24	04/20/78	Repairs
110	Baltimore, Md.	GO5	SUPSHIPS Portsmouth, Va.	Bethlehem Steel Fort McHenry Yard	459	20	06/15/79	Repairs
	Baltimore, Md.	G	SUPSHIPS Portsmouth, Va.	Bethlehem Steel Sparrows Point Yard	1,200.5	28	Not scheduled	Building
	Boston, Mass.	DD No. 1		Navy	415	25.2	Not scheduled	(1833)
	Boston, Mass.	DD No. 2		Navy	750	29.8 (blocks)	Not scheduled	(1905)
	Boston, Mass. (South Boston Annex)	DD No. 3 (commonwealth)		City of Boston	1,175	44.5	Not scheduled	Repairs (1920)
	Boston, Mass. (South Boston Annex)	DD No. 4		City of Boston	693.5	~30.8 (blocks)	Not scheduled	Repairs (1943)
	Boston, Mass.	DD No. 4		Bethlehem Steel	256	16.5	Not scheduled	Repairs
	Boston, Mass.	DD No. 5 (commonwealth)		Navy	518	20.5	Not scheduled	Building (1942)

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
	Bayonne, N.J. (Repairs)	DD No. 7		Army (Bethlehem Steel)	1,092	47	Not scheduled	(1942)
	Brooklyn, N.Y.	DD No. 2	SUPSHIPS Brooklyn	New York City	465	25	Not scheduled	
126	Brooklyn, N.Y.	DD No. 1 G	SUPSHIPS Brooklyn	Commerce Labor Industry Corp. of Kings (Coastal Drydock and Repairs)	349	25	12/31/79	Repairs
127	Brooklyn, N.Y.	DD No. 3 G	SUPSHIPS Brooklyn	Commerce Labor Industry Corp. of Kings (Coastal Drydock and Repairs)	758	36	12/31/79	Building and repairs
128	Brooklyn, N.Y.	DD No. 4 G	SUPSHIPS Brooklyn	Commerce Labor Industry Corp. of Kings (Coastal Drydock and Repairs)	727	36	12/31/79	Repairs
0099	Brooklyn, N.Y.	DD No. 5 (GO5)	SUPSHIPS Brooklyn	Commerce Labor Industry Corp. of Kings	1,093	41	05/21/79	
				(Seatrain Shipbuilding Corp.)	1,093	41		

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
0100	Brooklyn, N.Y.	DD No. 6 (GO6)	SUPSHIPS Brooklyn	Commerce Labor Industry Corp. of Kings (Seatrain Shipbuilding Corp.)			05/21/79	–
0062		Graving Dock No. 1 (G)	SUPSHIPS Brooklyn	Todd Shipyards Corp.	716	30 (blocks)	12/31/79	Repairs
0032	Bremerton, Wash.	DD No. 1 GO1	Puget Sound NSY	Navy	639	35	11/30/78	
0033	Bremerton, Wash.	DD No. 2 GO2	Puget Sound NSY	Navy	867	43	12/01/78	
0034	Bremerton, Wash.	DD No. 3 GO3	Puget Sound NSY	Navy	927	28	01/01/79	
0035	Bremerton, Wash.	DD No. 4 GO4	Puget Sound NSY	Navy	998	50	02/01/79	
0036	Bremerton, Wash.	DD No. 5 GO5	Puget Sound NSY	Navy	1,001	50	03/01/79	
0037	Bremerton, Wash.	DD No. 6 GO6	Puget Sound NSY	Navy	1,152	53	04/01/79	
–	Chicago, Ill.	DD No. 2	–	American Shipbuilding Co.	730	16	Not scheduled	Repairs
–	Duluth, Minn.	DD No. 1	–	Fraser Shipyards Inc.	628	15	Not scheduled	Repairs

