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EARTH PRESSURES AND RETAINING STRUCTURES

OHIO RIVER VALLEY SOILS SEMINAR

XI

Clarksville

Indiana

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EARTH PRESSURES AND RETAINING STRUCTURES

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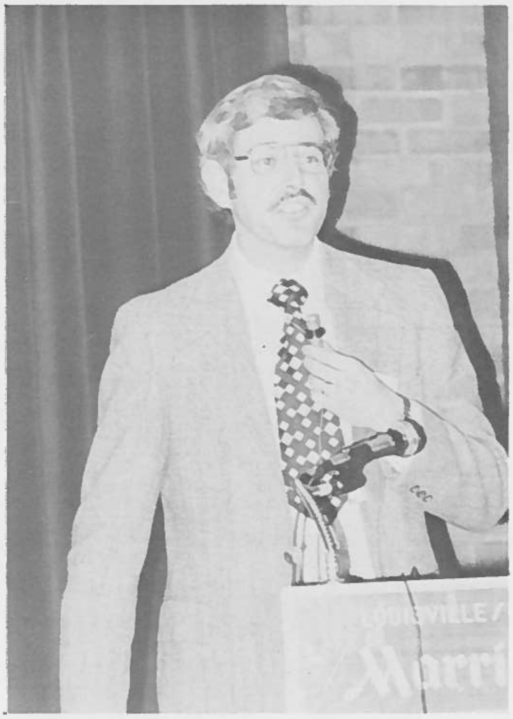

Dr. Thomas D. O'Rourke addresses the morning session on braced excavations

Dr. Yves Lacroix speaks on the design and construction of anchored bulkheads

Dr. Charles O. Riggs relates experiences with tie-back membrane walls in Venezuela

Dr. D. Joseph Hagerty presides at the morning session

Photo

Mr. James J. Kerr describes underpinning techniques to afternoon session

Drs. Thomas D. O'Rourke, Edward B. Kinner, Robert C. Deen, and Ernest T. Selig (left to right) at ORVSS-XI

Luncheon discussion amongst attendees at ORVSS-XI

Dr. Charles R. Ullrich with evening speaker, Dr. Delon Hampton

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GROUND MOVEMENTS CAUSED BY BRACED EXCAVATIONS

By

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Abstract. This paper examines the ground movements caused by braced excavations with emphasis on the relationship between ground movements and various aspects of the construction process. Sources of lost ground are reviewed and deformation patterns at the excavation wall are related to displacement patterns at the ground surface adjoining the excavation. Case histories of braced cuts in both sands and soft clays are summarized and recommendations for ground movement control are made on the basis of observational data.

Introduction

The design of braced excavations is performed on the basis of two fundamental requirements: 1) the selection of excavation and support measures necessary for stability of the cut, and 2) the control of ground movements affecting adjacent property. Stability generally means preventing failure, that is, designing the braced cut to remain open without danger to construction personnel or very large movements that jeopardize surrounding structures. Design measures for stability include the selection of earth pressures, determination of the potential for base soil failure, and the layout of a dewatering system to prevent piping and blow-out along the walls and bottom of the cut. The control of ground movements generally is based on the types of buildings and public facilities near the excavation. Foremost among the measures necessary for control is the prediction of displacements at various distances from the cut. In addition, control measures may include the protection of adjacent structures and the use of construction methods to restrict ground movements to levels consistent with the acceptable performance of surrounding facilities.

With regard to stability, apparent earth pressure envelopes are recommended for a variety of soil types[1,2,3], and methods are available for estimating the likelihood of plastic deformation and base failure for excavations in clay[1,4]. In contrast, the patterns of settlement associated with braced cuts are summarized only in very general terms, and horizontal displacements are treated mostly on a case history basis. The wide range of performance resulting from different soil conditions and construction methods often complicates the decision-making process regarding both the potential for damage and protection of nearby structures.

This paper examines the ground movements caused by braced excavations with emphasis on the relationship between ground movements and various aspects of the construction process. Sources of lost ground are reviewed and deformation patterns at the excavation wall are related to displacement patterns at the ground surface adjoining the excavation. Case histories of braced cuts in both sands and soft clays are summarized and recommendations for ground movement control are made on the basis of observational data.

Site Preparation

Braced cuts commonly are visualized as a stepwise process of excavation and support. Following each increment of excavation, inward deflection of the walls occur. The inward wall movement represents the volume of displaced soil, which is transmitted through the ground adjoining the cut as settlement and horizontal offsets. This process forms the conceptual basis for much of the analysis and finite element modeling of braced cut performance and has been helpful for understanding the ground response to the actual excavation and bracing procedures. Nevertheless, deep excavations often are performed in combination with other activities that cause movement, particularly during the preparatory stages for construction.

Site preparation may include 1) relocation and underpinning of utilities, 2) dewatering of aquifers above and below the base of excavation, 3) construction of the excavation wall, and 4) the installation of deep foundations. In some cases, the movements associated with site preparation will exceed those occurring as a result of the excavation and support process. The relocation of utilities, for example, may have a locally severe impact on adjacent property, especially when trenching is carried close to other pipelines and communication conduits. Alternatively, dewatering may consolidate the soil over an area that exceeds substantially the area affected by excavation-induced movements. Wall construction may require predrilling for soldier piles, the use of vibratory hammers to install sheet piles, or the excavation of slurry panels for a concrete diaphragm wall. Each of these can cause permanent movements, the magnitude and distribution of which will vary according to the soil conditions and details of the construction procedures.

Ground movements associated with site preparation are shown clearly in the records of deep basement construction. As an example, Figure 1 relates typical settlement and horizontal movements with the

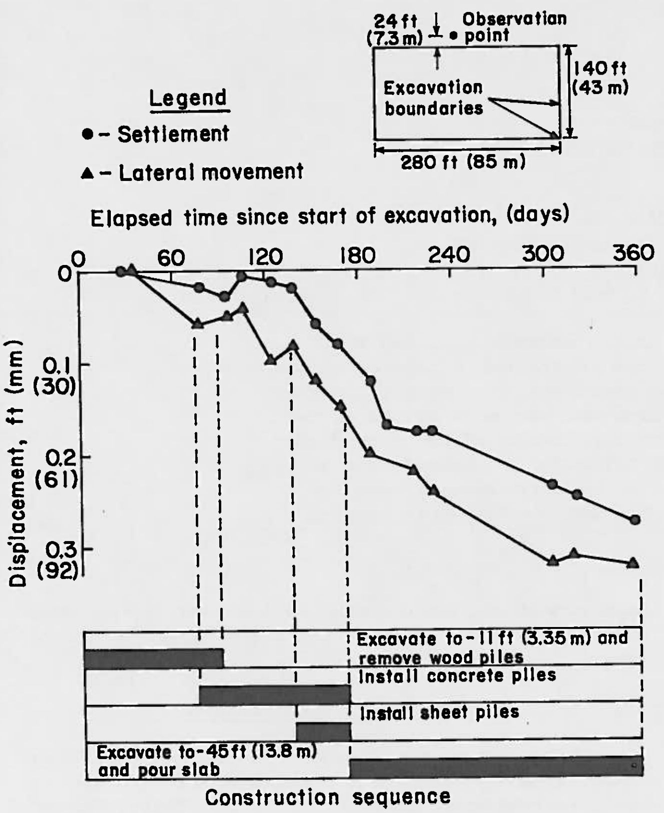
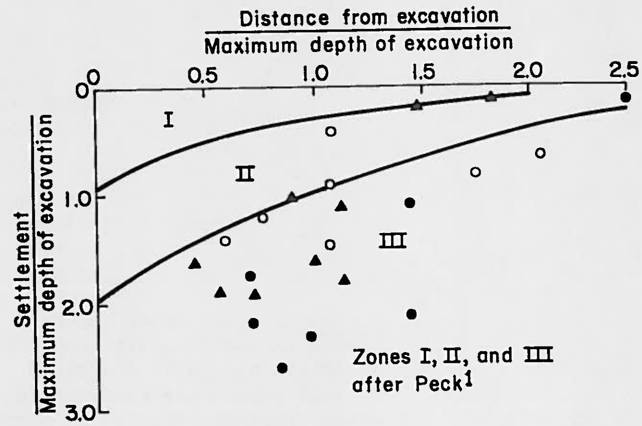


Figure 1. Ground Displacement Record for a 45-ft (14 m)-deep Cut in Soft to Medium Clay.

various construction activities that were performed for a 45-ft (14 m)-deep braced cut in San Francisco. The cut was excavated through approximately 22 ft (6.5 m) of granular fill and sands and 23 ft (7 m) of soft to medium silts and clays. The bottom of the cut was underlain by roughly 20 ft (6 m) of medium clay followed by a deposit of interbedded dense sand and stiff clay.

The ground movements were strongly influenced by construction activities related to site preparation for deep excavation and pile driving. Over 800 timber piles were removed from the site and a similar number of prestressed concrete piles were driven. Sheet piles were installed along the perimeter of the cut with a vibratory hammer. Between Day 90 and 170, approximately 30 percent of the total ground movements occurred even though no excavation was performed during this time interval. In some cases, there was a clear relationship between movement and construction activity. For example, when timber piles were removed near the property line, horizontal movements were induced that cracked the street surface for a distance of 15 to 25 ft (4.5 to 7.6 m) from the excavation's edge.

In a similar manner, the construction of drilled shafts in Chicago has led to large settlements. Figure 2 summarizes the settlements adjacent to three braced cuts where large movements were caused, principally as the result of ground loss during drilled shaft, or caisson, construction. The settlements and distances are plotted in dimensionless form as fractions of the maximum excavation depth. Zones of settlement, delineated by Peck[1] for various soil



Case	Symbol	Max. depth ft (m)	Support	Special problems
1	▲	45 (14)	Slurry wall; 3 levels of rakers	Pumped fines from base of caissons
2	●	28 (8.5)	Soldier piles & lagging; 1 level of rakers	Soil squeeze during caisson construction
3	○	26 (8)	Slurry wall; 1 level of struts	Soil squeeze during caisson construction

Figure 2. Settlements Adjacent to Braced Cuts with Caisson Construction Difficulties.

types and excavation conditions, are shown for comparison.

Chicago practice for caisson construction has been discussed in detail by Baker and Khan[5], and only a brief description is given here. Essentially, the caissons were advanced as a series of oversized shafts, which were telescoped into successively smaller diameters until shafts slightly larger than the caissons were obtained at a depth roughly 40 ft (12 m) below the construction surface. The caissons were extended to limestone bedrock, approximately 110 ft (34 m) below street level. Steel linings were used to support the shafts. Some linings were installed as permanent parts of the caissons, whereas others were removed after pouring concrete.

Much of the settlement associated with Case 1 resulted from dewatering difficulties in a deposit of sand, silt, and gravel directly overlying bedrock. Pumping was required to prepare the caisson bottoms for concrete and, in the process, sand and silt were pumped out with the water. A large portion of the settlements for Cases 2 and 3 was caused by clay squeezing into the shafts during excavation and into annular voids in the shafts after steel linings had been installed. In each case, the movements associated with caisson construction were between 50 and 70 percent of the total settlements shown in Figure 2.

General Patterns of Ground Movement

Braced excavations generally are performed in three prominent stages. During each stage, excavation and support methods contribute to the ground displacements in a characteristic way. These stages include 1) initial excavation before bracing, 2) excavation to subgrade after the upper braces are installed, and 3) removal of the braces.

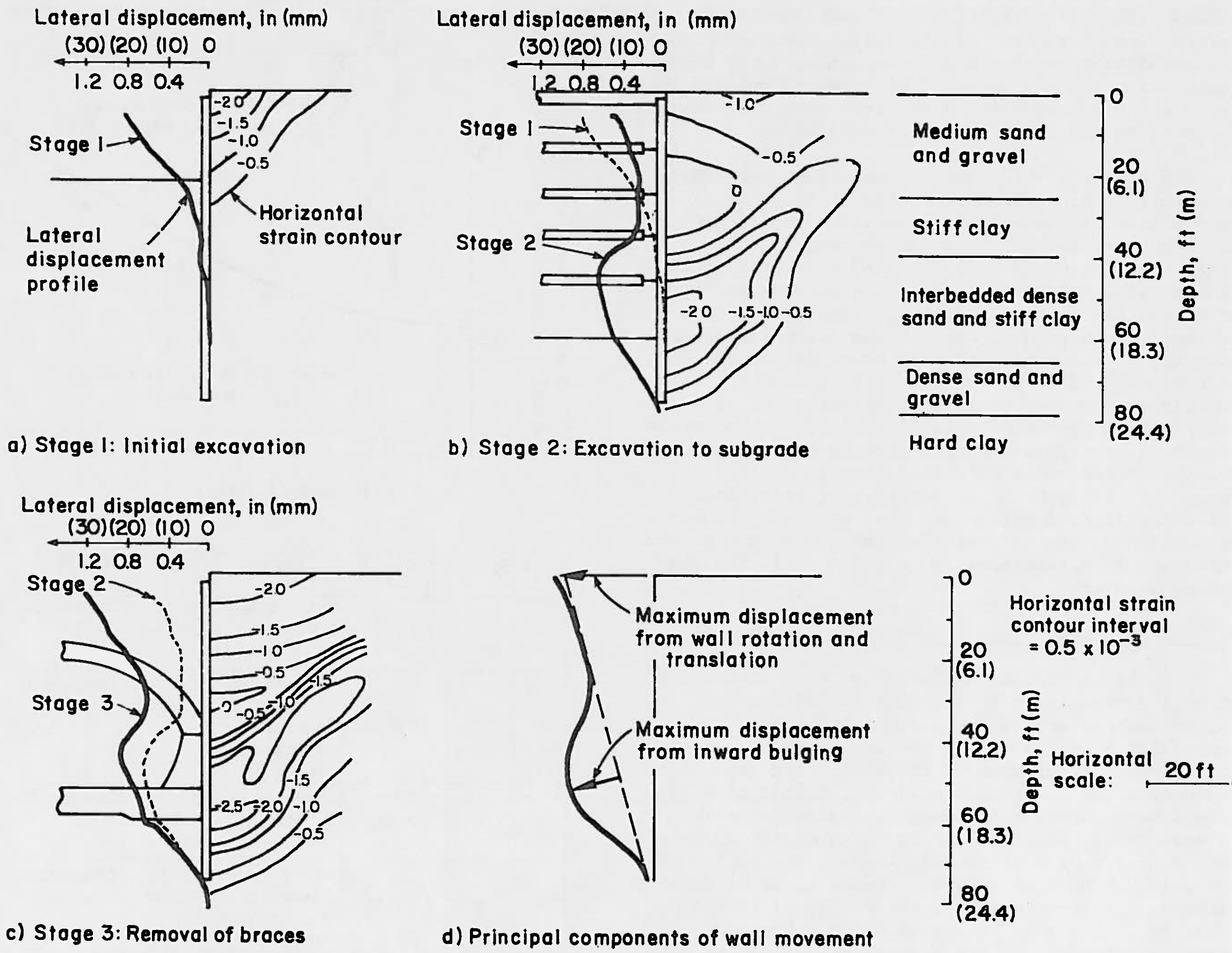


Figure 3. Horizontal Strains Associated with Various Stages of Braced Cut Construction.

Figure 3 traces the development of lateral soil movement as a function of the construction history for a cut-and-cover excavation in Washington, D.C. The excavation was 60 ft (18 m) deep and was supported by soldier pile and lagging walls with five levels of cross-lot struts. The soldier piles (24 W 130) were 75 ft (23 m) long and were located on 7.5 ft (2.3 m) centers. Field observations of this excavation and details of the instrumentation have been reported by O'Rourke[6].

The figure shows the horizontal wall movements and cumulative horizontal strains in the retained soil for each of the construction stages. The horizontal strains have been estimated from inclinometer measurements by dividing the differential lateral displacement between two points at a given elevation by the distance separating them and plotting the strain at the mid-point between the two measurements. The contour interval for horizontal strain is 0.5×10^{-3}, which is equivalent to the tensile strain at which cracking initiates in structural masonry[7]. Strain magnitudes prefixed with a minus sign indicate tension. The construction sequence and lateral movements are discussed under the following headings.

Initial Excavation Before Bracing

The excavation was deepened 20 to 25 ft (6 to 7.5 m) before braces were installed. Hence, deformation of the wall occurred primarily as a cantilever-type movement. The horizontal strains reflect this mode of deformation in a triangular pattern of contours that decrease in magnitude with depth and distance from the wall.

Excavation to Subgrade After Upper Braces Installed

As braces were installed, the upper portion of the wall was restrained from further lateral movement. In fact, preloading the upper level struts decreased the horizontal movement at the top of the cut, resulting in a slight recompression of the soil. In the deeper portion of the cut, inward bulging of the wall caused tensile strains, the contours for which loop upward at an angle of approximately 45 degrees from the vertical.

Removal of the Braces

As the bottom braces were removed to build the underground structure, further inward bulging of the wall occurred. When the upper braces were removed, the wall was supported in its lower portion by the subway structure, resulting in a cantilever-type deformation of the top of the wall. Consequently, the cumulative strains are a composite of the horizontal distortion associated with inward bulging at depth and the distortion associated with cantilever movement at the upper levels of excavation.

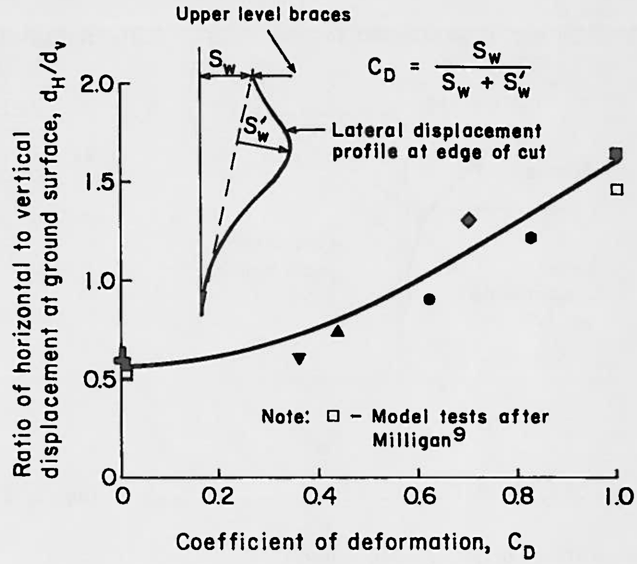
In summary, the lateral movements generated by open cutting are related directly to the mode of deformation at the excavation wall which, in turn, is related to the construction procedure. The final movement pattern is composed essentially of two zones of horizontal strain. A deep-seated zone of horizontal strain develops in response to inward bulging of the excavation wall and forms the lower boundary of the mobilized soil mass. An upper zone of horizontal strain develops in response to either cantilever movement or lateral translation. The two principal modes of wall deformation are shown in Figure 3d. Because the soil strains are closely related to the modes of wall deformation, it is reasonable to look for a quantitative relationship between wall deformation and the relative proportion of horizontal to vertical displacement at the ground surface. This relationship is explored in the following section.

Wall Deformation and Ground Surface Movement

To illustrate the influence of wall distortion on soil movements, a coefficient of deformation, C_D, is defined in Figure 4. The numerator of the expression is a measure of the cantilever movement and lateral translation of the wall. The denominator represents the combined cantilever, translation and deep inward movement components of displacement. In some cases, particularly for excavations in very deep deposits of soft to medium clay, the bottom of the wall may deflect inward[8]. Under these circumstances, inward movement at the bottom of the wall would be included in the denominator. The coefficient is intended to show general trends in the data pertaining to the immediate displacements caused by open cutting. Consequently, field observations analysed in terms of the coefficient were chosen for cases where there was little influence on movements from either site preparation or consolidation.

Figure 4 shows the ratio of horizontal displacement to settlement at the ground surface adjoining the cut as a function of the coefficient of deformation. Inclinometer measurements of wall deformation and optical surveys of settlement and lateral displacement were used to develop the plot. The field data pertain to maximum displacements, which were measured after the excavations had been taken to subgrade and a significant portion of the braces removed. The table in the figure summarizes information about the braced cuts. Data were derived from a total of seven braced cuts, ranging in depth from 25 to 60 ft (8 to 18 m). The field data are related to surface movements taken within distances of 0.35 to 1.0 times the maximum excavation depths from the cuts.

The data for each excavation have been screened in accordance with two guidelines: 1) each ratio of lateral to vertical displacement derives from the combined measurement of lateral movement and settlement at the same observation point; 2) measurements equal to or less than 1/4 in (6 mm) have been neglected in order to minimize the influence of survey error on the computed ratio. Furthermore, the computed ratios were analysed statistically to



Case	Symbol	Max. depth ft (m)	Support	Soil	Location
1	●	60 (18)	Soldier piles & lagging, 5 strut levels	Sand and stiff clay	Washington, D.C.
2	▲	44 (13)	Soldier piles & lagging, 3 levels of struts and rakers	Soft to medium clay	Chicago, Ill.
3	▼	27 (8)	Sheet pile, 3 raker levels	Soft to medium clay	Chicago, Ill.
4	■	44 (13)	Slurry wall, 2 levels of tiebacks and rakers	Soft to medium clay	Chicago, Ill.
5	◆	30 (9)	Soldier piles & lagging, 2 raker levels	Soft to medium clay	Chicago, Ill.
6	⬢	25 (8)	Sheet pile 1 raker level	Soft to medium clay	Chicago, Ill.
7	♣	45 (14)	Sheet pile, 3 levels of struts and rakers	Soft to medium clay	San Francisco, Calif.

Figure 4. Ratio of Horizontal to Vertical Ground Movement as a Function of the Coefficient of Deformation.

determine mean values and coefficients of variation. The mean values are plotted. The coefficients of variation, which range from 0.15 to 0.47, indicate that the data for each cut are reasonabley well clustered.

The field data show limits for the ratio of horizontal to vertical movement equal to approximately 0.6 and 1.6 for wall deformation caused solely by inward bulging and solely by cantilever movement, respectively. These bounds compare favorably with the limits determined from model tests, performed by Milligan[9], which also are plotted in Figure 4. In the model tests, soil strains and displacements in medium dense sand were measured in response to wall movements caused by incremental excavation. The combined field and test data show that, as cantilever-type movements are allowed to dominate during construction, the relative magnitudes of horizontal ground movement increase significantly.

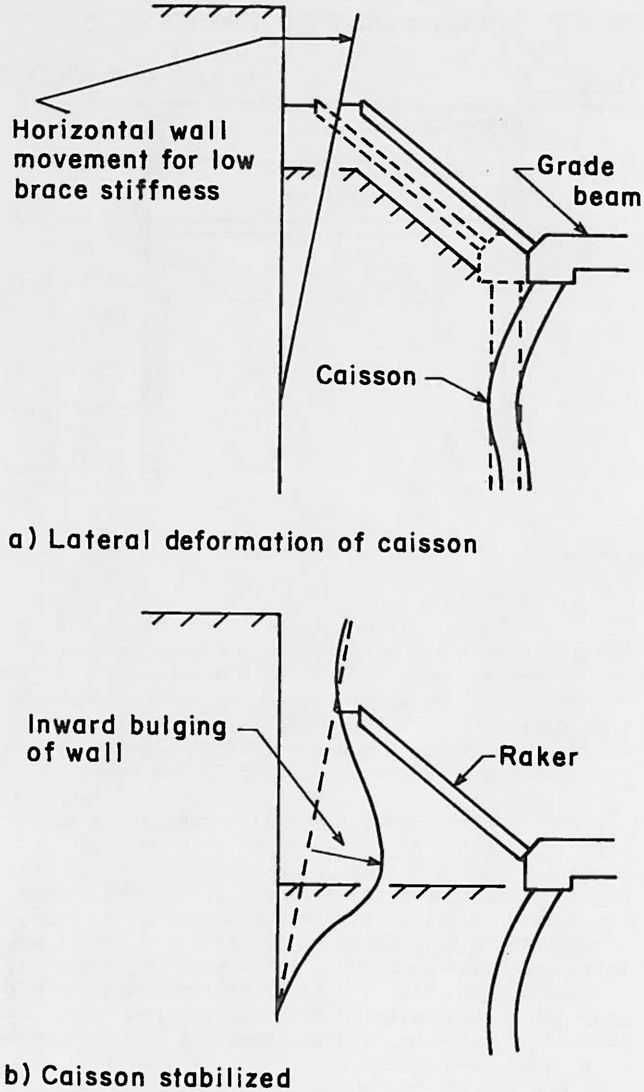


Figure 6. Wall Deflections Associated with Raker and Caisson Bracing.

and stiff clay and 15 ft (4.5 m) of hard, fissured clay. The water table was located at approximately 35 ft (10.5 m) below the ground surface. The wall was 82 ft (25 m) deep and 33 in (0.85 m) wide and was reinforced with steel piles (30 W 124) on 7 ft (2 m) centers. All braces were pre-stressed to approximately 75 percent of their design loads.

The figure shows lateral wall displacements corresponding to three successive levels of excavation beneath the second level struts. All movements and excavation depths are referenced to the time when the second level struts were installed and preloaded (Day 0). As is common practice during cut-and-cover construction, berms were left against the excavation walls as the cut was deepened. Inward displacements developed in response to excavation beneath the lowest braces. As the central portion of the cut was deepened from 35 to 55 ft (10.5 to 17 m), the inward movement increased by a maximum 0.7 in (18 mm) and the volume of displacement nearly doubled.

The movements illustrated in Figure 7 emphasize the importance of excavation depth beneath the lowest braces for controlling ground loss during open cutting. The section stiffness of the concrete diaphragm wall, which is the product of the wall's elastic modulus and moment of inertia, was approximately ten times that of soldier pile and lagging walls used elsewhere during construction of the Washington, D.C. Metro. Nevertheless, the inward movements of the diaphragm wall are comparable to those reported for the other cuts[6,14]. The deflections of excavation walls under distributed earth loads depend not only on the section stiffness, but on approximately the fourth power of the unsupported

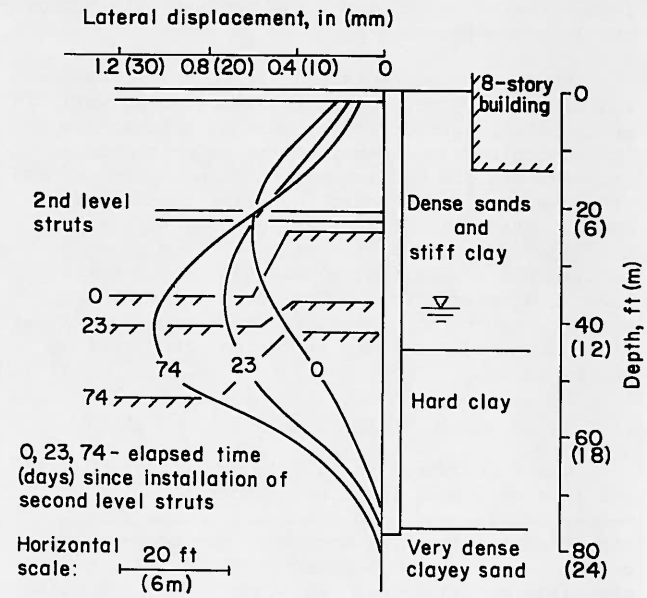


Figure 7. Observed Lateral Displacements of a Concrete Diaphragm Wall.

depth. Hence, an excavation carried 27 ft (8 m) as opposed to 15 ft (4.5 m) beneath the lowest braces will offset the benefit derived from a tenfold increase in wall section stiffness.

When concrete diaphragm walls are used as an alternative to underpinning adjacent structures and dewatering, they must support foundation and hydrostatic pressures that exceed the earth pressures imposed on traditional support systems. Under these conditions, it is unlikely that the deflection of a concrete diaphragm wall will be substantially less than that of a soldier pile and lagging wall subject only to earth pressure. For good control of ground movements, experience in Washington, D.C. suggests that excavation depths be limited to approximately 18 ft (5.5 m) beneath the lowest braces for deep cuts in medium to dense sands and interbedded sands and stiff clay. This guideline should apply for excavations supported by concrete diaphragm walls as well as for cuts retained by soldier pile and lagging systems.

The Use of Berms

A berm is a mass of soil that is left in place against the walls of an excavation. Because braced cuts must be performed in stages, berms lend themselves to the excavation and support process in ways so straightforward that they often become an integral part of the construction technique. When deep basements are built in deposits of soft to medium clay, berms frequently are used to restrain the

Conversely, if the wall is firmly braced at an early stage of excavation, inward deflection of the wall will lead to horizontal movements that are significantly less than settlements.

The data summarized in Figure 4 can be used to estimate horizontal movements, either directly from settlement surveys or, on a planning basis, prior to construction. Most excavation and bracing schemes will generate both cantilever displacement and deep inward movement of the wall. Consequently, a value of one will provide a reasonable, preliminary estimate of the ratio of horizontal movement to settlement. Settlements can be estimated on the basis of local experience or from field measurements summarized in the literature.

Because settlements and horizontal movements are linked directly to the movement of the wall, it is important to understand how wall deformation is influenced by the excavation and support process. Many factors can be distinguished among the methods of excavation and support that affect ground movements. Three, in particular, deserve attention. They include brace stiffness, the depth of excavation beneath the lowest braces, and the use of berms. Each is discussed in the following sections and examples are taken from field observations to show ground movement response to typical construction procedures.

Brace Stiffness

Brace stiffness has a critical effect on wall deformation. Studies of braced cut behavior by Palmer and Kenney[10] and Jaworski[11], show that significant horizontal movements can occur when braces lack tight connections with the wall or compressible materials are used to shim or pack voids between members of the support system. Commonly, braces are pre-stressed to a percentage of their design load to induce rigid contacts.

O'Rourke and Cording[12] have discussed preloading practices during construction of the Washington, D.C. Metro. It is useful to review these practices in order to emphasize some important characteristics of the preloading operation. Preloading was performed by inserting a hydraulic jack on each side of an individual cross-lot strut between the wall and a special abutment plate welded to the strut. Hydraulic pressure was applied until the strut was stressed to approximately half its design load after which the separation between the strut and wall was shimmed with 1/4-in (6 mm)-thick steel plates. Figure 5 shows soldier pile displacements (shaded area) caused by preloading a level of cross-lot struts. The measurements, which were taken with inclinometers attached to the soldier piles, are accurate to about 0.01 in (0.25 mm) at the level of preloading. The braced cut is the same excavation for which ground movement patterns are summarized in Figure 3.

The outward movement of the soldier piles shows that preloading was effective in closing separations within the bracing system. Movement at the level of the preloaded struts was a maximum 0.1 in. (2.5 mm) at each side of the excavation, and was distributed primarily within 10 ft (3 m) of the elevation of the jacking and wedging operation. As is characteristic of soil/structure interaction under these conditions, preloading had only a localized effect on soil movement. Consequently, preloads increased above the magnitude needed to close separations in the

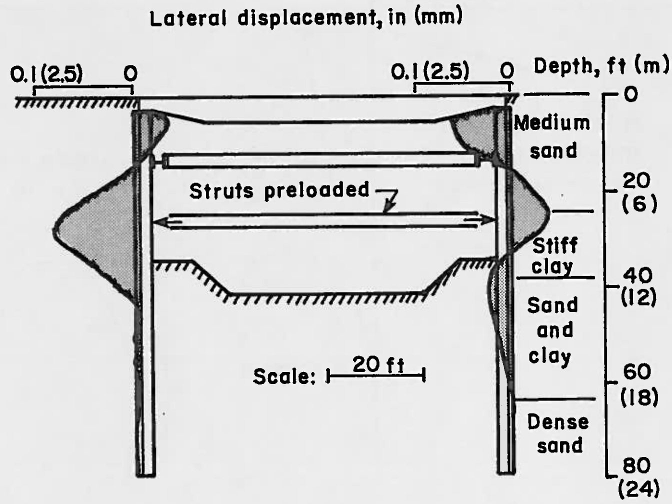


Figure 5. Wall Displacements from Preloading Struts.

bracing system are not likely to regain previous movements nor to prevent further displacement as the excavation is deepened. On the contrary, practices that call for very high preloads are likely to cause local concentrations of earth pressure that may exceed the pressures assumed during design.

After preloading, the wall generally is considered to be fixed at the point of brace contact. However, there are some instances where substantial movements can occur at the brace level even though preloading is used. Figure 6 illustrates a relatively common bracing procedure in which rakers are installed between the wall and a portion of the completed foundation. A raker transmitting loads to only one or two caissons may lack the restraint to prevent significant wall movement. Elastic analyses, such as those outlined by Davisson and Gill[13], show that the lateral movements of a simple raker and caisson bracing system can be as high as 3 in (76 mm) for braced cut loads typical of construction conditions in Chicago. Displacements of this nature lead to cantilever deformation of the wall.

Figure 6a shows an idealized view of wall deformation associated with horizontal loading of a caisson. As grade beams are joined to several caissons and foundation slabs are poured, the restraint transmitted through the raker becomes increasingly more efficient. Figure 6b shows additional wall movement after caisson deflections have ceased and the excavation has been completed. Although the volumes of incremental displacement shown in Figure 6a and 6b are approximately equal, the influence of wall movement on the pattern of soil displacement is substantially different. In Figure 6a the cantilever deformation of the wall may induce horizontal ground movements that are more than twice as large as those generated by the inward bulging shown in Figure 6b.

Excavation Depth Beneath the Lowest Braces

Figure 7 shows the lateral wall movements for a concrete diaphragm wall that was used during construction of part of the Washington, D.C. Metro. The 70-ft (21 m)-wide excavation was extended through approximately 45 ft (14 m) of dense sands excavation walls as the basement structure is assembled at the center of the cut. In a similar manner, berms are used during cut-and-cover excavation. As the cut is extended beneath the lowest braces, the excavation may be further deepened at the center to remove additional soil and provide a path for construction vehicles. This practice is referred to as "slotting out" the cut. Under these conditions, the central excavation often is expanded so that relatively small berms are left in place against the walls.

To study the performance of berms, this latter condition was modeled with the finite element method. The finite element mesh is shown in Figure 8. The leftmost boundary of the mesh is located far enough from the wall to have a negligible effect on the system response. The rightmost boundary is located at the centerline of the cut. The computer program used for the analysis is GEOSYS, which has been described in several studies of excavation and tunneling systems[6,15].

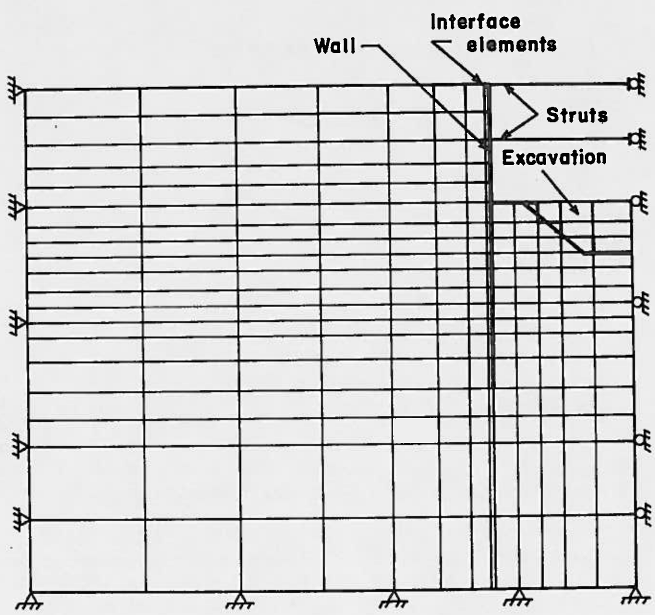


Figure 8. Finite Element Mesh for Analysis of Berm Performance.

The excavation is typical of conditions encountered during construction of the Washington, D.C. Metro. The excavation wall and soil deposit are 75 ft (23 m) and 130 ft (40 m) deep, respectively. The excavation sequence prior to the use of berms was simulated by deepening the cut 20 ft (6 m), installing and preloading two brace levels, and further deepening the cut to 30 ft (9 m). The excavation from 30 to 43 ft (9 to 13 m) was performed in four stages. At each stage, elements were removed from the central portion of the cut, thereby representing excavation with successively smaller berms until the entire lift was removed.

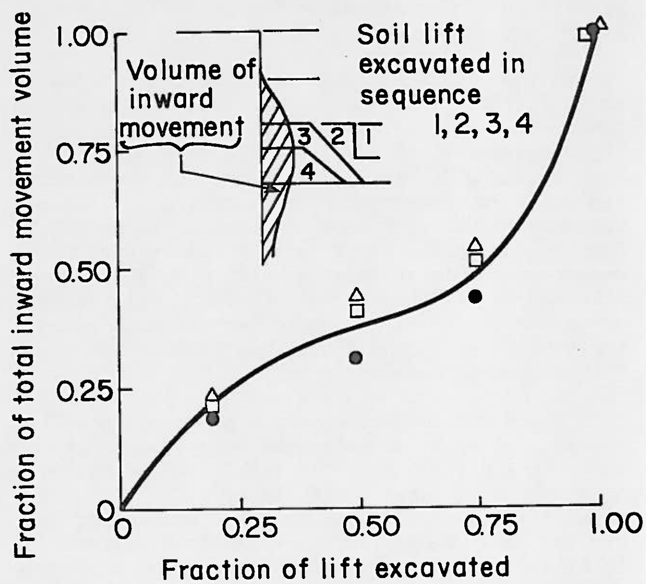
Several types of soil were modeled, including loose and dense sand and stiff clay. The soil was represented as an elasto-plastic material with a yield surface defined on the basis of the Drucker-Prager model of failure[16]. The sands were assumed to be cohesionless. Angles of shearing resistance of 30 and 35 degrees were assigned to the loose and dense sand, respectively. Stiff clay was assigned an undrained shear strength of 1000 psf (48 KN/m^2).

The intial modulus of the soil was assumed to increase in proportion to the effective confining pressure at depth. The relationship between the modulus, E_i, and the confining pressure, σ_C, was adapted from Janbu[17] as

$$E_i = Kp_a\left(\frac{\sigma_C}{p_a}\right)^n \qquad (1)$$

where K is a dimensionless modulus number, n is an exponent, and p_a is a constant equal to atmospheric pressure in the same units as σ_C. In all cases, the exponent, n, was estimated as one half. For sands, K was chosen as 240 and 1100 to represent loose and dense deposits, respectively. For stiff clay, K was chosen as 90.

Incremental movements were recorded for several combinations of wall and soil type. At each stage of the excavation from 30 to 43 ft (9 to 13 m), the volume of inward displacement was normalized with respect to the total incremental movement caused by excavation of the entire lift to show the relative efficiencies of the berms.



Symbol	Wall stiffness, EI	Soil modulus, E_s
●	115 x 10^3 K-ft^2/ft (156 x 10^2 MN-m^2/m)	23000 psi (160 MPa)
△	115 x 10^3 K-ft^2/ft (156 x 10^2 MN-m^2/m)	5300 psi (38 MPa)
□	878 x 10^3 K-ft^2/ft (1190 x 10^2 MN-m^2/m)	2000 psi (14 MPa)

Figure 9. Fraction of Total Movement Volume as a Function of the Fraction of Lift Excavated.

Figure 9 shows the fraction of the total movement volume plotted as a function of the total lift excavated. This plot is developed for three different conditions of soil modulus and wall stiffness. A section stiffness of 115 x 10^3 K-ft^2/ft (156 x 10^2 MN - m^2/m) correspond to a soldier pile and lagging wall with piles (24 W 130) on 7.5 ft (2.3 m) centers. A section stiffness of 878 x 10^3 K-ft^2/ft (1190 x 10^2 MN - m^2/m) corresponds to a 33-in (838 mm)-wide concrete diaphragm wall with soldier piles (30 W 124) embedded in the wall on 7 ft (2.1 m) centers. Each soil modulus recorded in the figure is the weighted average throughout a 50 ft (15 m) depth surrounding the excavation. Average moduli of 2000 psi (14 MPa), 5300 psi (38 MPa), and 23000 psi (111 MPa) are characteristic of stiff clay, loose sand, and dense sand, respectively.

The plot shows a consistent response regardless of the soil type and wall stiffness. Small berms appear to be relatively efficient at restricting wall displacement. By the time 80 percent of the lift is excavated, only 40 to 50 percent of the total incremental movement has occurred. The plot implies that, if berms are used on a relatively short-term basis, they will be effective in limiting movements, at least for conditions where there is little plastic deformation or heave at the base of the cut.

Sloughing, seepage, creep, and construction activity will lead to the gradual distortion of berms. These influences are not easily modeled by the finite element method. It is instructive, therefore, to look for field examples that show how progressive deformations of the berm will affect braced cut performance.

Figure 10a shows the inward wall movements for an 82-ft (25 m)-deep braced excavation in sands and interbedded stiff clay. The braced cut was part of the construction of Gallery Place Station on the Washington, D.C. Metro. Details of the construction and field instrumentation are described elsewhere[6]. The excavation walls were composed of 90 ft (27.5) long soldier piles (24 W 130) on 7.5 ft (2.3 m) centers with timber lagging. All struts were prestressed to half their design loads. Wall displacements and levels of excavation are referenced to the time at which the fourth level struts were installed and preloaded (Day 0).

The displacements show the progressive wall movement as soil was excavated from a depth of 58 to 82 ft (18 to 25 m). The total incremental volume of movement per unit length is given by the area between the lateral displacement profiles for Day 0 and Day 160. By Day 44, approximately half the total incremental movement had occurred in response to excavation at the center of the cut. This fraction is consistent with the trends shown by the finite element analyses. From Day 44 to 147, a berm was left in place against the wall of the cut. The approximate dimensions of the berm are shown in the figure. Owing to a general labor strike, no additional excavation was performed during this time interval. The wall movements increased from Day 44 to 147, until 80 percent of the total incremental displacement had taken place.

There was a relationship between wall movement and increased strut load. Figure 10b shows the change in average load for the fourth level struts as a function of time since their installation and preloading. The load increased steadily as the central portion of excavation was deepened and the berm was left in place. The average load increased approximately 180 kips (800 KN) from an initial preload of 90 kips (400 KN). Although increased temperature affected the load, the influence of temperature was small relative to that of the excavation and deformation of the berm.

When berms are used during excavation in soft to medium clays, the magnitude and time-dependency

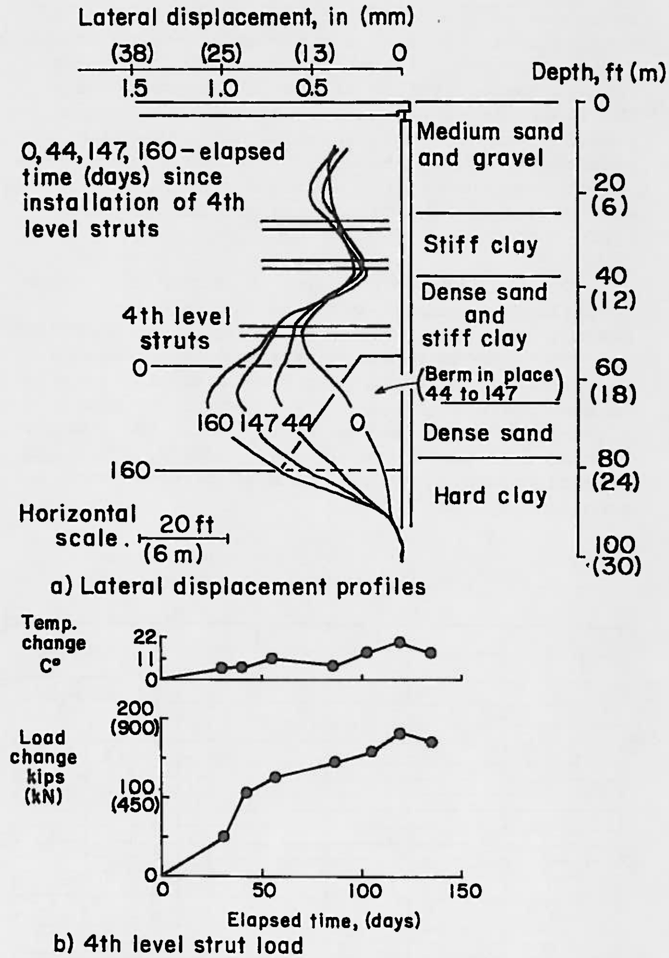


Figure 10. Observed Lateral Displacements and Strut Loads Associated with Berm Performance.

of movements will increase substantially in comparison to those which will occur for excavation in sands. As an example, Figure 11 summarizes the displacements measured during the excavation of a deep basement in Chicago.

The 26-ft (8 m)-deep excavation was extended through approximately 13 ft (4 m) of sand and stiff clay and 13 ft (4 m) of soft clay. The bottom of the cut was underlain by 20 ft (6 m) of soft to medium clay. The excavation wall was composed of steel sheet piles (MZ-32) driven to a stratum of stiff clay approximately 46 ft (14 m) below the ground surface. Measurements are summarized for three distinct stages of excavation and support. As shown in the figure, these stages include a) excavation at the center of the cut with a berm adjacent to the wall, b) partial excavation of the berm and installation of the upper rakers, and c) continued excavation of the berm and installation of the bottom rakers. All displacements and levels of excavation are referenced to the start of open cutting (Day 0).