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
—	Duluth, Minn.	DD No. 2	—	Fraser Shipyards Inc.	831	19	Not scheduled	Repairs
—	Duluth, Minn.	Dock	—	Fraser Shipyards Inc.	131	—	Not scheduled	Building
0018	Charleston, S.C.	DD No. 1 (GO1)	Charleston NSY	Navy	622	35	10/30/78	Repairs (1962)
0019	Charleston, S.C.	DD No. 2 (GO2)	Charleston NSY	Navy	597	38	10/30/79	Building and repairs (1968)
	Charleston, S.C.	DD No. 3	Charleston NSY		366	11		Primarily building (1943)
	Charleston, S.C.	DD No. 4	Charleston NSY		366	11		Primarily building (1943)
0020	Charleston, S.C.	DD No. 5 (GO5)	Charleston NSY	Navy	751	37	10/30/78	Repairs (1964)
0001	Kittery, Maine	DD No. 1 (GO1)	Portsmouth, N.H. NSY	Navy	435	25	06/30/78	Building and repairs (1905)
0002	Kittery, Maine	DD No. 2 (GO2)	Portsmouth, N.H. NSY	Navy	740	31	06/30/78	Repairs (1943)

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
0003	Kittery, Maine	DD No. 3 (GO3)	Portsmouth, N.H. NSY	Navy	486	37	06/30/78	Building and repairs (1962)
—	Groton, Conn.	DD No. 1	—	General Dynamics Electric Boat Division	520	—	Not scheduled	Repairs
—	Groton, Conn.	DD No. 2	—	General Dynamics Electric Boat Division	690	—	Not scheduled	Repairs
—	Groton, Conn.	Graving Dock No. 3	—	General Dynamics Electric Boat Division	600	—	Not scheduled	Repairs
—	Lorain, Ohio	DD No. 2	—	American Shipbuilding Co.	766	21	Not scheduled	Repairs
—	Lorain, Ohio	DD No. 3	—	American Shipbuilding Co.	925	21	Not scheduled	Repairs
0022	Long Beach, Calif.	DD No. 1 (GO1)	Long Beach NSY	Navy	1,092	44	08/15/79	Repairs (1942)
0023	Long Beach, Calif.	DD No. 2 (GO2)	Long Beach NSY	Navy	688	36	08/15/79	Repairs (1943)
0024	Long Beach, Calif.	DD No. 3 (GO3)	Long Beach NSY	Navy	688	36	08/15/79	Repairs (1943)

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
–	Lyons, N.Y. (Repair)	DD	–	New York State Department of Transportation	330	12	Not scheduled	Repairs
	Hookers Point, Fla.	DD	–	Tampa Ship Repair and Drydock Co.	547	20 (blocks)	–	Repairs
0205	Hunters Point, Calif.	DD No. 2 (GO2)	SUPSHIPS San Francisco	Navy (Triple A Shipyard)	743	29	11/01/79	Building and repairs (1904)
0206	Hunters Point, Calif.	DD No. 3 (GO3)	SUPSHIPS San Francisco	Navy (Triple A Shipyard)	1,005	40	11/01/79	Building and repairs (1916)
0207	Hunters Point, Calif.	DD No. 4 (GO4)	SUPSHIPS San Francisco	Navy (Triple A Shipyard)	1,092	47	09/01/80	Building and repairs (1942)
–	Hunters Point, Calif.	DD No. 5 –	SUPSHIPS San Francisco	Navy (Triple A Shipyard)	420	27	Not scheduled	Building and repairs (1944)
–	Hunters Point, Calif.	DD No. 6 –	SUPSHIPS San Francisco	Navy (Triple A Shipyard)	420	27	Not scheduled	Building and repairs (1944)
–	Hunters Point, Calif.	DD No. 7 –	SUPSHIPS San Francisco	Navy (Triple A Shipyard)	420	27	Not scheduled	Building and repairs (1944)