As shown in Figure 11a, the initial excavation was performed in conjunction with a berm, 15 ft (4.5 m) wide at its top and inclined at a grade of IV:2H from a depth of 15 to 26 ft (4.5 to 8 m) below the ground surface. From Day 11 to 36, the volume of inward wall movement doubled in response to gradual deformation of the berm even though no

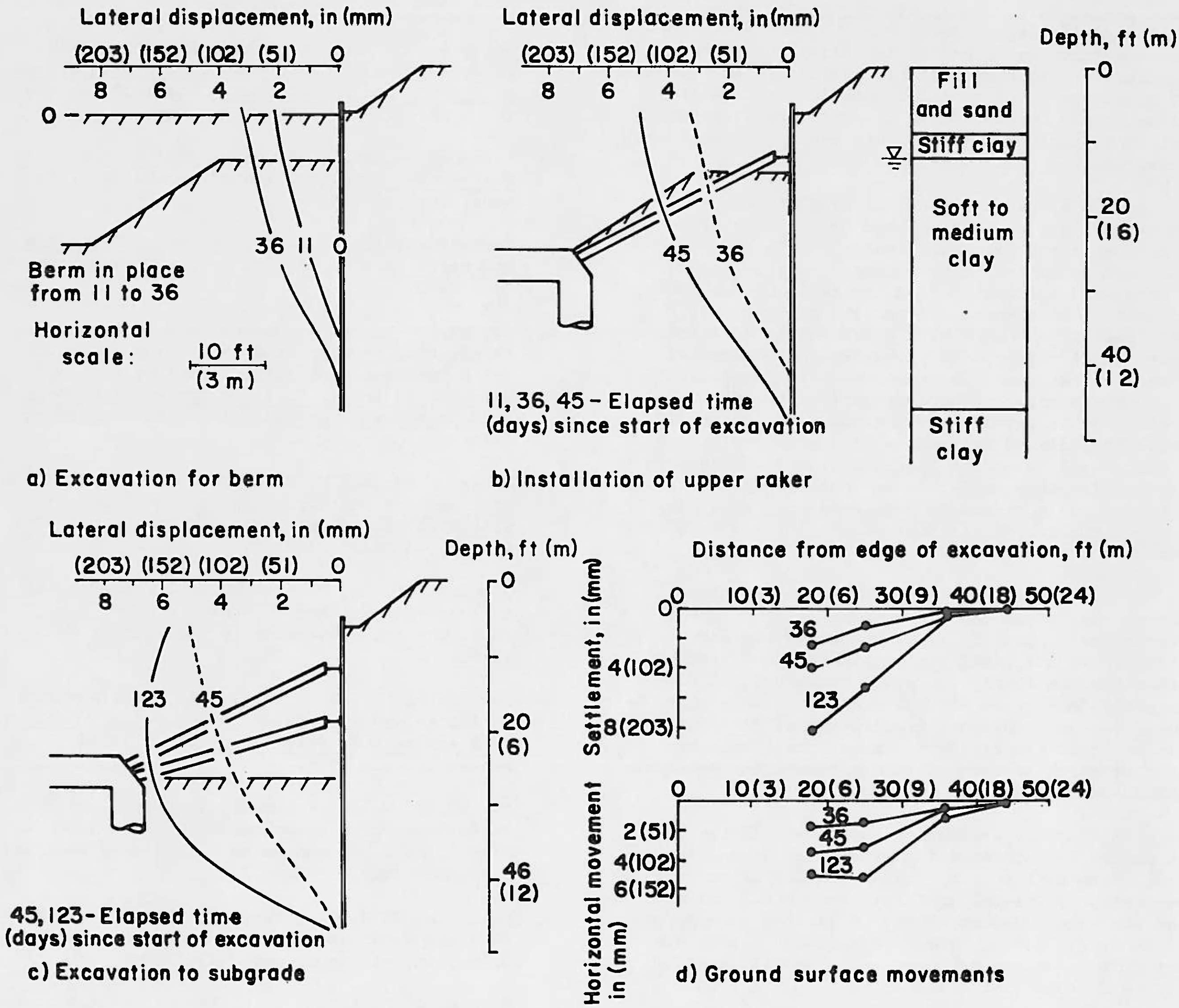


Figure 11. Observed Displacements for a Raker and Berm Excavation in Soft to Medium Clay.

additional excavation was performed. On Day 45, when the upper level rakers were installed, the berm had been cut back to about 70 percent of its initial volume. The rakers were shimmed with steel plates, but not preloaded. At this time, the wall deformation was primarily in the form of cantilever movement, with a maximum displacement of 5 in (127 mm) at the top. As the berm was cut back and the bottom rakers installed, the wall movements occurred increasingly as inward bulging. In Figure 11c, the maximum wall displacement is shown as 7 in (178 mm) at the base of the excavation. The settlements and horizontal movements measured on the street adjacent to the cut are summarized in Figure 11d.

The major part of the ground movements occurred prior to Day 45, when the upper rakers were installed. As shown by the increased wall deformation from Day 11 to 36, a significant portion of these displacements was caused by a gradual yielding of the berm adjacent to the wall. A similar time-dependent loss of ground caused by berm deformation has been reported by Clough and Davidson[18] for an excavation in soft to medium clay in San Francisco.

These observations emphasize the potential for large displacements when berm and raker excavations are performed in plastic clays. Successive reductions in the weight of the berm leads to increased plastic heave at the base of the excavation. Cutting back the berm, therefore, results in loss of both horizontal and vertical restraint. Under these circumstances, it seems prudent to use large berms and to brace the excavation wall firmly at an early stage of construction. Guidelines for proportioning berms have been developed by Clough and Denby[19]. Guidelines and rules of thumb, however, should be used carefully. The time-dependencey of deformation caused by creep and the gradual attrition from construction activities makes the use of berms a process that is difficult to generalize.

Conclusions

Because braced excavations often call for the integration of several distinct activities within the general construction plan, ground losses from each activity should be evaluated when estimating displacements. During site preparation, ground movements may be caused by 1) relocating and underpinning utilities, 2) dewatering, 3) construction of the excavation wall, and 4) installations of deep foundations. When excavation records are reviewed and summarized, the ground losses during site preparations should be considered carefully to distinguish real from apparent displacements associated with the excavation and bracing process.

Ground movements resulting from the excavation and bracing process are related to the mode of deformation of the excavation wall. Field observations and model test data show a direct relationship between wall deformation and the ratio of horizontal movement to settlement adjacent to the cut. As cantilever-type movements are allowed to dominate during construction, the proportion of horizontal to vertical movement increases and, in the limit, approaches a value of approximately 1.6. Conversely, if the wall is firmly braced at an early stage of excavation, inward deflection of the wall will lead to horizontal movements that are significantly less than settlements. Under these conditions, the ratio of horizontal movement to settlement may be as low as 0.6. The ratio will decrease further if significant consolidation occurs.

Preloading is essential to promote a stiff bracing system and thereby restrict wall movement. Nevertheless, preloads increased above the magnitude needed to induce rigid contacts within the bracing system are not likely to regain previous soil displacements nor prevent further deformation as the excavation is deepened. Practices that call for very high preloads are likely to cause local concentrations of earth pressure that may exceed the pressures assumed during design.

Limiting the excavation depth beneath the lowest braces is important for preventing ground loss during open cutting. For close control of ground movements, experience in Washington, D.C. implies that excavation depths should be limited to approximately 18 ft (5.5 m) beneath the lowest braces for deep cuts in medium to dense sands and interbedded sands and stiff clay.

Finite element analyses show that berms are effective in restraining wall movements, if they are used on a short-term basis for conditions where there is little plastic heave or deformation at the base of the cut. Sloughing, seepage, creep, and construction activity will lead to the gradual distortion of berms. Time-dependent deformations contribute to ground loss, particularly when berms are used for excavations in soft to medium clays. Guidelines and rules of thumb regarding the performance of berms should be used carefully.

Acknowledgements

Special recognition is extended to Mr. T. R. Maynard of the Bureau of Engineering of the City of Chicago for helping to obtain records of braced cut displacements and sharing his insights on excavation behavior. Thanks must be given to Professor E. J. Cording and Dr. M. D. Boscardin of the University of Illinois at Urbana for their assistance in acquiring and analyzing portions of the data. W. R. Sawbridge and T. M. Foote drafted the figures and typed the manuscript, respectively.

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TIE-BACK MEMBRANE WALLS IN VENEZUELA

Charles O. Riggs[1]

Abstract - The installation of continuously reinforced concrete tie-back retaining walls has become a common method of providing highway cut stability in Venezuela. The method is often used to prevent further movement of slide zones within residential areas. The evaluation of effectiveness and economy of design of tie-back retaining walls in Venezuela will require continued observation with comparisons of supported and unsupported cut zones in similar materials.

Introduction - The City of Caracas, Venezuela is located predominately within the valley of the Rio Guaire which flows generally from west to east parallel to the Sierra da Avila mountain range which separates the densely populated valley from the northern coast of Venezuela (1) (Figure 1). Up slope of the Rio Guaire and its tributaries the terrain is described as being extremely steep and rugged. The predominant rock type is a micaceous schist with steeply dipping phyllite stringers, random faulting and foliation shear zones. Geotechnical engineers use typical friction angles of approximately 20° and 10° for the schist and the phyllite respectively. An accurate laboratory evaluation of the shear strengths of these materials would be extremely difficult if not impossible to perform because of sampling difficulties.

The population density within the valley of the Rio Guaire has forced residential development of the rugged hillsides surrounding Caracas. Also, the development of a modern highway system to connect Caracas to port cities, agricultural areas and other industrial centers has necessitated the building of four to six lane trafficways on the sides of valleys where a winding two-lane pavement was previously sufficient.

The purpose of this presentation is to provide a basic review of the design and construction of reinforced concrete, tie-back walls that are used to assure the stability of both highway cut slopes and residential building sites in and around Caracas, Venezuela.

Highway Construction - The often-faced problem of building multi-laned highways in the hills surrounding Caracas is depicted on Figure 2. The weathered schist hillside must be cut back to provide a roadway of sufficient width. To provide reasonable grades, roadway cuts may be as high as 100 to 150 ft (30 to 45 m). Often these cuts are made without support, the highway is constructed and the slope remains stable for a period of several months. Then, following either time dependent stress relief (release of true cohesion) or relief of negative pore water pressures due to changed drainage conditions, some of the slopes begin to creep noticeably and some eventually slide. Often the slide material is hauled away and the slope, following cleanup and possible installation of horizontal drains, remains stable. This procedure represents a reasonable design practice so long as the cost of cleanup plus lost benefit to users does not exceed what would have been additional cost of real estate at the top of the slope and the cost of earth moving or the cost of retaining structures to provide an initial slide-free highway.

At some locations, such as tunnel portals or bridges, cut slopes must be stabilized for public safety. During 1975, for the construction of a multi-laned highway east of Caracas, the contractor was required to provide the design and construct a reinforced concrete, tie-back retaining wall at both entrances of a tunnel. At one tunnel entrance the roadway first crosses a bridge over a deep gorge and then "hangs" on the side of a mountain below a 100 to 150 ft (30 to 45 m) cut slope which extends several hundred feet toward the tunnel. The cut slope then bends around and over the tunnel entrance.

To assure stability of the cut slope at this critical location the contractor and designer utilized a two-way continuously reinforced, continuous concrete wall with two layers of

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deformed bars. The tie-backs consisted of a multi-strand smooth bar system (Figure 3) within the vicinity of the tunnel and a single deformed bar system in the vicinity of the bridge. The ties were approximately 100 ft (30 m) in length and were installed at variable densities according to their location below the top of the cut slope and their location with respect to the tunnel entrance. In general the tie-backs were installed approximately 5 to 6 ft (1.5 to 1.8 m) on centers within a rectangular pattern becoming much denser near the tunnel portals (Figure 4).

The reinforcing bars were approximately 1/2 in. (1.3 mm) to 3/4 in. (2.3 mm) in diameter, placed on approximately 8 in. (200 mm) to 14 in. (350 mm) centers in both directions with approximately 4 in. (100 mm) between the two layers.

The construction generally progressed by starting from the top of the cut by successively benching and installing wall segments and tie-backs as shown on Figure 5.

The concrete mix was designed to provide a cylinder strength of approximately 8000 psi (55,000 kP). The concrete was dry-batched with the water added at the "gun" during application.

The tie-backs were installed using horizontal air rotary percussion drills with air flush. Prior to installation of each tie-back, the "horizontal" borehole was cleaned further with a high volume air blast through an open pipe inserted to the end of the borehole.

At several locations between the bridge and tunnel portal, horizontal drains were installed at the base of the cut.

An inspection of the tunnel site during March of 1980 indicated no adverse stability problems at the bridge and tunnel site; therefore, either (1) the tie-back retaining walls are working very well, or (2) the tie-back retaining walls were not required. Several years of observation, comparing unsupported and supported cuts in similar materials, may be required to arrive at conclusions concerning efficiency of design.

A Residential Installation - A ridge line as depicted on Figure 6 was developed into residential building sites. The owner of Parcel A wanted to build a house at the top edge of the slope and retained an engineer to evaluate the slope and if required, make design recommendations. The engineer correctly evaluated the site and informed the owner of stability problems.

This site was somewhat unusual because of the convex shape of the hillside which causes the lateral stresses on a sliding increment to be zero or very low.

Because the owner of Parcel A could not persuade the owner of Parcel B to participate in the installation of a tie-back retaining structure, the engineer was severely restricted in solving the problem of a potential slide for the owner of Parcel A. Nevertheless, the engineer was encouraged by the owner of Parcel A to design the best possible tie-back retaining wall. The engineer proceeded with reservations.

After the wall was installed (similar to previously described walls), the engineer made frequent trips to the site to observe the performance of the hillside. In the meantime the owner of Parcel B started construction of a residence. During construction, the engineer who analyzed the site and designed the wall noticed the inevitable was about to happen. The owner of the house on Parcel A was advised to move out of the house and homeowners down the hill were advised that they might soon have additional houses on their property. (Refer to Figure 7.)

The houses did not come down, but only because the foundations were reinforced and behaved as cantilevers. The retaining wall may very well have saved the house on Parcel A, but was more than 50% destroyed as it was partially peeled back like the lid on a can of sardines.

This case presents a difficult lesson and a classic conflict in private engineering practice. While the wall was probably both a success and a failure, the failure was justified if the engineer presented a fair evaluation of risk to the client.

Conclusions - There are indications that continously reinforced concrete tie-back retaining walls can be economically utilized for difficult construction circumstances in Venezuela. They have been used at many locations within the last five years. Long term corrosion of unprotected metal tie-backs must be observed.

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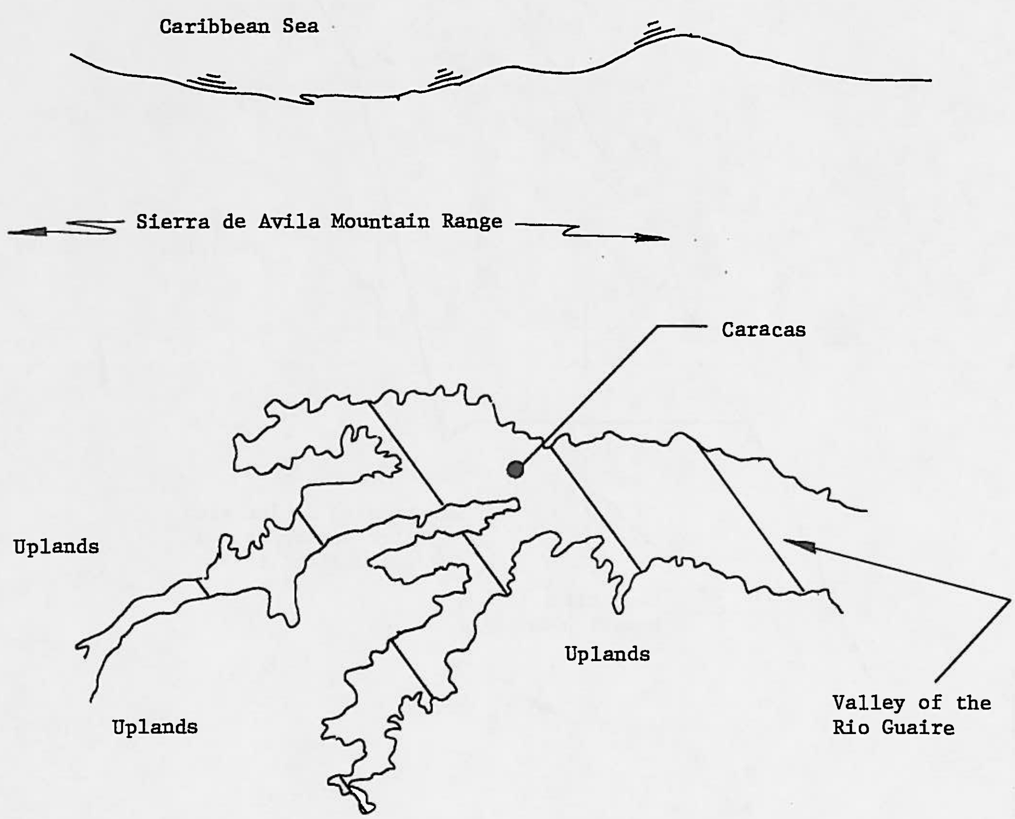


Figure 1. - Map of Caracas Showing Surrounding Uplands and Mountains (From Seed, et. al., 1972).

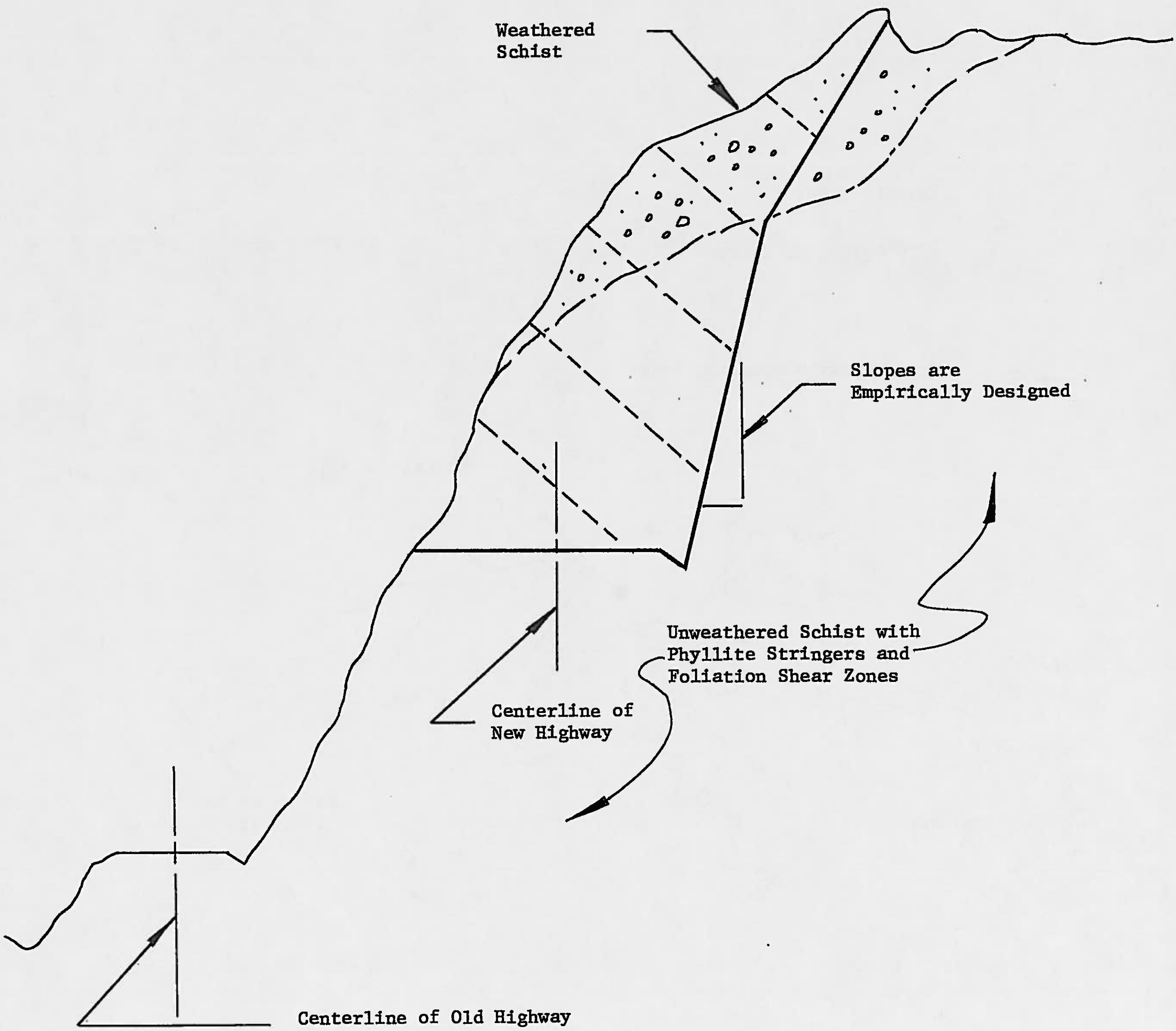
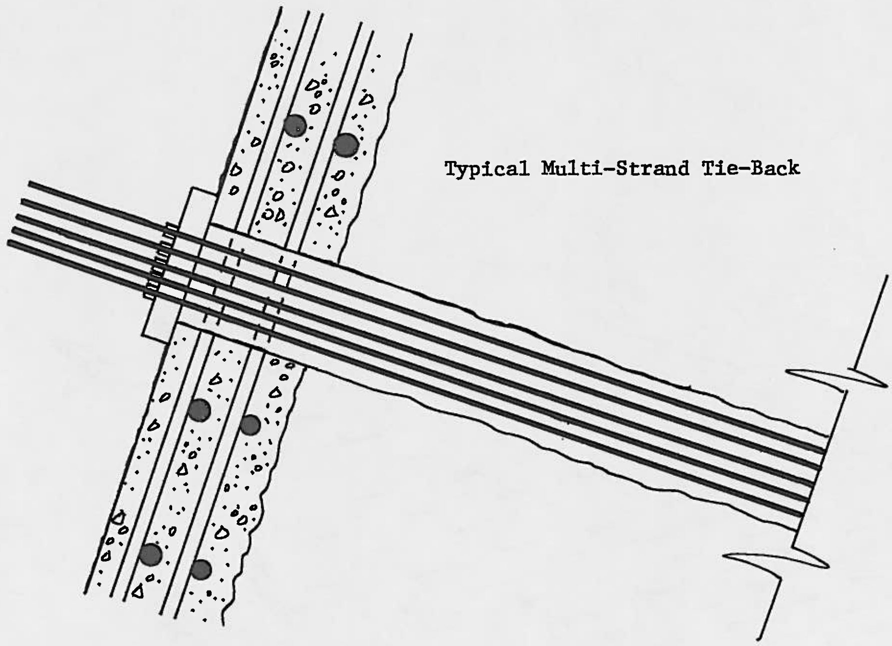


Figure 2. - Often Faced Problem of Highway Design and Construction Near Caracas, Venezuela.



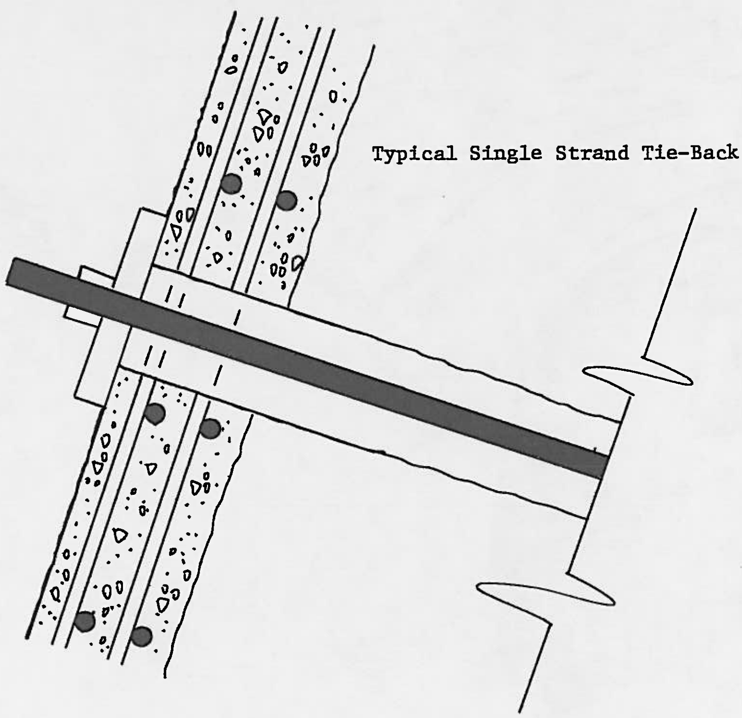


Figure 3. - Type of Tie-Backs Used at a Highway Bridge and Tunnel Site Near Caracas.

Figure 4. - Increased Density of Tie-Backs Above a Tunnel Portal.

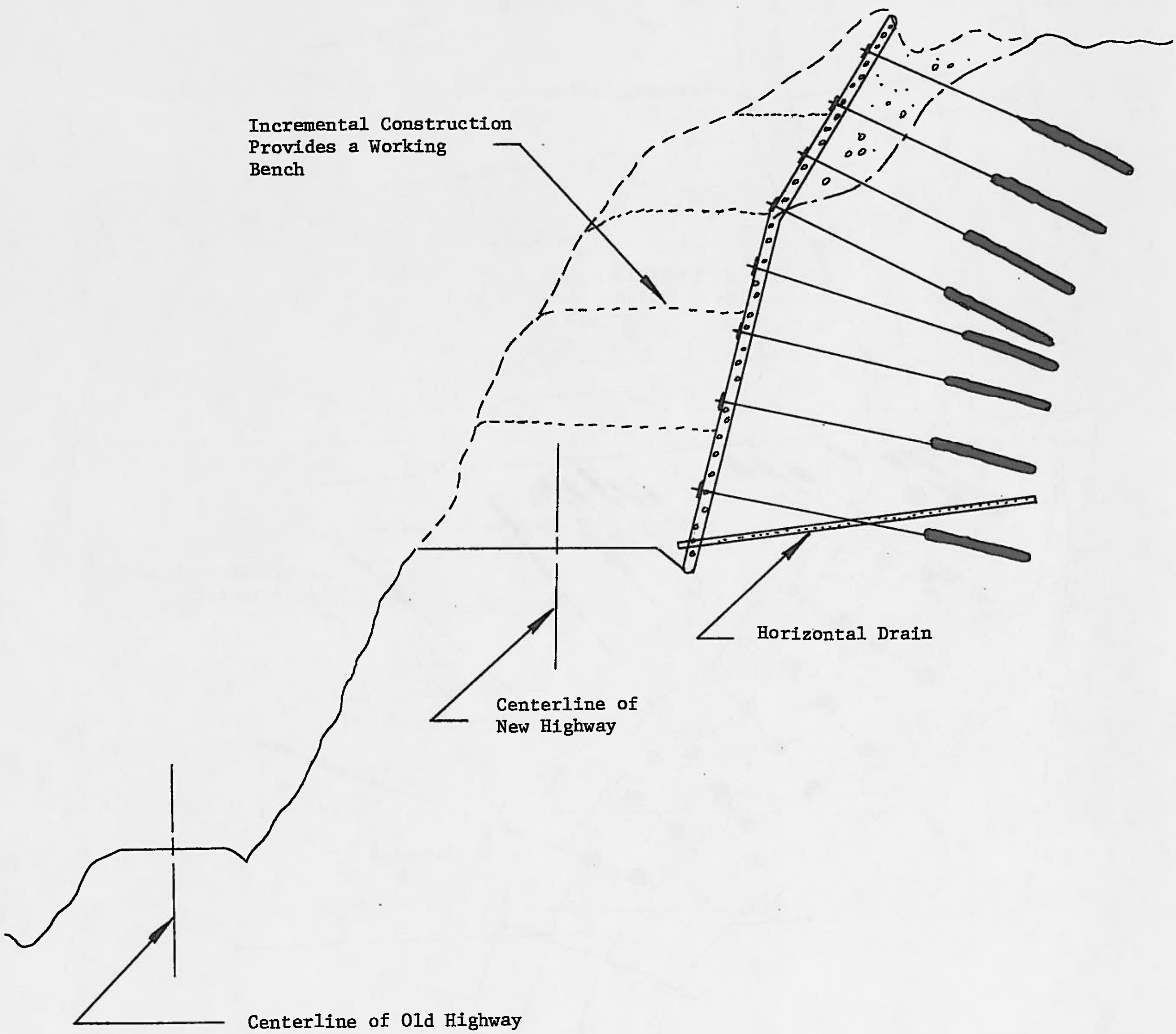


Figure 5. - Tie-Back Wall is Constructed in Vertical Increments.

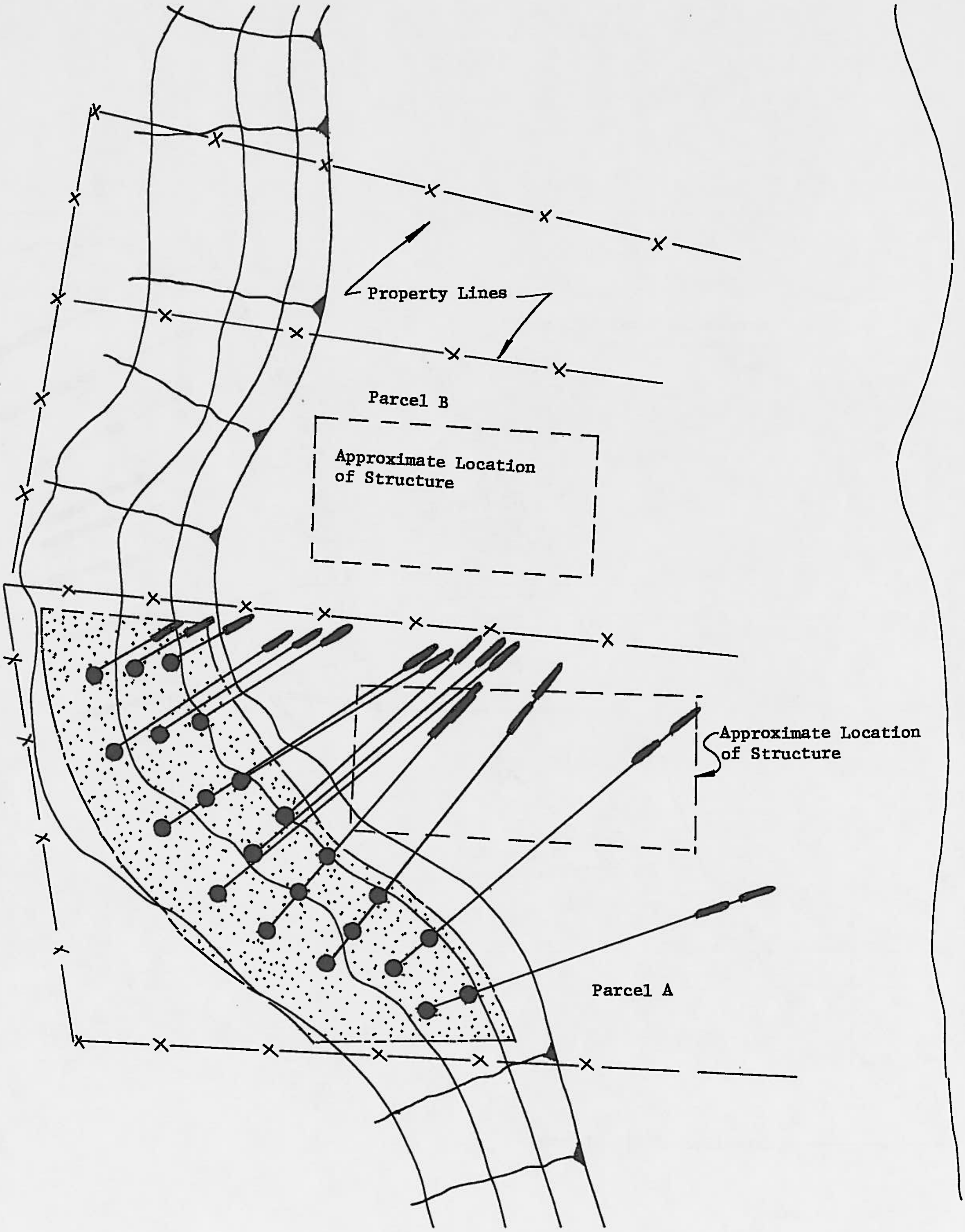


Figure 6. - Schematic of Residential Tie-Back Wall Installation.

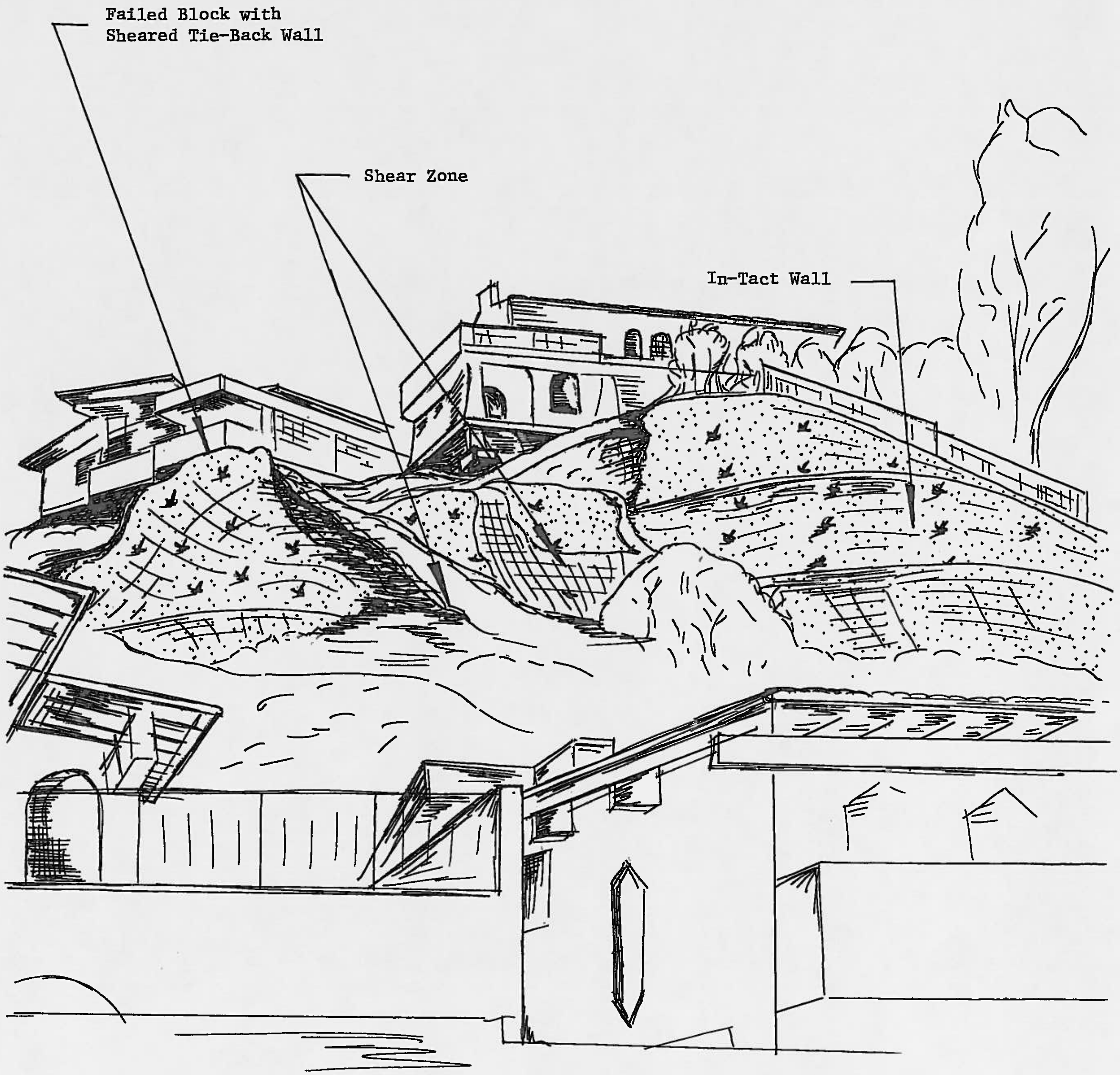


Figure 7. - Sketch of a Shear Failure Through a Residential Tie-Back Wall.

DESIGN, CONSTRUCTION, AND PERFORMANCE OF ANCHORED BULKHEADS

by

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Abstract. Anchored bulkheads have been the subject of numerous theoretical and laboratory and field experimental investigations, but a uniform design procedure has not been adopted. Documented failures of anchored bulkheads indicate that unsafe conditions are most frequently due to deficiencies in the construction procedures and the operation of a bulkhead rather than to the assumptions and techniques involved in particular design methodologies. In this presentation, the available design methods are reviewed concentrating on assumptions and concepts rather than on computational techniques; safety considerations associated with the construction and performance of anchored bulkheads are discussed; and recommendations are advanced reflecting the opinion of the authors.

Introduction

Anchored bulkheads are widely used as structural components of dock and harbor facilities to form a vertical wall for ships to tieup alongside or to form a pier which may jut out into the water. The working principle of an anchored bulkhead is schematically illustrated in Fig. 1. To provide for a vertical face, a retaining wall is constructed which should resist the pressures applied by the soils behind it. Resistance is provided by the soils in front of the wall and by the action of fixed anchors. The wall-soil-anchor system should remain stable.

Although reinforced concrete slurry walls (cast in situ or pre-cast panels) are sometimes used to construct anchored bulkheads, the material most frequently used for this purpose is steel sheetpiles. Some examples of standard sheetpile sections and high modulus sections are shown in Fig. 2. Standard sheetpile sections are divided into Z type and U type; detailed information on weights and section moduli is available from manufacturers.

Most anchored bulkheads are supported by a single row of horizontal tie rods which are connected to isolated anchorages or deadmen. These may be plain concrete blocks, vertical reinforced concrete slabs, or small groups of short sheetpiles. In some cases, anchorages are constructed as continuous vertical walls. Alternatively, the anchorage may take the form of tension steel H piles raking back from the wall or soil tiebacks which can be installed as steep as 45°. Horizontal tie rods may be connected to A-frame anchorages formed of forward and backward raking bearing and tension piles with a small or large pile cap. Typical anchor types are shown schematically in Fig. 3.

Before proceeding to the design of an anchored bulkhead, it is necessary to evaluate the pressures and resistances that will act on the wall during the various stages of construction and after completion of construction. Therefore, it is essential to determine the engineering properties of the in-situ soils and of materials to be used as backfill. Soil properties that are significant to anchored bulkhead design include unit weight, cohesion, angle of internal friction, stress-strain relationship, and volume change or consolidation characteristics. Furthermore, forces acting on the wall due to tidal and wave action, mooring loads, impact of vessels, surcharges, and earthquakes should be evaluated.

A rather large number of methods exist for the design of anchored bulkheads and widely varying results may be obtained when methods involving different assumptions and computation procedures are employed. For safe and economic structures to be constructed, it is necessary for the practicing engineer to develop (a) a thorough understanding of the assumptions, limitations, level of effort, and cost associated with the various available design methods, and (b) an appreciation of the factors which affect the safety of the structures.

Accordingly, the scope of this presentation includes (a) a review of available methods for the analysis and design of anchored bulkheads, with emphasis placed on a discussion of assumptions and concepts rather than on a detailed presentation of computational techniques, (b) a discussion of safety-related problems associated with the construction and long-term performance of anchored bulkheads, and (c) recommendations which also reflect the opinion of the authors.

Design Methods

Conventional design methods for walls of anchored bulkheads are generally divided into two broad categories, the free-earth and the fixed-earth support methods (Terzaghi, 1943). Walls designed according to the free-earth support method have lesser penetration than walls designed according to the fixed-earth support method. The former are assumed to act as vertical beams spanning two supports, one at the anchorage level and another at the bottom of the wall. The latter are assumed to be fixed in direction at the bottom and to act as vertical propped beams fixed at the lower end.

Through the years, modifications have been recommended and frequently accepted for these basic design methods. These modifications were based on the results

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of model or full-scale experimental investigations, adaptation and improvement of numerical techniques, and advances in computer capabilities. The design methods discussed next include those that are most frequently used at the present time. However, the material presented herein should not be considered as a summary of all literature that is available on the subject.

Free-Earth Support

This method is the oldest and most conservative of all. It is based on the questionable assumption that all the soil below the dredge line has reached its limit shear strength throughout the depth of sheetpile embedment and is, therefore, incapable of producing effective restraint of the sheetpile to the extent necessary to reverse the bending moments.

Failure is assumed to occur by rotation about the anchor rod. Passive pressures develop in the soil in front of the wall and active pressures develop behind it. Typical net-pressure diagrams are shown in Fig. 4. The depth of embedment is computed by taking moments about the anchor rod and setting their sum equal to zero. The anchor force is then computed by considering force equilibrium in the horizontal direction. Safety factors are introduced by (a) considering that the passive pressures are not mobilized fully and computing the depth of embedment according to a reduced passive pressure diagram; (b) increasing the computed depth of embedment by 20% or more; and (c) increasing the computed anchor force. Bending moments, shears, and deflections are calculated by treating the wall as a beam with simple supports at the anchor rod level and at the bottom. The computed large maximum bending moments require the use of sheetpiles with heavy cross section.

Moment Reduction for Free-Earth Support

Based on theoretical analyses and extensive experimental investigations on model anchored walls, Rowe (1952, 1955a, 1955b, 1956, 1957a, 1957b, 1957c) proposed a technique for reducing the maximum bending moment computed according to the free-earth support method. This technique is recommended for use with uniform silty sand or sand deposits and may also be applied to clay deposits. The design takes into account the mechanical behavior of the sheetpiles by introducing a flexibility number ρ, which is a function of the total length of the sheetpile H, the modulus of elasticity E, and the moment of inertia of the sheetpile cross section I($\rho = H^4/EI$).

Design by the moment reduction method proceeds by first making a free-earth support analysis to determine the maximum bending moment and the length of the sheetpiles. Depending on the anchor rod location, the length of the sheetpile, and the soil type, an appropriate moment reduction curve is selected from available graphs, such as those shown in Fig. 5. Using a table of sheetpile sections, the actual bending moment of piles and the corresponding flexibility number can be computed. Using the ratio of the actual bending moment to the maximum bending moment from free-earth analysis and the flexibility number, a second curve is plotted. The intersection of the two curves indicates the most economical sheetpile section that can be used safely.

Fixed-Earth Support

In this method, it is assumed that the soil beneath the dredge line exercises effective restraint on the bulkhead deformations. As a result, the bulkhead acts like a beam which is partially fixed at the bottom and subjected to bending moments, and which deflects somewhat like the line shown in Fig. 6. To obtain fixity, passive earth pressure is assumed to act on the back of the wall, at the lower end, and this results in a net-pressure diagram as indicated in Fig. 6. The point of fixity of the sheetpile is defined as that point of the embedded portion where the tangent to the elastic line is vertical and passes through the anchor rod point on the upper portion of the sheetpile. The elastic line has a point of inflection (or contraflexure) indicating that both positive and negative bending moments are applied on the sheetpiles. However, the location of this point is unknown and this complicates the design procedures.

The design proceeds by assuming a distance from dredge line to the point of fixity, replacing all passive pressures below that point by a force acting outward at the point of fixity, constructing the bending moment diagram, and computing the tangential deviation of the point of fixity from the anchor-rod point. This procedure is repeated by revising the assumed location of the point of fixity until the computed tangential deviation is within stipulated tolerance. Then, the depth of embedment is taken as 20% larger than the depth of the point of fixity from the dredge line. This procedure requires substantial effort and, although graphical solutions have been developed, its application is laborious.

Equivalent Beam

A simplification of the fixed-earth method, called the equivalent beam method, has been extensively used because it eliminates the possibility of having to make many trials to obtain the location of the point of fixity. The simplification becomes possible by assigning a fixed position to the point of contraflexure of the elastic line. The distance to the point of contraflexure has been evaluated as a function of the angle of internal friction of the soil below the dredge line (Blum 1931, Verdeyen and Roisin 1961). Therefore, this method is applicable for cohesionless soils only. Furthermore, the method should not be applied where significant differential water level may develop, when the surcharge load is large compared to the wall height, or when the tie rods are located at an exceptionally low level.

As shown in Fig. 7, this method is applied by first developing the net earth pressure diagram as a function of the unknown depth from the dredge line to the point of fixity. The sheetpile is separated into two sections by assuming a hinge at the point of zero bending moment (point of contraflexure) and by computing reactions and moments, an equation is obtained in terms of the unknown depth to the point of fixity (Bowles 1968).

Further Modifications of the Equivalent Beam

A number of modifications to the equivalent beam method have been proposed to simplify the computation and eliminate the uncertainty of selecting a position for the point of contraflexure as a function of soil properties. Anderson (1956) recommended locating the point of contraflexure at the point of zero net earth pressures and then proceeding with the equivalent beam method. Tschebotarioff (1973) recommended that, for cohesionless soils, the point of contraflexure be taken at the dredge line. The sheetpile over the dredge line is considered to be a simply supported beam with overhanging end; the depth of embedment is computed as 43% of the height of the sheetpile above the dredge line and this is supposed to provide a safety factor at about two.

Danish Rules

Based on studies of a number of existing sheetpile structures, the Danish Society of Civil Engineers has proposed a set of empirical guidelines, known by the name of Danish Rules, for the design of anchored bulkheads (Tschebotarioff 1973; U.S. Steel 1974). Earth pressure distributions similar to those assigned according to the Danish Rules have not been obtained from any model or field experimental measurement. Use of the Danish Rules is considered to lead to potentially unsafe designs, especially with respect to depth of penetration of the sheetpile which may be quite insufficient.

Recent Development in Classical Methods Design

Use of classical methods for the design of anchored bulkheads can lead to widely varying results according to the methods and assumptions employed. A computer program recently developed for the US Army Corps of Engineers (Dawkins 1979) provides the practicing engineer with the means for making classical design according to five different methods, comparing the results, and selecting a safe and economical design. This program is available through the Corps of Engineers WES G-635, CSC H6000 at Macon, Georgia, and Boeing Computer Services.

The general wall-soil system considered in this program is shown in Fig. 8. A one-foot slice of a straight, initially vertical, linearly elastic wall with a constant cross section throughout its depth is considered. The single horizontal anchor is attached to the wall at any elevation and is assumed to prevent horizontal displacement of the point of attachment. Soil surface and soil layer interfaces can be horizontal or inclined. The properties required for each soil layer are the unit weight, the angle of internal friction, the cohesion, and the angle of wall friction. Water surface can be at any elevation. Various types of surcharge loads can be accommodated, and earthquake effects are taken into consideration, by applying empirical corrections to the unit weight of the soils.

Design pressure distributions are computed according to classical soil mechanics methods, assuming that fully plastic state develops in the soil on both sides of the wall, but reducing the computed passive pressures by a factor of two. According to the geometry of the soil layers and the type of surcharge applied, the Coulomb earth pressure or a wedge method are used. Lateral loads due to surcharge and water pressures are superimposed on the net soil pressure diagram.

Five procedures are incorporated in the computer program for the design of anchored bulkheads and the analysis of existing ones. These are: (a) the free-earth method, (b) the fixed-earth method, (c) the equivalent-beam method, (d) the equal-moment method, and (e) the Terzaghi method. The important features of each of the procedures are summarized in Fig. 9. A one-dimensional finite element procedure for linear, prismatic beams is used for the structural analysis for each case.

Numerical Analyses

Several attempts have been made to develop a theoretical analysis which can be used to determine stresses and strains in an anchored bulkhead. Earlier approaches were based on the assumption of linear pressure-deflection response of the soils at each depth below the dredge line, and the stiffness of the soil mass below dredge level was being approximated by a theory of subgrade reaction. An elasto-plastic model has been proposed by Terzaghi (1955) and supported by experiments by Rowe (1955a). The model is based on the concept that the earth pressure acting at any point on the wall is dependent on the strain experienced by the soil as the wall deflects. The active and passive states of stress constitute the limit cases of the pressure-strain relationship. Between these two states, the horizontal soil stress acting on the wall is equal to the product of the displacement times the coefficient of subgrade reaction which varies with soil type and strength. Values for the coefficient of subgrade reaction are difficult to establish and this limits the usefulness of computer programs which require selection of values for the coefficients of subgrade reaction. A method for computing subgrade reaction coefficients using the results of Menard pressuremeter tests has recently been proposed (Baguelin, Jezequel, and Shields 1978).