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
0027	Mare Island, Calif.	DD No. 1 (GO1)	Mare Island NSY	Navy	497	36	03/07/78	Building and repairs (1968)
0028	Mare Island, Calif.	DD No. 2 (GO2)	Mare Island NSY	Navy	741	31	03/07/78	Building and repairs (1910)
0029	Mare Island, Calif.	DD No. 3 (GO3)	Mare Island NSY	Navy	693	36	12/31/78	Building and repairs (1940)
0030	Mare Island, Calif.	DD No. 4 (GO4)	Mare Island NSY	Navy	435	23	12/31/78	Building and repairs (1942)
	Newport News, Va.	DD No. 1		Newport News Shipbuilding and Drydock Co.	654	33	Not scheduled	
	Newport News, Va.	DD No. 2		Newport News Shipbuilding and Drydock Co.	865	31	Not scheduled	
	Newport News, Va.	DD No. 3		Newport News Shipbuilding and Drydock Co.	540	33	Not scheduled	
	Newport News, Va.	DD No. 10		Newport News Shipbuilding and Drydock Co.	960	35	Not scheduled	

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
	Newport News, Va.	DD No. 11		Newport News Shipbuilding and Drydock Co.	1,100	40	Not scheduled	
	Newport News, Va.	DD No. 12		Newport News Shipbuilding and Drydock Co.	1,600	33	Not scheduled	
	Perth-Amboy, N.J.	DD No. 2		Perth-Amboy Drydock Co.	260	14	Not scheduled	
	Perth-Amboy, N.J.	DD No. 3		Perth-Amboy Drydock Co.	400	16	Not scheduled	
	Port Aurther, Tex.	DD No. 1		Gulfport Shipbuilding	282	—	Not scheduled	
	Port Aurther, Tex.	DD No. 2		Gulfport Shipbuilding	112	—	Not scheduled	
	Port Aurther, Tex.	DD No. 3		Gulfport Shipbuilding	160		Not scheduled	
	Port Aurther, Tex.	DD No. 4		Gulfport Shipbuilding	282		Not scheduled	
	Orange, Tex.	DD No. 1 DD No. 2 DD No. 4		Levingston Shipbuilding Co.	349 220 388		Not scheduled Not scheduled Not scheduled	

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
0038	Pearl Harbor, Hawaii	DD No. 1 (GO1)	Pearl Harbor NSY	Navy	1,002	35	08/25/78	Repairs (1919)
0039	Pearl Harbor, Hawaii	DD No. 2 (GO2)	Pearl Harbor NSY	Navy	1,000	47	08/25/78	Repairs (1941)
0040	Pearl Harbor, Hawaii	DD No. 3 (GO3)	Pearl Harbor NSY	Navy	497	23	08/25/78	Repairs (1942)
0041	Pearl Harbor, Hawaii	DD No. 4 (GO4)	Pearl Harbor NSY	Navy	1,089	49	08/25/78	Repairs (1943)
	Pascagoula, Miss.	DD		Ingalls Shipbuilding	485	34	–	Repairs
0004	Philadelphia, Pa.	DD No. 1 (GO1)	Philadelphia NSY	Navy	442	24	12/31/78	Repairs (1956)
0005	Philadelphia, Pa.	DD No. 2 (GO2)	Philadelphia NSY	Navy	745	30	12/31/78	Repairs (1908)
0006	Philadelphia, Pa.	DD No. 3 (GO3)	Philadelphia NSY	Navy	1,011	40	12/31/78	Repairs (1921)
0007	Philadelphia, Pa.	DD No. 4 (GO4)	Philadelphia NSY	Navy	1,092	37	04/15/78	Building and repairs (1941)
0008	Philadelphia, Pa.	DD No. 5 (GO5)	Philadelphia NSY	Navy	1,092	40	04/15/78	Building and repairs (1942)