As the power of digital computers increased, analyses incorporating non-linear soil response became possible and were employed by Rauhut (1966) and Haliburton (1968) using the beam-column element. The beam-column was considered as being divided into short equal lengths (crude finite elements), each of which could rotate without bending, experience transverse forces and couples, and be subject to transverse and rotational restraints. Each element could also sustain an axial force which contributed only to the bending moment. The soil was essentially replaced by a series of unconnected, non-linear springs. This implies that the soil was not treated as a continuum, arching effects could not be modeled, and loads on the walls originating remote from the wall had to be estimated by separate analyses. The uncoupled nature of the soil response and the difficulty to estalish realistic non-linear subgrade reaction mechanisms, limits the applicability of these methods in practice.

True Finite Element Analyses

Sophisticated two-dimensional mathematical models of flexible walls and their surrounding soils have been developed in an attempt to obtain a more realistic representation of soil-structure interaction. In particular, Bjerrum et al (1972) analyzed three walls in loose sand. The soil continuum was represented by four-noded rectangular plane strain elements and was assigned non-linear stress-strain properties (hyperbolic elastic relationship) in accordance with the procedures described by Duncan and Chang (1970). Two columns of rectangular plain-strain elements were used to represent the wall and one-dimensional joint elements were used to represent the interface between soil and wall to allow for the non-linear development of wall friction. Driven and dredged conditions were analyzed. Excavation was simulated by calculating the stress at the boundary to be created and applying equivalent and opposite nodal loads to the finite element mesh, thus creating a stress-free surface. Elements having been excavated were assigned very low stiffness for subsequent computations. The computed maximum bending moments were in good agreement with those estimated using free-earth support and moment reduction methods for walls in loose sand.

A parametric study of anchored bulkheads in sands with a single tie was conducted by Smith and Boorman (1974) who developed three finite element programs for this purpose. The first program, called "two stage", was similar to that of Bjerrum with the basic difference that bending rather than plane-strain wall elements were used. The second program, called "one stage", provided a more realistic representation of the stress-strain response of soil elements during excavation. The third program, called "plastic", allows elements to deform perfectly plastically with no volume change after the Mohr-Coulomb value of failure deviator stress is achieved.

Using the "one stage" and "plastic" programs, the results of model tests conducted by Rowe (1955b, 1956, 1957b 1957c) were reproduced analytically for a wide range of wall and soil conditions. The analytical results obtained by Bjerrum were also reproduced using "two stage". The results obtained strongly indicate that design methods of the type recommended by Rowe can form the basis of modern design practice. Furthermore, use of finite element techniques allows the successful modeling of soil heterogeneities and spatial soil property variations as well as construction sequence.

Design of Anchorage Systems

The anchor force, usually per linear foot of wall, is computed as part of the procedures for the design of the wall of an anchored bulkhead. It is then necessary to design a system which will transfer this force safely to the soil; this can be achieved by a variety of anchorage systems. In contrast to the numerous, and diverse, methods that have been developed for the design of the walls of anchored bulkheads, the anchorage systems are designed according to well-established conventional approaches. Rather than considering which design method to adopt, the engineer has to render a decision as to which type of anchor to use. This decision is based on a comprehensive review of site conditions and available structural components, and includes the selection of anchor inclination, horizontal and vertical anchor spacing, structural design of the anchorage system, location of the anchorage, and evaluation of the resistance of the anchorage to loading.

The three main types of anchorages have been shown schematically in Fig. 3. In general, the anchorage system derives its stability from the passive resistance of the soil in front of it and near the ground or backfill surface, or from soil layers at a much lower level. In the latter category are included battered tension piles, soil anchors, and A-frame anchorages. Anchorage systems in the former category consist of horizontal tendons or tie rods attached to isolated or continuous anchorages.

A basic requirement to be satisfied by any type of anchorage system is that the anchor forces should be transferred to a soil mass which is not affected by the possible active failure zone behind the wall of the bulkhead. A second requirement is that settlement in the backfill or in the underlying soils should not affect the safety and performance of the anchorage system. Finally, anchorage systems should be designed for the worst possible loading conditions using large factors of safety.

The horizontal spacing of the anchors is influenced by their load-carrying capacity, as well as by the horizontal continuity of the wall. With the exception of reinforced concrete retaining walls which are built in successive horizontal blocks, all other types of walls are discontinuous even though some interlocking may be provided (such as for sheetpile walls). When walls are made of separate vertical panels (reinforced concrete or slurry walls), each panel should be anchored individually unless the panels are connected together by a horizontal beam.

When inclined soil anchors or battered tension piles are used, the reaction of the anchorage system on the wall of the bulkhead is inclined, and a force directed downward is applied on the wall. This force should be safely supported by end-bearing and/or by skin friction along the embedded portion of the wall. When wall stability under this type of loading may not be feasible (for example, when sheetpile walls are embedded in weak soil layers), inclined anchor forces should not be used. Use of A-frame type anchorages provides for horizontal anchor forces and, at the same time, transfers the loads to soil layers at substantial depths from ground or backfill surface.

Anchor tendons may be formed of bars, wires, or strands, and the choice of an appropriate tendon type depends on conditions. Strands and wires have advantages with respect to tensile strength, ease of transportation, storage, and fabrication. Bars are more readily protected against corrosion and are often easier and cheaper to install in the case of shallow, low-capacity anchors. In the United States, bars (tie rods) are most frequently used. Tie rods are usually structural steel bars with upset threaded ends to avoid a reduction in the net cross-sectional area due to the threads.

The working load of the tie rods is usually assumed to be 20% higher than the computed anchor force and is used for the structural design of the tie rod. When soil layers below the tie rods consolidate, the resulting deformation and sagging of the tie rods may cause high stresses to develop in the tie rods and their connections to the wall and the anchor. Because of their ductility, the tie rods rarely fail; however, this is not the case with the connections which should be designed with a higher factor of safety. To avoid the detrimental effects of settlement, tie rods can be supported by light vertical piles or can be installed within concrete pipes which allow a settlement equal to their diameter to occur before the tie rods are affected. If the connections of the tie rods to the wall and the anchorage are rigid, large negative bending moments will develop near those points as a result of settlement. To prevent overloading and failure, hinges can be provided near the connections.

Connections between the tendons or tie rods and the wall are facilitated by the use of flexural members (wales). When steel is used as the structural material, the wale normally consists of two spaced channels placed with their webs back to back in a horizontal position. Typical details of a steel wale are shown in Fig. 10. The best location for the wales is on the outside face of the wall, but they are sometimes placed inside the wall to provide a clear outside face. For design purposes, the response of the wall can be considered to be somewhere between that of a continuous beam on several supports (the locations of the tendons) or a single span on simple supports. This, however, is only an approximation and an exact analysis would have to take into account the stress-strain behavior of the tendons, the rigidity of the wale, and the residual stresses which develop during bolting operations.

Definite guidelines exist with respect to the location of the anchorage in the soil mass behind the wall of the bulkhead. For anchorage systems which derive their stability from the passive resistance of soils, typical guidelines are illustrated in Fig. 11. In cohesionless soils, the anchorages should be located within the shaded zone shown in Fig. 11a to avoid interaction between the passive soil wedge of the anchorage and the active wedge of the wall. For cohesive soils, two criteria are suggested. First, the tie rods should not be shorter than the total height of the wall; and, secondly, the horizontal resistance to shearing for a length equal to the tie rod length should be checked. If it is not possible to locate the anchorage according to these guidelines, the allowable resistance of the anchorages should be reduced according to methods described by Terzaghi (1943).

The overall design of anchorage systems should be based on the least favorable combination of conditions which are applicable to the structure as a whole. For example, the anchor force may be smallest at high water, but the anchorage resistance may also be a minimum for this condition if the anchorage becomes partially or totally submerged below groundwater level. When the design is based on the resistance generated from a passive soil wedge, at most, only one-half of the passive resistance is considered to be mobilized. Based on model tests and theoretical analyses, simplified procedures have been proposed for the design of anchor slabs or deadmen in granular or cohesive soils (Ovesen 1964, Brinch Hansen

1966, Mackenzie 1955). The development length of battered tension piles or soil anchors is determined according to the properties of the soil layer in which they are embedded and the anticipated interaction between the anchorage and the soil.

Construction and Performance Considerations

The discussion of design methods indicates that anchored bulkheads can be designed according to any one of a rather large number of methodologies. This observation, combined with the fact that thousands of anchored bulkheads are operating successfully and that a relatively small number of failures have been reported in the literature, may lead to the conclusion that anchored bulkheads can be designed, constructed, and operated safely. However, it must be emphasized that a knowledge of the mechanics of earth pressure and of the design of structural systems alone does not guarantee safety during construction and operation of an anchored bulkhead. Accordingly, the practicing engineer should have a thorough understanding of the factors that could cause anchored bulkheads to fail in any of several ways. The following discussion provides a summary of such considerations.

Failure Modes of Anchored Bulkheads

The main types of failure mechanisms of anchored bulkheads are schematically shown in Fig. 12 and can generally be characterized as due to soil failure around the bulkhead or due to failure of the structural components of the bulkhead.

Toe Failure. Failure of this type (Fig. 12a) can occur if there is inadequate penetration of the wall, if the depth of the dredge line is inadvertently lowered below design levels, or if the shear strength of the soils around the bottom of the wall is reduced.

Anchor Failure. Wall failure can occur due to the inadequacy of the bolts attaching the wales to the sheetpiles, the wales, the tie rods and their end fixings, or the anchorages themselves. Excessive settlement of the backfill can also cause failure of the anchorage system because it can drag down the tie rods and cause excessive stresses in them and their attachments.

Penetration Failure. When inclined anchors are used, the wall is subjected to axial forces. If the depth of penetration of the wall is not sufficient to resist these axial forces, or if the shear strength of the soil at the bottom of the wall is reduced, the wall will penetrate into the underlying soils and, at the same time, move outward as shown in Fig. 12c.

Slip Failure. A slip surface which passes below the wall (Fig. 12d) may develop as a result of changes in loading or in the shear strength of the soils around the bulkhead.

Wall Failure. This type of failure will develop if the flexural resistance of the sheetpiles is exceeded of if buckling develops due to axial loads. Although this type of failure is a possibility, an actual failure has not been reported.

Excessive earth pressure, inadequate consideration of deflection, poor design details, corrosion or deterioration of the structural components, and lack of consideration of effect of construction operations are other causes of failure. Most of the reported failures of anchored bulkheads can be attributed to toe failure, anchor failure, and slip failure. However, there is usually more than one cause of failure and it is seldom possible to evaluate the relative importance of each (Sowers and Sowers 1967).

Construction Sequence

The success of an anchored bulkhead depends on the sequence of construction activities more than on any other single factor. Excessive lateral movement, overstressing, and failure result from improper construction methods more often than from inadequate design. Proper inspection and monitoring should be provided during construction to ensure that design requirements are met and that any potentially hazardous conditions, which were not identified during subsurface exploration and design, are identified promptly and corrected properly. Examples of conditions which can lead to failure during construction are discussed in the next sections.

Dredging. The time and method of dredging is often critical. When specified in design, removal of soft material should precede driving of sheetpiles. However, if the existing soil behind the wall is to remain in place, dredging in front of the wall should be made after completion of the structure. Because the lateral loads during backfilling are frequently larger than those occurring at any other time in the life of the bulkhead, it is advantageous to obtain lateral resistance from material existing over the level of final dredging. Post-completion dredging should be performed in stages to avoid uneven and rapid changes of loading conditions on the bulkhead.

Backfilling. The process of backfilling must be rigidly controlled. As shown schematically in Fig. 13a, backfilling should progress away from the wall rather than toward it to avoid trapping of soft soil adjacent to the wall or creating a mud wave. Cases of such failures have been described by Sowers and Sowers (1967) and Tschebotarioff (1973). Failure of the bulkhead shown in Fig. 13b was attributed to excessive earth pressures due to mud waving and inadequate anchor system. However, if the safe performance of the bulkhead depends on the generation of passive resistance at the anchorage, filling against the wall should not be performed until the backfill has been placed in front of and to finished design grade above the anchorage.

Surcharging. If compressible soil layers cannot be practically removed, it may be necessary to improve their properties and reduce potential future settlement by surcharging the area. This should be done before constructing any part of the bulkhead. The bulkhead shown in Fig. 14 (Sowers and Sowers 1967) failed for a length of 100 ft when bending at the batter pile anchors overstressed the welds at the top, progressively ripping them off completely. The batter piles deflected from the weight of the sand backfill as the clay below consolidated. Similar effects can be produced by surcharge placed on ground surface after completion of backfilling.

Pile Driving Behind Bulkhead Wall. Relieving platforms are sometimes constructed to reduce surcharge loads on bulkheads during operation. This requires driving of piles through the backfill and into in-situ soils to support the platform. The directional sequence of pile driving may be very important. In the cases shown in Fig. 15, the piles were driven from the anchorage toward the wall. In the sandy backfill (Fig. 15a), a progressively increasing lateral pressure was applied on the sheetpiles and resulted in failure of the anchor-bolt connections. Although no catastrophic failure developed for the case shown in Fig. 15b, the sheetpiles deformed outward by as much as 18 in. at the mud line, and inward at the top. The volume of the clay in the outward bulge was about the same as that of the displacement of the piles in the clay strata.

Pile driving near the wall of an anchored bulkhead may cause excess pore-water pressures and temporary reduction of shear strength of the soils near the bottom of the wall. This can lead to excessive downward movement of the wall due to loss of bearing capacity, outward deflection at the toe, and even circular slip failure. Cases where these types of failure were documented have been presented by Broms and Stille (1976) for anchored walls in soft clay.

Hydraulic Considerations

Different water levels can exist between the back and the front of the wall of an anchored bulkhead. This can be the result of (a) tidal fluctuations of the free water level in a sea environment, (b) rapidly receding high water level of lakes and rivers, and (c) heavy rainfalls which saturate the backfill. As a result, unbalanced water pressures are applied on the wall and seepage takes place around the tip of the wall. As shown in Fig. 16, upward directed seepage forces in the soil in front of the wall reduce the effective unit weight of the soil and, consequently, reduce the passive earth pressure. These effects should be quantitatively evaluated using appropriate procedures, such as flow net construction, and accounted for in the design of a project.

Although the effects of differential water level across the wall may be taken into consideration in the design of an anchored bulkhead, every effort should be made to minimize the potential water level differential across the wall by utilizing backfill materials that drain readily and providing the wall with a drainage system which, in its simplest form, consists of weepholes with graded filters or geotextiles behind them. Even if overloading of the bulkhead is prevented, poorly designed filters can lead to erosion, particularly in fine sand backfills. Fluctuations of groundwater create a gradient toward the wall or the drain. If a graded filter is not provided, the soil can be eroded and piping will develop. The resulting volume changes and possible settlement in the backfill may have detrimental effects on the anchorage system (overloading of tie rods) and on structures located at ground surface (differential settlement and cracking).

Corrosion

Steel members of anchored bulkheads (sheetpiles, anchor tendons, or tiebacks) are subject to the detrimental effects of corrosion, especially if they are in salt water or in water with acid industrial waste. The corrosion reaction is controlled by the water and the electrochemical potential (pH). For a corrosive reaction to occur, an ionic medium is required. Water provides this medium through the many ions in solution. Renewal of the water increases the corrosion risk and a supply of oxygen accelerates the corrosion reaction.

In general, exposure of sheetpile structures in water is described as occurring in one or more of five zones, as indicated in Fig. 17. The most critical zone in seawater service is the splash zone (area from mean low tide to upper limit of wave action). Special steel compositions (such as the USS Mariner) are available and should be used to ensure long-term satisfactory performance of a bulkhead. Protection can also be provided by applying special coatings, encasement in concrete, and cathodic protection.

Repetitive Loading

Anchored bulkheads may be subjected to a systematic or regular repetitive change in load. For example, the lateral loading on an anchored wall caused by tidal fluctuations is a repetitive loading that may occur a few tens of thousands of times throughout the life of the structure. Dynamic loads, such as those due to earthquake or wind, are associated with high inertia forces and are not comparable to the static forces described above. Little information exists regarding the effect of repetitive loading on anchored bulkheads. However, the results of model tests on plate anchors lead to some interesting observations.

A comprehensive model test study of the behavior of plate anchors in dry sand subjected to repetitive loading was conducted by Hanna et al (1978). Up to 100 000 sinusoidal cycles were used, and it was observed that the anchor displacement depends primarily on the amplitude of the cyclic load. As illustrated in Fig. 18, the higher the cyclic load, the smaller the number of cycles required to cause failure or large displacement. Even a repetitive load equal to 25% of the ultimate static capacity of the anchor caused large deformations after 20 000 cycles. However, failure in the form of sudden pullout never occurred and the anchors became stiffer with increasing number of cycles. Hanna et al (1978) also subjected the plate anchors to alternating repetitive loads (tension to compression) and observed a great reduction in the number of cycles to failure depending on the alternating load. Andreadis et al (1978) performed a similar study of plate anchors in saturated sand and observed a decrease in the number of cycles to failure with increasing cyclic load amplitude. For a cyclic load level of only 20% of the ultimate static capacity, a significant increase in strain occurred after 5000 cycles.

In general, repetitive load tests on anchors indicate that repetitive loading can cause failure even at a load substantially below their ultimate static capacity. This effect is not taken into consideration by the methods presently used to design anchored bulkheads. However, the potentially detrimental effects of repetitive loading can be minimized, if not eliminated, by providing the anchored bulkhead with a backfill which can drain at a rate equivalent to that of the rate of change in water elevation.

Dynamic Loading

Dynamic lateral forces on anchored bulkheads can result primarily from earthquakes, explosions, and vehicular traffic. The effects of moving loads can readily be taken into account in design by the application of appropriate equivalent surcharge or line loads. Earthquakes and explosions have many similarities, but they differ basically due to the direction of propagation of motions through the soil formations (upward for earthquakes, horizontal for explosions). A frequently quoted rule for avoiding damage from blasting is to limit the peak particle velocity at the point of concern to 2 in/sec This limit has been suggested on the basis of field observations, but has recently been challenged and is now under review. The effect of blasting on anchored bulkheads has not been adequately evaluated to date.

Seed and Whitman (1970) made a thorough review of reported cases of failure of anchored bulkheads during earthquakes and concluded that increased lateral pressures due to earthquake effects should be considered in the design of anchored bulkheads. The degree of ground shaking that a bulkhead can withstand depends, to a considerable extent, on the margin of safety provided for static loading conditions. Different conventional design procedures provide significant variations in bending moments and anchor loads. If a wall is designed using one of the more conservative methods, it will have a greater ability to withstand seismic forces than a wall designed more economically by a less conservative method. Therefore, it is not possible to make general assessments of behavior of anchored bulkheads during earthquakes.

Special attention should be given to the strength and location of tie rods and anchor systems. These should be designed conservatively and, in seismic regions, the anchorage should be located further behind the wall than for non-seismic regions. Careful consideration should be given to the possibility of a reduction in strength of the soils, both behind and in front of the wall, due to buildup of pore water pressures or liquefaction (complete loss of strength). This is particularly critical in the passive pressure zones near the toe of the wall and near the anchorage. Loose backfills are particularly vulnerable to liquefaction.

Discussion

The problem of anchored bulkhead design has been defined by Terzaghi (1943) in the following simple terms: What is the depth to which the sheetpile must be driven to ensure adequate lateral support for the lower part of the bulkhead? What is the intensity of the force which acts on the anchor rods? What is the value of the greatest bending moment in the sheetpiles? These objectives can be met during design of an anchored bulkhead by employing one of the many available methods.

The conventional design methods (free-earth support, fixed-earth support, and modifications) provide a rather expedient way for designing anchored bulkheads. It appears that the free-earth support method with appropriate bending moment reductions and the fixed-earth, equivalent-beam, method yield results that are in agreement with field and laboratory experimental measurements. However, due to the different assumptions involved, large variations can be obtained in terms of overall wall height and cross-sectional structural characteristics as indicated by the comparison of design moments shown in Fig. 19. For example, shorter, but heavier, sheetpiles are required when the bulkhead is designed according to the free-earth support method than according to the fixed-earth method. To obtain a safe, and yet economical design, it is necessary that a given anchored bulkhead be designed by a number of different methods and the results be evaluated on both a safety and cost-effectiveness basis. The computer program that is available through the Corps of Engineers (Dawkins 1979) facilitates such a comparative study by allowing the design to be made according to five different conventional methods.

Design methods that are based on the plastic theory do not take into consideration deformations of the soil. The elasto-plastic method (use of subgrade reaction) can be considered as an improvement since deformations are included in the analysis by directly correlating the pressures exerted by the soil on the wall to the deformations of the wall. From a theoretical point of view, the elasto-plastic method appears to be more satisfactory than the older conventional methods, but a major problem still exists in the fact that the coefficients which are used for determining the elasto-plastic laws are very difficult to ascertain. Furthermore, the elasto-plastic method is only valid for soils which undergo primarily horizontal movements during excavation, such as sands and stiff clays. This method cannot account for significant vertical soil movements that may occur in clays which experience a tendency for basal heave.

The elasto-plastic method is better suited for use after a preliminary design has been made by use of the classical earth pressure methods. The preliminary design can be checked and refined against allowable wall deflections and allowable anchor loads for different construction stages. An application of this approach was documented by Gigan (1979) for the design and construction of the anchored sheetpile wall shown in Fig. 20. The wall consisted of Larsen VS, U-shaped piles, with the interlock on the neutral axis and IRP tiebacks were used. Several sheetpiles were instrumented by means of pressure cells, strain gages, and inclinometers, and load cells were fixed on several of the tiebacks. The sequence of construction was according to the four phases indicated in Fig. 21. Moments and displacements for each phase of construction were calculated, with the aid of a computer program, using coefficients of subgrade reaction for the various soil types. The coefficients of subgrade reaction were selected on the basis of pressuremeter tests. The observed wall behavior was in good agreement with the calculated behavior. The field measurements showed that, under stresses, the U sheetpiles slid within the locks indicating that the moment of single sheetpile, rather than sheetpile pair, should be used in the bending moment calculations.

There are disadvantages to using the earth pressure or the elasto-plastic methods of analysis because soil deformations at the free surface or within the soil mass cannot be computed. The only fully satisfactory analysis in this respect is one where a finite element approach is employed. A complete and accurate solution can be obtained by using basic concepts of mechanics when the stress-strain behavior of the soils involved and the limit conditions of the problem are correctly represented. Theoretically, therefore, the finite element approach is a powerful tool which can be used to design anchored bulkheads and evaluate their behavior. However, practical considerations limit the use of this method of analysis. The computer programs which are needed are very complicated and a large number of parameters describing soil nature and behavior must be determined. The soil information needed is almost never available in practical cases and its determination necessitates extensive investigations and testing which are far in excess of routine practice. Further technical difficulties arise from the choice of mesh, type of element, and method of representing soil behavior.

The quality and safety of the final design of an anchored bulkhead depends on (a) the accuracy of the information regarding site conditions, such as soil properties and water levels, (b) the accuracy of the particular design method or theory employed, (c) the correct evaluation of possible excess surcharge, overdredging, scour, etc, and (d) the adherence to adequate construction procedures developed from an understanding of their effects on the forces acting on the bulkhead. It must be emphasized that use of even the most elaborate and advanced design and computational methods does not reduce the requirement of assessing the effects of numerous factors which have a bearing on the final design.

As with other engineering structures, anchored bulkheads should perform satisfactorily for a long period of time. The performance of bulkheads should be monitored to assure that potentially hazardous situations are identified and corrected, and that catastrophic failures are prevented. The magnitude of the observational program should depend on the size, character, and importance of the bulkhead. In order to monitor settlement and lateral movements, survey reference points should be located at selected points on the anchored wall, the backfill, and the anchorages. Groundwater level fluctuations can be monitored through observation wells placed in the backfill. The installation and monitoring of reference points and observation wells are rather inexpensive and are recommended for practically all anchored bulkheads. Deflections of the wall along its height can be monitored using portable slope indicators and a small number of such installations should be provided for routine inspection of bulkheads to assure early detection of conditions that may adversely affect their stability. Strain gages on anchor rods, load cells on tiebacks, and pressure cells against the inside of the wall should be used to supplement the instrumentation when a bulkhead of extreme importance is constructed or when performance data are required for research purposes.

Conclusion

Design methods and construction and performance considerations for anchored bulkheads were reviewed and discussed in order to provide a useful guide to the practicing engineer. Although numerous methods have been developed, a uniform methodology has not been advanced for the design of anchored bulkheads and this requires a substantial degree of initiative and judgement on the part of the designer. The effects of repetitive and dynamic loads have not been adequately introduced in a comprehensive design method, although they can be detrimental to the safety and long-term performance of anchored bulkheads. Available construction and performance records indicate that use of an appropriate design method assures the safety of an anchored bulkhead only if correct construction procedures were employed and the long-term performance is properly monitored. In conclusion, the design, construction, and performance of anchored bulkheads is an engineering problem which requires the judicious combination of available design methods, expert construction specifications, and engineering judgment.

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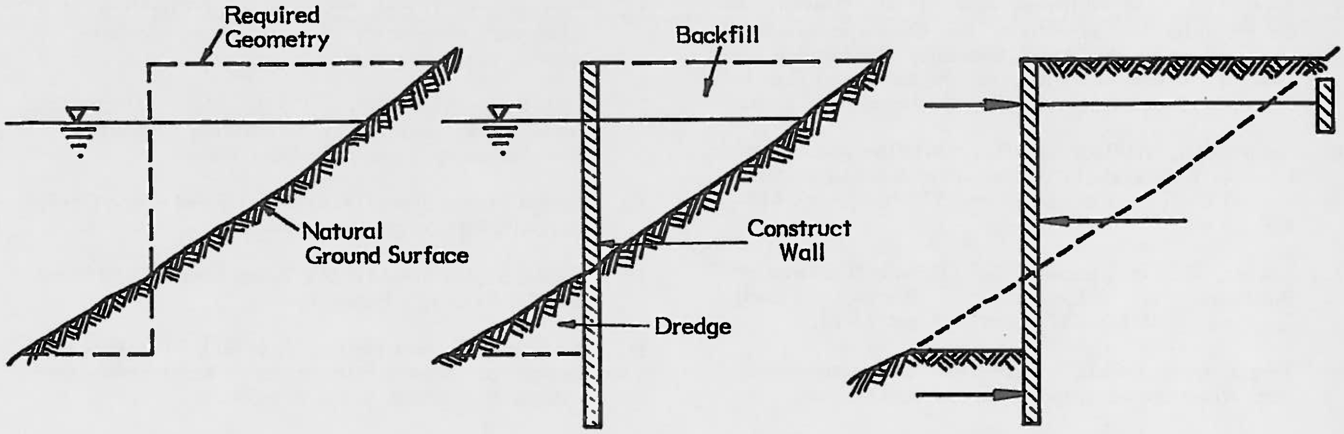
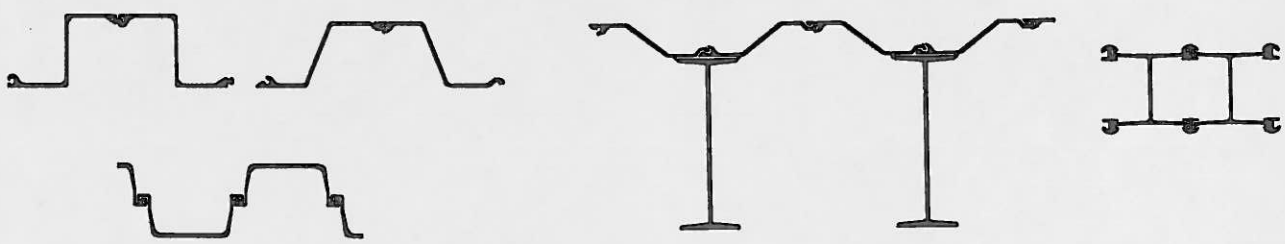


a. Problem

b. Solution

c. Wall should resist forces acting on its back; anchor and soil in front provide resistance

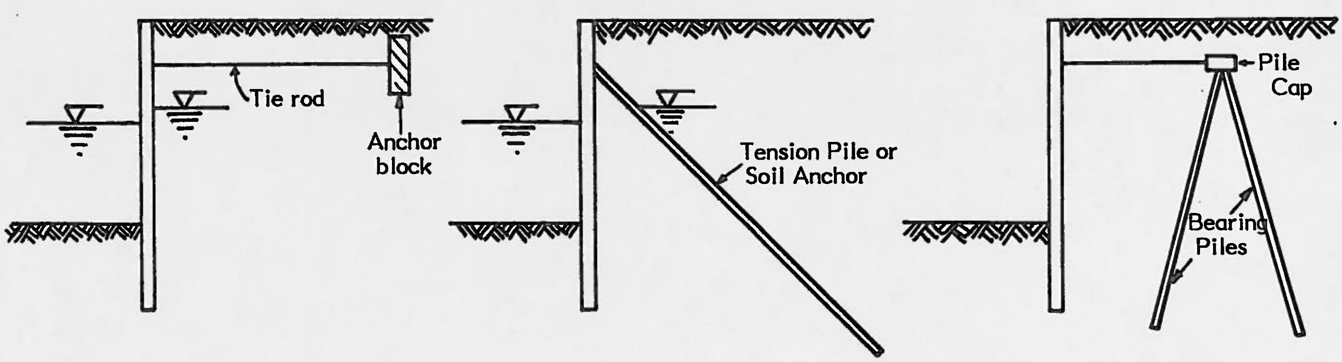
Figure 1. Working principle of anchored bulkhead



a. Z-sections and U-section

b. High modulus sections

Figure 2. Typical sheetpile sections (after Winterkorn and Fang 1975)



Deadman anchorage

Raking anchor

A-frame anchor

Figure 3. Typical anchorages

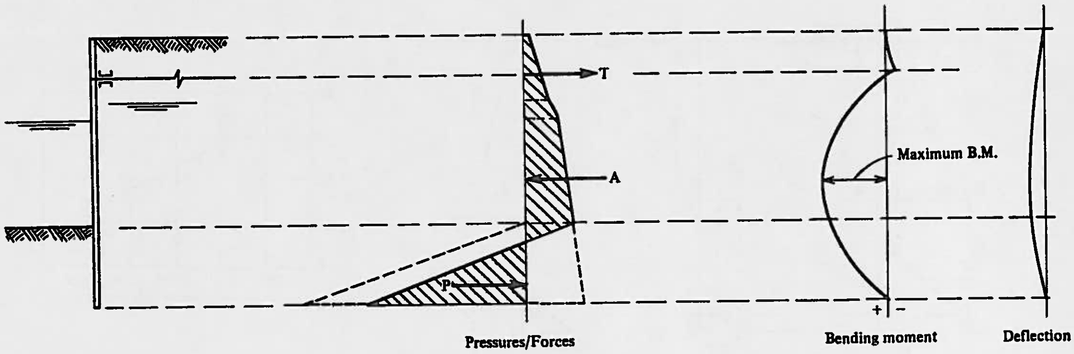
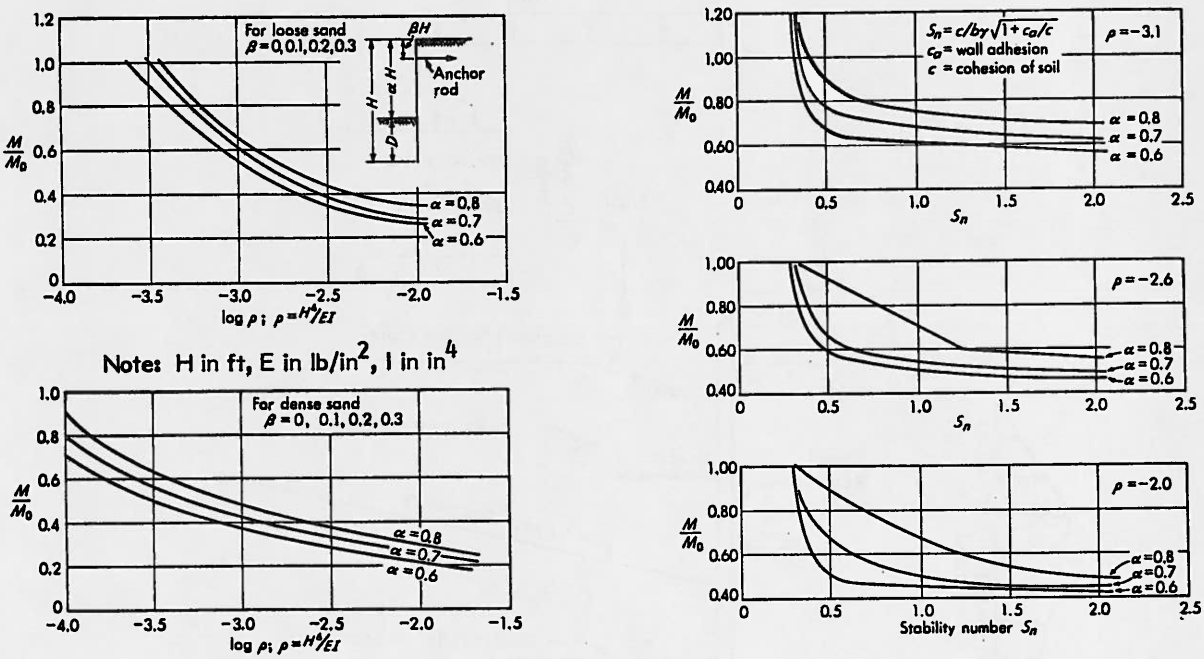


Figure 4. Free-earth support method (after Winterkorn and Fang 1975)



a. Sheetpiles in sand

b. Sheetpiles in clay

Figure 5. Rowe's moment reduction curves (after Bowles 1968)

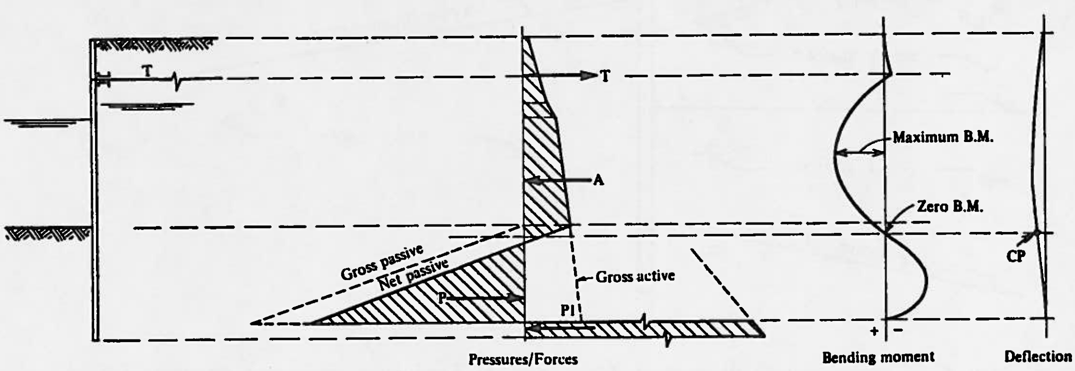


Figure 6. Fixed-earth support method (after Winterkorn and Fang 1975)

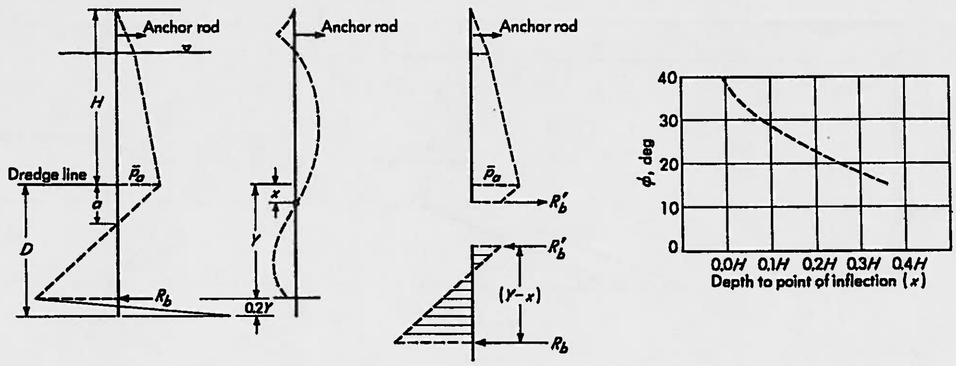


Figure 7. Equivalent beam method (after Bowles 1968)

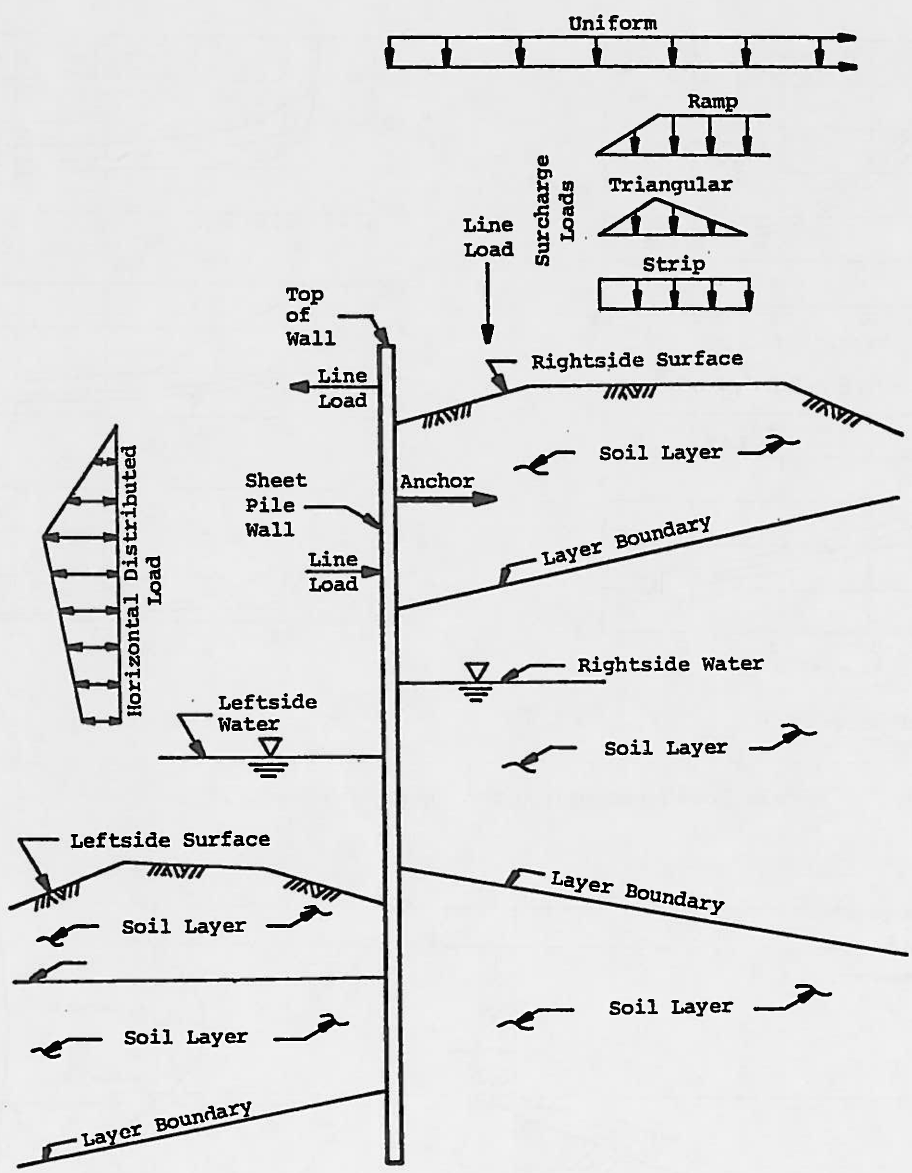


Figure 8. General wall/soil system for Corps of Engineers computer program (after Dawkins 1979)

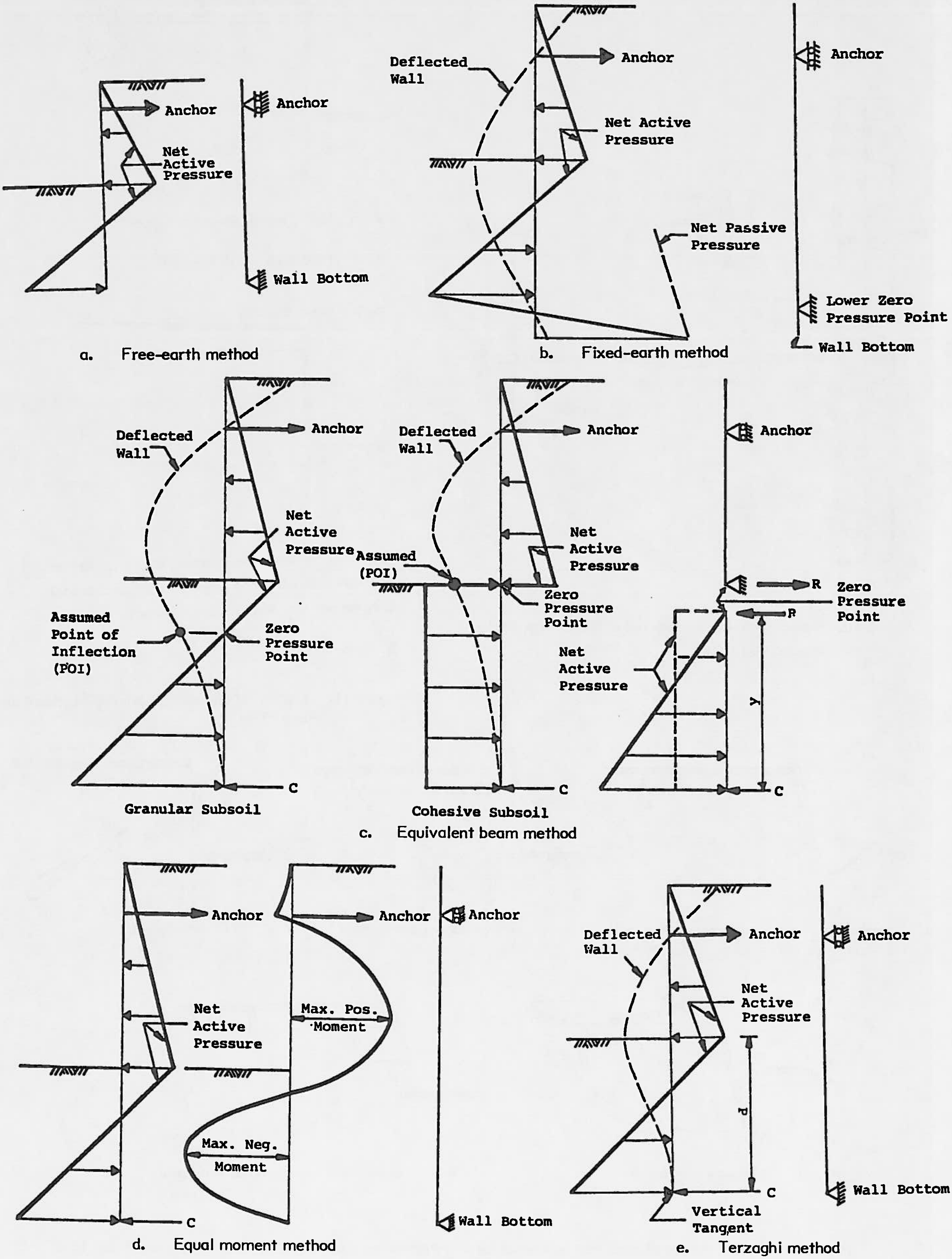


Figure 9. Design methods incorporated in Corps of Engineers computer program (after Dawkins 1979)

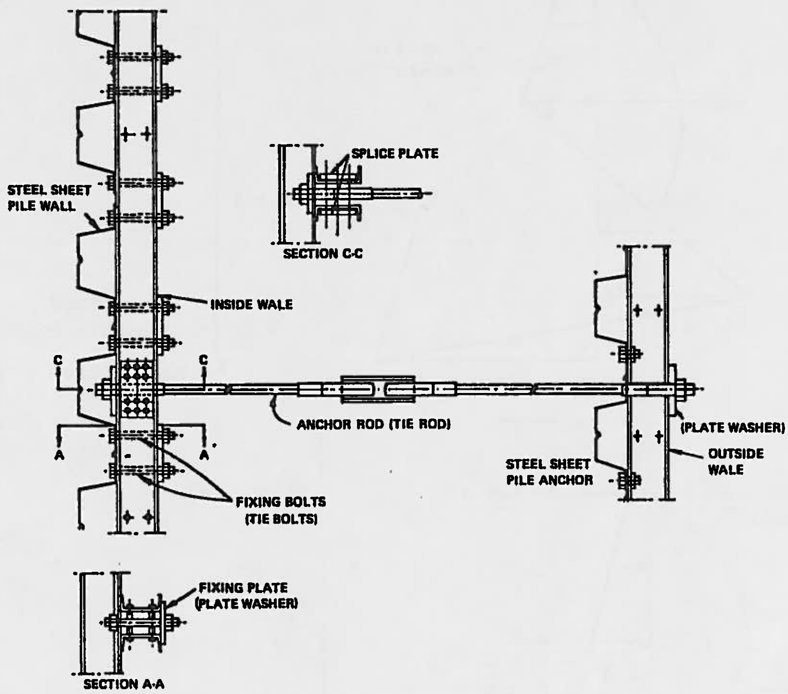
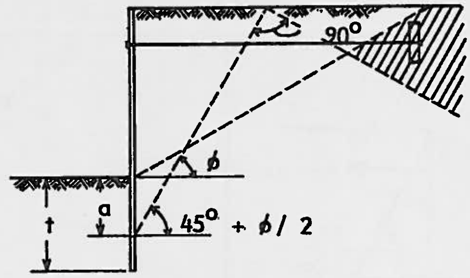


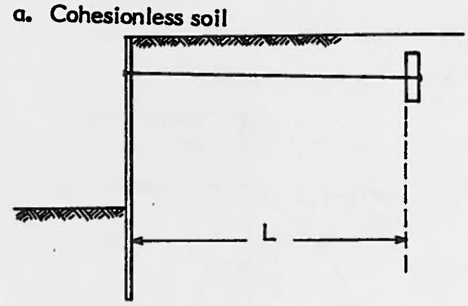
Figure 10. Typical wale and anchor rod details (after United States Steel 1974)



a = (2/3)t for fixed-earth support

a = t for free-earth support

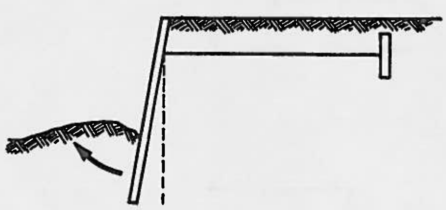
a. Cohesionless soil



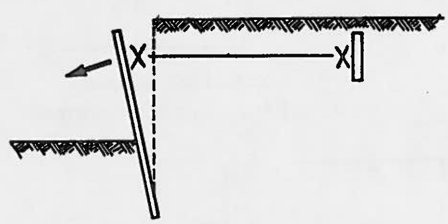
Total horizontal resistance over L to be not less than ultimate resistance of anchorage; L to be not less than total pipe length

b. Cohesive soil

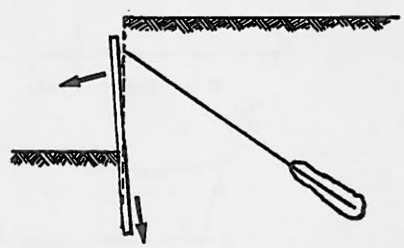
Figure 11. Location of anchorage (after Winterkorn and Fang 1975)