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
0011	Portsmouth, Va.	DD No. 1 (GO1)	Norfolk, Va. NSY	Navy	324	25	10/30/78	Repairs (1833)
0012	Portsmouth, Va.	DD No. 2 (GO2)	Norfolk, Va. NSY	Navy	498	37	03/31/78	Repairs (1966)
0013	Portsmouth, Va.	DD No. 3 (GO3)	Norfolk, Va. NSY	Navy	727	34	03/31/78	Repairs (1967)
0014	Portsmouth, Va.	DD No. 4 (GO4)		Navy	1,011	44	06/30/79	Repairs (1919)
0015	Portsmouth, Va.	DD No. 6 (GO6)		Navy	465	20	06/30/79	Repairs (1919)
0016	Portsmouth, Va.	DD No. 7 (GO7)		Navy	465	20	06/30/79	Repairs (1919)
0017	Portsmouth, Va.	DD No. 8 (GO8)		Navy	1,092	48	10/30/78	Repairs (1942)
–	Richmond, Calif.	DD No. 1	SUPSHIPS San Francisco	City of Richmond (Willamette Iron and Steel Co.)	600	20	Not scheduled	Building
0194	Richmond, Calif.	DD No. 2 (BO2)	SUPSHIPS San Francisco	City of Richmond (Willamette Iron and Steel Co.)	748	32	12/31/77	Building
–	Richmond, Calif.	DD No. 3	SUPSHIPS San Francisco	City of Richmond (Willamette Iron and Steel Co.)	600	32	Not scheduled	Building

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
--	Richmond, Calif.	DD No. 4	SUPSHIPS San Francisco	City of Richmond (Willamette Iron and Steel Co.)	600	20	Not scheduled	Building
0195	Richmond, Calif.	DD No. 5 (BO5)	SUPSHIPS San Francisco	City of Richmond (Willamette Iron and Steel Co.)	600	20	12/31/77	Building
0202	San Diego, Calif.	DD No. 1 (GO1)	SUPSHIPS San Diego	Campbell Industries	160	12	10/23/77	Repairs
	San Diego, Calif.	DD No. 2	SUPSHIPS San Diego	Campbell Industries	200	15	Not scheduled	Repairs
	San Diego, Calif.	DD No. 3	SUPSHIPS San Diego	Campbell Industries	200	16	Not scheduled	Repairs
	San Diego, Calif.	DD No. 4	SUPSHIPS San Diego	Navy (Campbell Industries)	389	18	Not scheduled	Repairs
	San Diego, Calif.	DD No. 5	SUPSHIPS San Diego	Campbell Industries	389	19	Not scheduled	Repairs
	San Diego, Calif.	DD	SUPSHIPS San Diego	Navy	694	37	Not scheduled	(1942) Inactivated 1965
	San Diego, Calif.	DD	SUPSHIPS San Diego	National Steel and Shipbuilding Co.	694	32	Not scheduled	Repairs

continued

NAVSEA Serial No.	Site Location	Dock Identification	Cognizant Activity	Owner or Operator	Overall Length (ft)	Depth MHW Over Sill (ft)	Certification Schedule	Comments[a]
–	Savannah, Ga.	Graving Dock	–	Savannah Machine and Shipyard Co.	540	18	Not scheduled	Repairs
–	Sturgeon Bay, Wis.	DD	–	Bay Shipbuilding Corp.	27[illegible]	16	Not scheduled	
–	Sturgeon Bay, Wis.	DD	–	Bay Shipbuilding Corp.	1,150	17	Not scheduled	(1976)
–	Toledo, Ohio	DD No. 1	–	American Shipbuilding Co.	540	13	Not scheduled	Repairs
	Toledo, Ohio	DD No. 2	–	American Shipbuilding Co.	660	14	Not scheduled	Repairs
–	Roosevelt Roads, P.R.	DD No. 1	–	Navy	1,088	48	Not scheduled	Inactivated (1944)
–	San Juan, P.R.	DD No. 1	–	Navy	654	29	Not scheduled	Inactivated (1942)

[a]Date of completion or modernization is in parentheses.

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