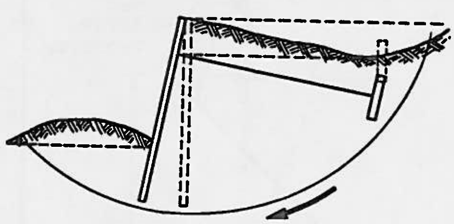
a. Bottom of sheetpiles moves outward



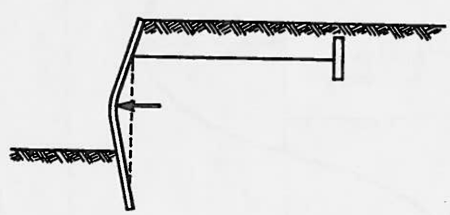
b. Failure of anchorage system



c. Penetration failure



d. Circular slip failure



e. Excessive deformation of sheetpiles

Figure 12. Failure modes of anchored bulkheads (after (Winterkorn and Fang 1975 and Browns and Stille 1976)

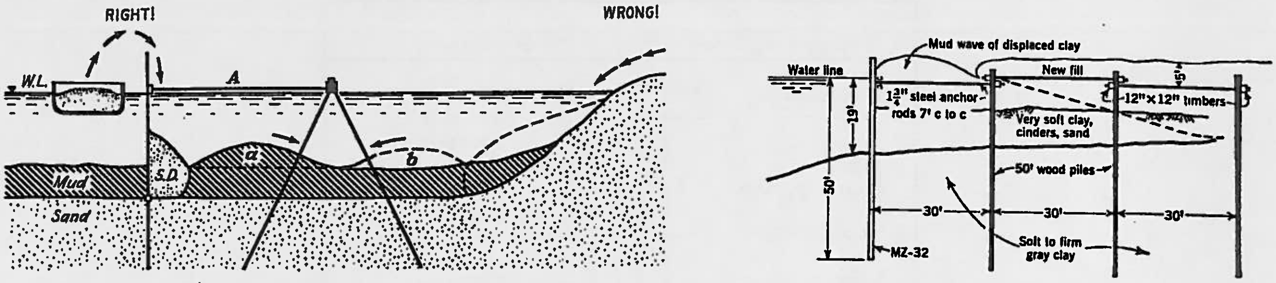


Figure 13. Effect of backfill placement; (a) after Tschebotarioff 1973, (b) after Sowers and Sowers 1967

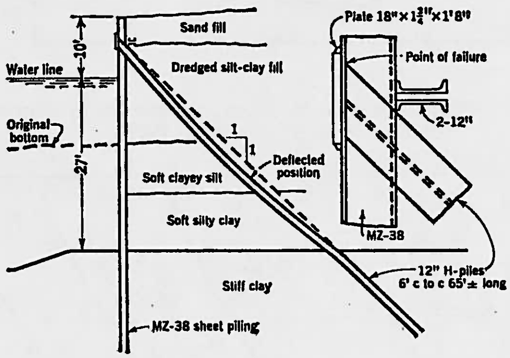


Figure 14. Effect of settlement (after Sowers and Sowers 1967)

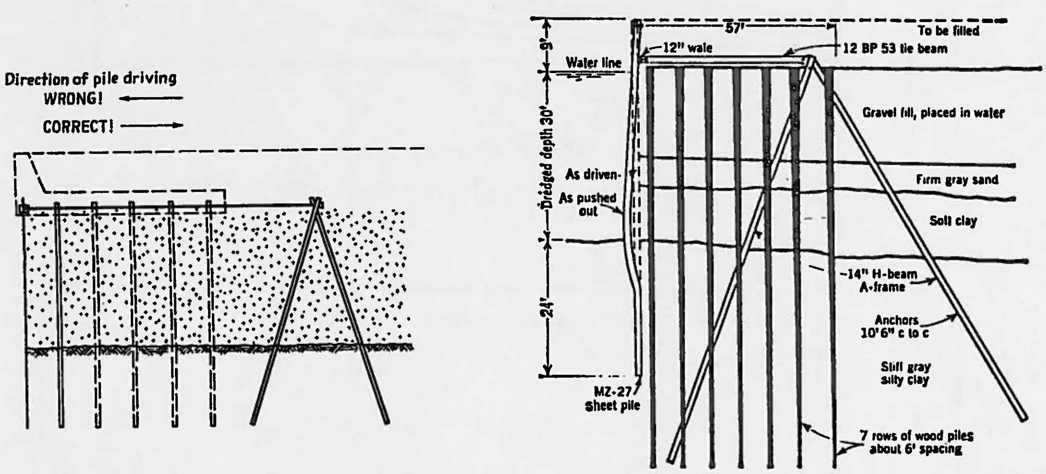


Figure 15. Effect of pile driving; (a) after Tschebotarioff 1973, (b) after Sowers and Sowers 1967

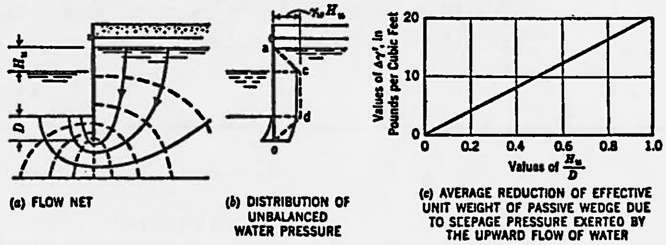


Figure 16. Effects of unbalanced water pressure (after Terzaghi 1954)

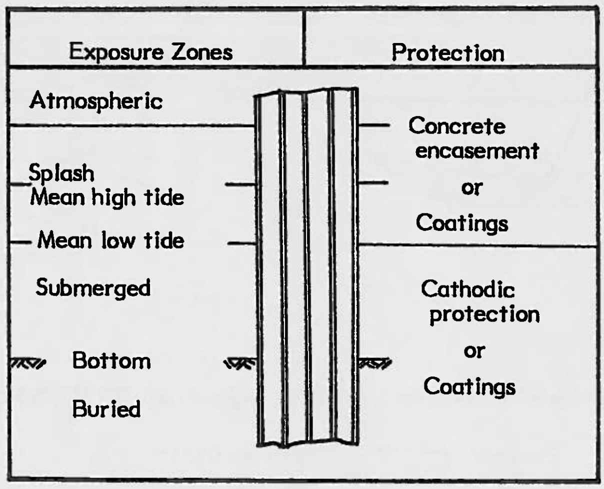


Figure 17. Sheetpile exposure zones in seawater environment (after United States Steel 1976)

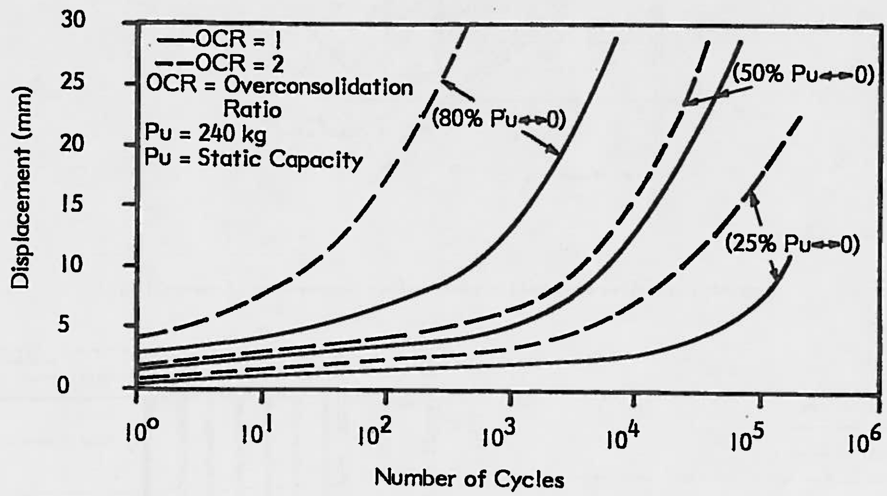


Figure 18. Effects of repetitive loading on plate anchors (after Hanna et al 1978)

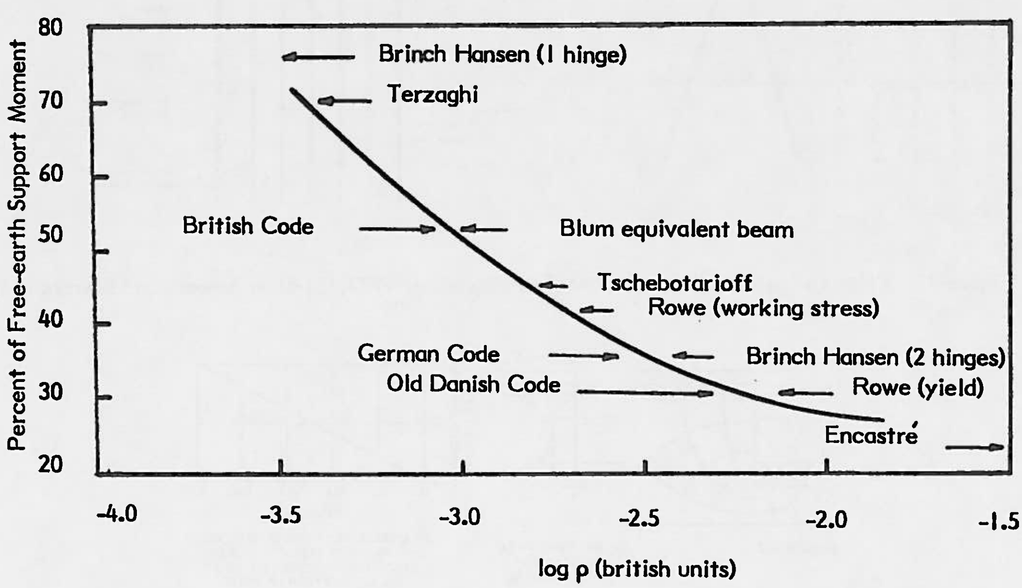


Figure 19. Maximum bending moments obtained by different methods (after Rowe 1972 and Smith and Boorman 1974)

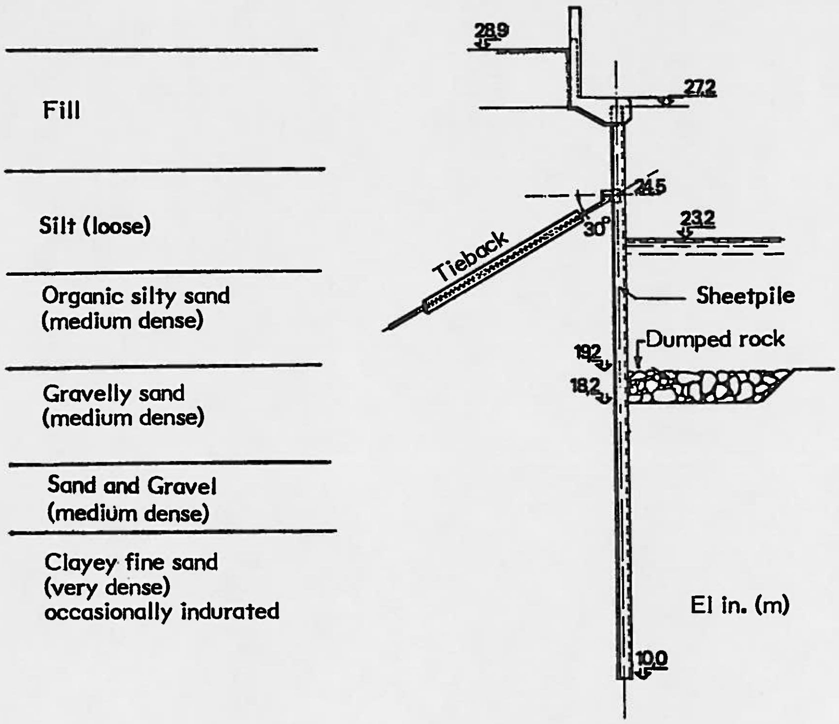
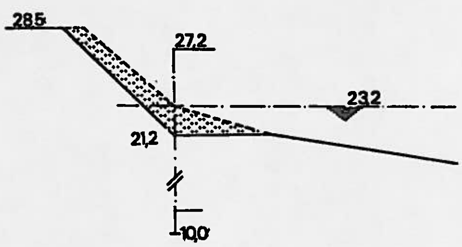
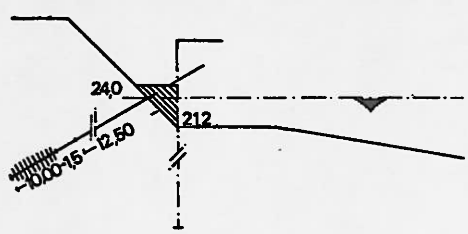


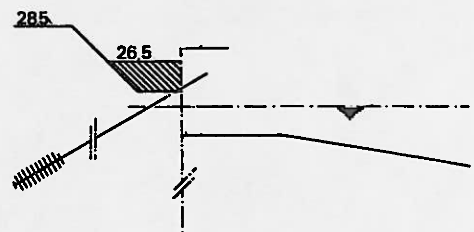
Figure 20. Wall cross-section and soil profile (after Gigan 1979)



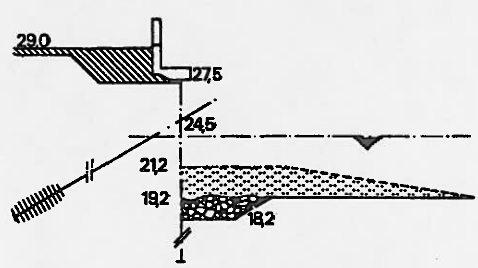
Phase 1: Predredging and sheetpile driving



Phase 2: Partial filling and partial stressing of tiebacks



Phase 3: Further filling and final stressing of tiebacks



Phase 4: Final filling and dredging

Figure 21. Construction phases (after Gigan 1979)

LARGE BURIED METAL CULVERT DESIGN AND CONSTRUCTION

by

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Abstract. Corrugated-metal buried structures, known as long-span structures, have increased in size to where structures with spans up to 51 ft have been successfully built. Their load-carrying capability results from an efficient use of soil-structure interaction. This paper describes the configurations of these structures, the basic soil-structure interaction concepts involved, the criteria for design, the construction considerations, and analytical modeling requirements. Then examples of performance and analytical results are given. The paper concludes with a summary of some of the experience gained to date.

Introduction

Long-span, corrugated-metal buried structures are large conduits constructed of structural-plate with spans exceeding 15 to 25 ft or radii of curvature exceeding 8 to 12 ft. They are also known as flexible structures because they have relatively low bending stiffness. The principal applications for long-span structures are culverts and grade separations, with many installations serving as alternatives for small bridges. Over 600 structures of this form have been installed in North America since 1960.

A comprehensive survey of the design and construction state-of-the-art of these long-span structures was recently published (Ref. 1). A few field instrumentation studies have also been performed to investigate the behavior of these structures during construction. These are described in Refs. 2, 3 and 4, except for a more recent project that has not yet been reported.

Analysis of the field performance by finite element computer models is providing insights into the soil-structure interaction. An example is given in Ref. 5. The finite element method has recently been applied to the design of these structures (Refs. 6, 7) with the intent of supplementing the existing empirical methods developed by AASHTO (Ref. 8).

This paper will first provide a description of the long-span design configurations and the basic concepts of the soil-structure interaction. The paper will then review the design and construction of long-span structures and give examples of observed performance. Further details may be found in the cited references.

Structure Configurations

Several of the structures involved in the referenced instrumentation studies are shown in Figs. 1-3. These were all constructed for road crossings over streams as an alternative to, or replacement for, conventional bridges.

A typical installation for a pipe culvert is shown in Fig. 4. To minimize the likelihood of inadequate soil properties, an envelope of good granular (sands and gravels) backfill is placed around the structure. Such materials are easy to compact to an adequate strength and stiffness and they are very stable over the life of the structure. At present, the size of this envelope is empirically established, because rational design criteria have not been defined. When rock is present as the natural ground, an arch culvert with footings on rock may be used instead of a complete pipe with the invert resting on bedding soil.

Common shapes for long-span structures are shown in Fig. 5. However, special features are added to the basic shapes to aid in construction or load-carrying ability because the available corrugated plates are not considered adequate for large structures. Examples of these special features are given in Figs. 6-9.

Among the reasons why these structures are economical alternatives to conventional bridges are that: 1) they can be constructed without skilled labor, given proper supervision, 2) they can be erected rapidly, 3) they are relatively easy to deliver and install at difficult-to-reach sites, and 4) they can be used where foundation soil conditions are poor.

Basic Concepts

Some of the basic terminology and parameters are shown in Fig. 10. The load on the structure is caused by the weight of the soil above it, together with any surface load, which may be distributed as shown (P_o) or may be concentrated. The vertical soil pressure at the crown elevation in the absence of the structure is called the free-field pressure or stress (P_v). It will be equal to the surface pressure plus the geostatic stress. The soil pressure on the crown will generally be different from P_v because the structure deforms differently than

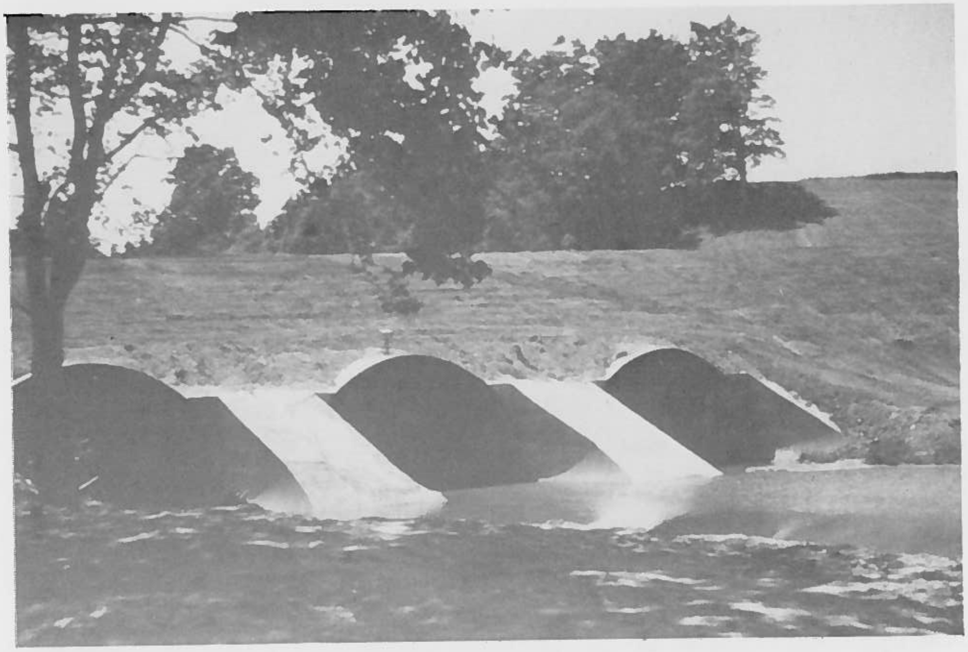

Figure 1. Three Armco Long-Span Horizontal Ellipses with 27-ft Rise, 37-ft Span, and 29-ft Height of Cover

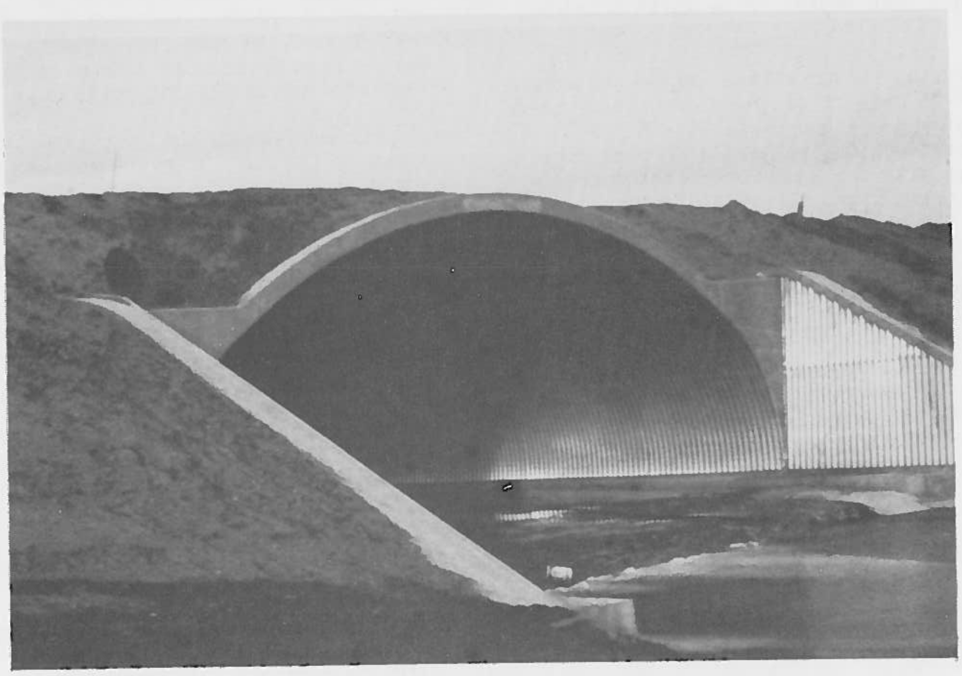

Figure 2. Armco Long-Span Arch with 18-ft Rise, 50-ft Span, 5-ft Height of Cover

Figure 3. Republic Steel Long-Span Low-Profile Arch with 16-ft Rise, 38-ft Span, and 11-ft Height of Cover

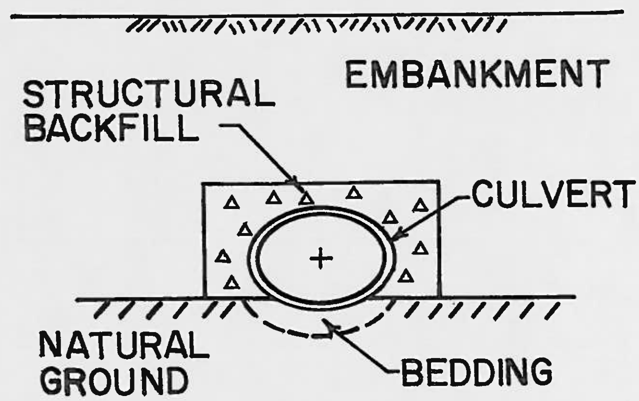

Figure 4. Components of Typical Installation

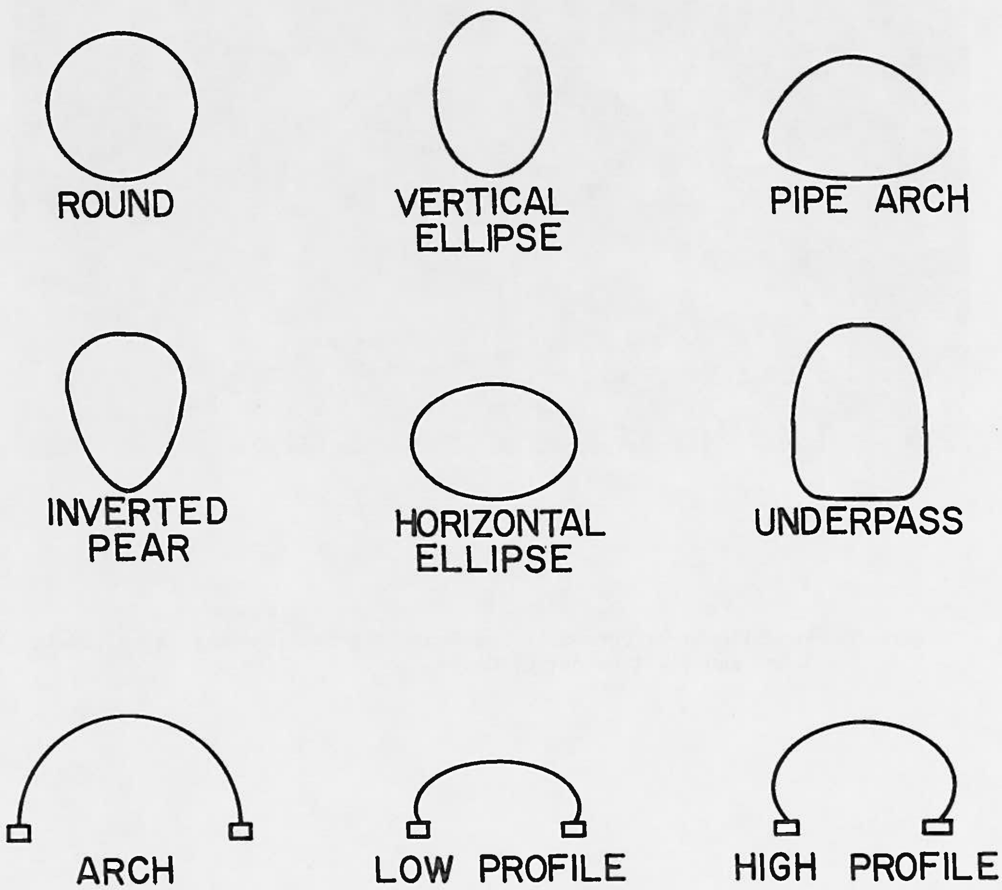

Figure 5. Common Long-Span Structure Shapes

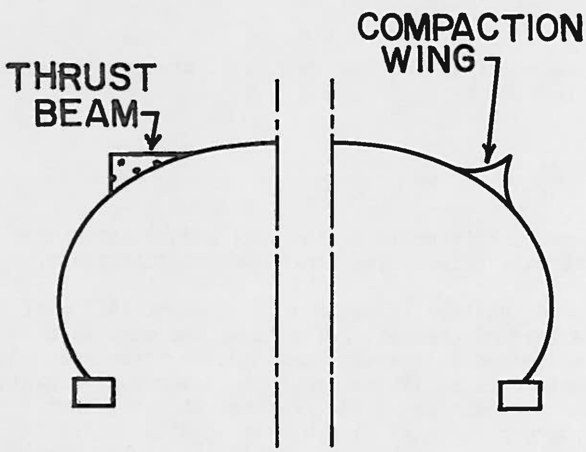

Figure 6. Thrust Beam and Compaction Wing

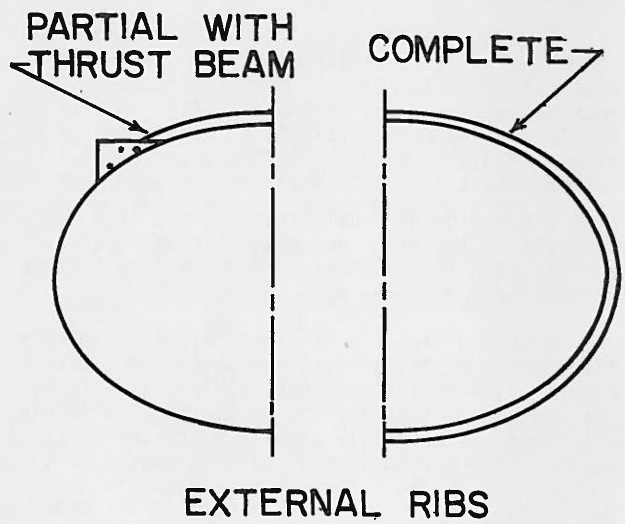

Figure 7. External Rib Stiffener

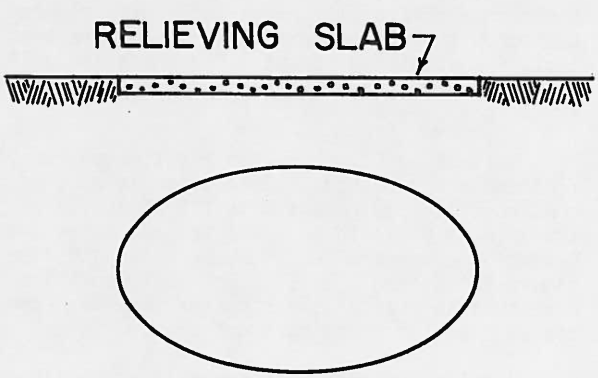

Figure 8. Relieving Slab for Shallow Cover

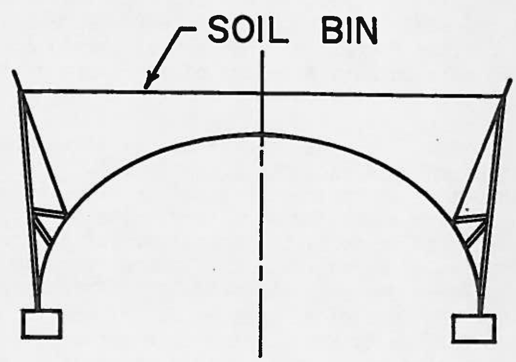

Figure 9. Soil Bin on Top of Structure

the soil. Likewise, the horizontal soil pressure at springline, P_{se}, will be different from $k_o P_{vs}$, where k_o is the coefficient of earth pressure at rest and P_{vs} is the free field vertical stress at the springline elevation.

To understand the soil-structure interaction concepts involved, it is helpful to start with the equilibrium of forces in a simple structure. A four-member structure is shown in Fig. 11 with pinned joints and with vertical and horizontal reactions. Application of basic mechanics shows that not only must $V_1 = V_2 = V$ and $H_1 = H_2 = H$, but also that $V/H = a/b$ for equilibrium. Thus, if $a = b$, then all of the reactions must be equal. Also, if $b = 3a$, then the horizontal reaction H is 3 times the vertical reaction V.

A buried flexible structure may be considered like a many-member structure with pinned joints. An extension of the concepts in Fig. 11 leads to the conclusion that a circular flexible structure must have approximately a uniform pressure distribution. Also, a horizontal ellipse must have a much larger horizontal pressure at the sides (springline) than vertical pressure at the crown.

A better method to estimate soil pressure distribution on the structure can be derived from the equilibrium considerations of a thin curved plate shown in Fig. 12, which neglects bending moments and surface shear. For a given value of thrust, T, the pressure on the plate, p, is inversely proportional to the radius of curvature, r. This relationship verifies the above comparison of circular and elliptical shapes (Fig. 13). For structures such as the horizontal ellipse, where the springline radius of curvature is much greater than the crown radius, large horizontal passive pressure may be needed to satisfy equilibrium. The soil strength must be adequate for this purpose. However, even without failing the soil will yield in the process of developing the passive resistance. This will cause the radius at the springline to decrease, hence further increasing the lateral pressure and causing more outward movement of the sides of the structure.

A simplified version of the soil-structure interaction concept is shown schematically in Fig. 14. The thrust is approximately equal to half the weight of soil prism above the structure. The springline pressure must be equal to thrust divided by springline radius of curvature. The lateral force may be roughly represented by the product of lateral pressure and the rise of structure, R. If the lateral soil resistance is represented as a spring of stiffness, k_s, then the lateral displacement is $d_s = \frac{P_s}{k_s}$. Experience has shown that thrust can be roughly estimated in the manner shown in Fig. 13. However, the final shape, including effects of lateral deformation, cannot. This requires an appropriate soil-structure interaction model. If the shape is controlled during construction to approximately the design dimensions, then the pressure, P_s, can also be reasonably estimated in the manner shown. However, the problem of estimating the changes in d_s during backfilling still remains because the value of k_s has a high degree of uncertainty.

Arching is the term which refers to the amount of soil weight (and surface pressure) directly above the structure that is carried by the structure. Arching, defined by soil pressure at the crown, is

$$A_c = 1 - \frac{P_{cr}}{P_v} . \qquad (1)$$

In terms of springline thrust, T, arching is usually defined by

$$A_t = 1 - \frac{T}{\gamma HS/2} . \qquad (2)$$

However, this neglects the soil weight above the structure between the crown and the springline.

A positive value of arching means that part of the load of the soil prism above the structure is transferred to the soil next to the structure. A negative value implies just the reverse. However, both analysis and field measurements show that a structure which is flexible in bending, but stiff in ring compression, may simultaneously have a positive value of A_c and a negative value of A_t.

Design Criteria

The design objectives of the structure are two-fold. First, the structure must be stiff enough so that during construction the shape can be maintained and the backfill can be appropriately compacted near the structure. Second, when the completed structure is in service, it must accept its share of deadload and liveload with an adequate factor of safety against failure by 1) excessive deflection, 2) wall yielding, 3) wall buckling, and 4) bolted seam failure.

By neglecting bending moments and arching transfer of soil load, the thrust can be calculated by the method of Fig. 14. However, finite element methods with a computer can provide a more precise basis for obtaining thrust. The check for wall yield and seam failure is straight-forward after thrust has been determined.

Deflection is not reliably estimated except by finite element methods. Deflection is handled as a construction requirement with the objective of achieving a final shape equal to the design shape. Deviations from the design shape (span and rise) should not exceed 1 to 2%. The soil conditions must also be stable after the structure has been completed so that the shape does not change.

Buckling criteria are not well established and are seldom used for design of these structures. The development of plastic hinges in the metal structure does not infer buckling. Normally, the soil support is considered sufficient to give an adequate factor of safety against buckling collapse if the other design criteria are satisfied. However, for shallow soil cover with significant concentrated live loads from vehicles, buckling collapse is the most likely mode of failure. More research on this criteria is needed.

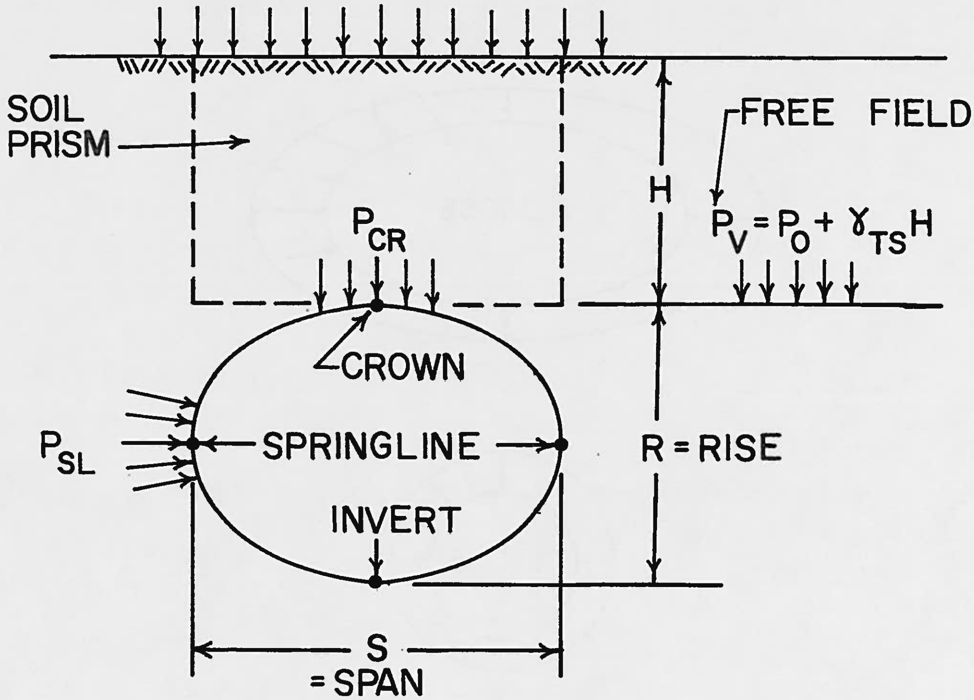

Figure 10. Basic Terminology and Parameters

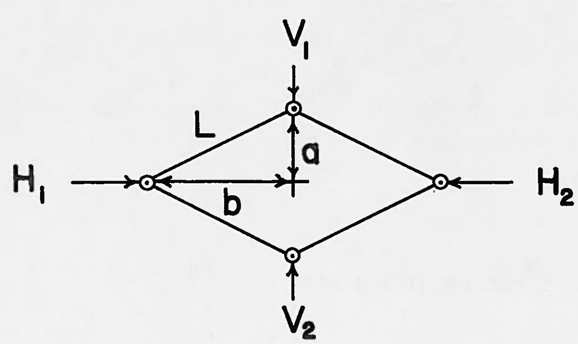

Figure 11. Four-Member Pinned Structure

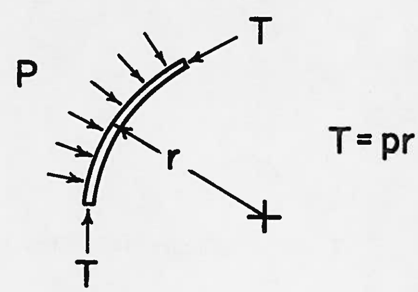

Figure 12. Relationship of Pressure to Thrust

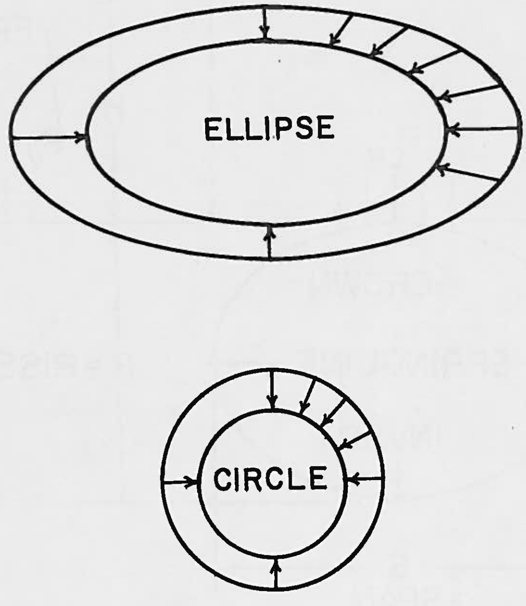

Figure 13. Pressure Distribution on Flexible Structures

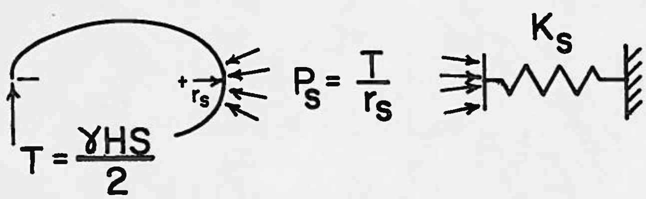

Figure 14. Simplified Version of Soil-Structure Interaction

Construction Considerations

One essential ingredient for successful long-span structures is proper construction. The placement and compaction of the backfill soil is carried out in a manner that will control the shape of the structure. But, in addition, the construction equipment must not create live load conditions that will buckle the structure.

The required amount of soil compaction is specified in an attempt to insure adequate soil properties. The normal highway approach of specifying a minimum end result density is typically used. It should be recognized, though, that this is not equivalent to specifying a desired minimum soil stiffness and strength, which would be more relevant.

Various stages of construction of long-span structures are illustrated in Figs. 15-19.

Analytical Modeling

Soil-structure interaction modeling using finite element methods is the best approach for design and analysis of long-span structures. The important features of the problem which can be handled by this approach are: 1) incremental construction, including soil compaction effects, 2) live load and dead load representation, 3) the geometry of the structure, 4) the soil boundary conditions, and 5) the material properties, including non-linear, stress-dependent soil behavior and seam slip in the structure. The output of the finite element model can be used directly to establish the degree of compliance with the design criteria presented earlier.

The simplest representation of the soil behavior for the finite element method is a linear-elastic model defined by a constant Young's modulus (E) and Poisson's ratio (ν). Poisson's ratio is usually assumed from past experience. Young's modulus is frequently estimated from reported values for similar conditions. However, it can be obtained from triaxial tests or indirectly from uniaxial strain tests (such as in an oedometer).

It is well known, however, that soil is a non-linear material whose stiffness is a function of stress conditions. More specifically, stiffness increases with mean normal stress level and it decreases with increasing shear stress, eventually reaching a failure state. Several hyperbolic formulations for either Young's modulus or shear modulus have been developed which represent this behavior (Refs. 9, 10).

An alternative soil model that has also frequently been used for culvert analysis is the overburden-dependent model. This model essentially determines Young's modulus by assuming that the soil elements around the culvert are in the same state of stress as in the uniaxial strain test. Young's modulus from the uniaxial strain test can be related to the axial stress in this test by assuming a value for Poisson's ratio. In one version of this approach, for example, the value of Young's modulus for a particular element is determined by substituting the vertical stress in the element for the axial stress in the test. In another version, the maximum principal stress in the element is substituted for the axial stress. The disadvantage of this model is that it includes no failure conditions.

Experience with these alternative approaches for the soil model indicates that the hyperbolic model is the preferred approach when an incremental solution is carried out. The overburden-dependent model appears to have no advantage and it is more restrictive in its applicability. The linear-elastic model should only be used for approximate analyses, particularly when the incremental approach is omitted to obtain preliminary design estimates at minimum computer cost.

Examples of the comparison of computed and measured behavior are given in Refs. 5 and 6. A more recent project (Fig. 3) provides further confirmation of the value of this approach. The significance of this last study is that, for the first time, soil samples were obtained and laboratory tested to get the soil properties for the computer predictions.

Conclusions from Research

A number of conclusions can be drawn from the review of past practice as well as the experience gained from field case studies and finite element analysis. These are:

1. In most cases, simple design methods have been adequate for long-span structures in the past. The success has been due to the effective use of soil-structure interaction combined with attention to shape control during construction and use of good backfill materials.

2. At the time backfilling begins, the crown of the structure, if unsupported, will drop below the design height because the weight of the structure is resisted only by its bending stiffness. As backfill is placed in layers, the sides are pushed in and the crown is pushed up. In order to achieve the proper final shape, when the backfill reaches the quarter point, the crown should be above the design height by the amount which the crown will be pushed back down as the remaining fill is placed. Experienced judgment is required in the field to achieve this goal.

3. The simple design methods in use cannot provide accurate information on the expected displacements. The numerical finite element method is needed to make reliable predictions. However, even though experience to date with this method is very encouraging, the uncertainty of the values of the soil parameters and the significant influence of construction procedures on the results still leaves uncertain the accuracy of deformation predictions. Nevertheless, the use of the finite element method for assessing alternative situations should be reliable.

4. Arching is a term applied to the soil load redistribution associated with buried structures. However, the mechanisms of arching have frequently been confused in the past by the use of simplified models of soil-structure interaction which do not represent the mechanics of the problem. The availability of the finite element models now eliminates the need to use inaccurate models. One of the findings from the finite element analyses is that, even though the metal culverts are very flexible in bending, their high stiffness in ring compression causes arching to be negative in terms of the thrust stress

Figure 15. Shaping Soil Bedding for Invert Plates using Template

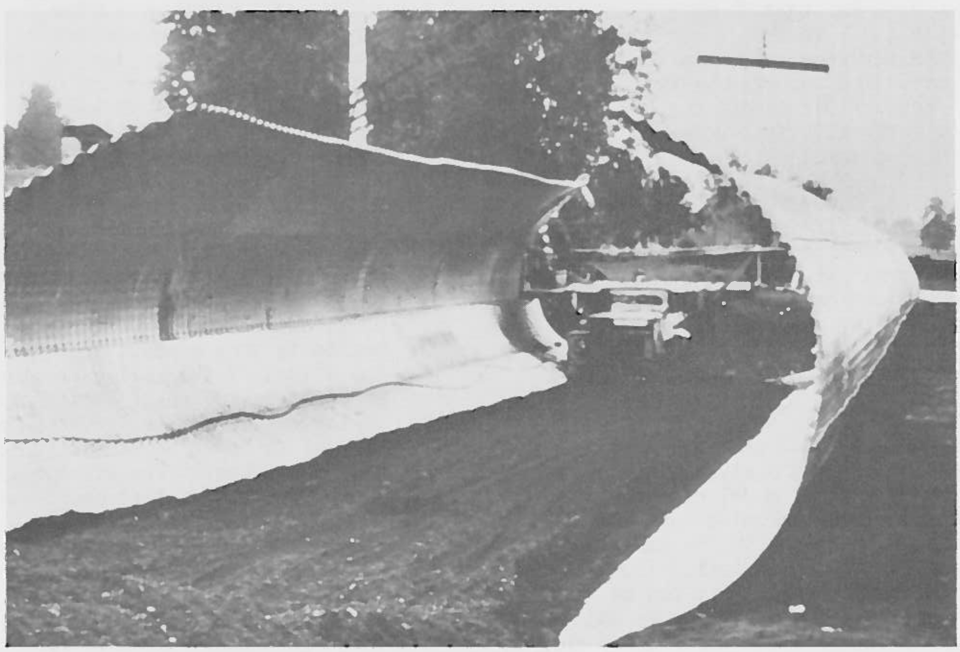

Figure 16. Assembly of Plates Prior to Completing Crown

Figure 17. Compacting a Layer of Structural Backfill

Figure 18. Constructing Concrete End Wall

Figure 19. Completed Horizontal Ellipse with Relieving Slab Undergoing Live Load Testing (16-ft Rise, 29-ft Span and 3½-ft Cover

developed in the structure. The structure can be expected to carry perhaps 0 to 40% more soil load than the weight of the prism of soil above the crown. The error would be less if the weight of soil between the crown and springline were included in the calculation with the ring compression theory.

5. Design based on thrust calculated from the soil load is acceptable only when live load forces are much smaller. Traditional methods of handling live loads are not conservative with respect to buckling. Improved results can be obtained using equivalent live loads with the two-dimensional finite element method. However, buckling behavior apparently is not included in the finite element models. Thus further study of live load design and buckling behavior is needed.

6. Because the loads on the flexible structures are intended to be carried by ring compression in the structure, the magnitude of the bending stresses is probably not a concern with ductile metal as long as the shape of the structure remains stable and concave inward under the applied pressures. If the shape of the structure is close to the formed shape of the plates, the bending moments will be small and the soil pressure distribution will be approximately inversely proportional to the radius of curvature. This fact is useful in assessing the supporting requirements of the surrounding soil.

7. An envelope of good quality soil around the structure is required. Thus a structural backfill, such as sand or gravel, compacted to at least 90% T-180 density is usually specified. However, more study is needed into such questions as how much structural backfill is needed and how do the properties of the rest of the fill and the existing soil effect the structural fill requirements. The use of finite element models can help assess these factors.

8. The successful performance of the buried culverts depends on the soil behavior. Thus, reliable prediction of performance requires proper modeling of the soil stress-strain properties. Recent experience in soil stress-strain modeling indicates that the minimum suitable requirements are provided by a bilinear or hyperbolic relationship, including a failure state based on triaxial tests at constant confining pressure. Because soil property tests of this type are not likely to be used for design on a routine basis, it is important that the parameters in the soil model be estimatable from available information. This is best accomplished through use of a soil model with physically meaningful parameters and with extensive prior application experience.

9. The application of long-span structures to high fills is worthy of study. If the ring stiffness can be reduced by controlled horizontal seam slip to induce positive arching, the thrust stresses may be kept within limits allowed by available corrugated plate, while permitting much greater height of soil cover than is presently possible. Another area needing investigation is long-term stability as influenced by changes in soil properties. For example, effects of moisture change, creep and consolidation after construction can be detrimental. The extent of these factors varies with soil type. A better understanding of their role in performance of buried structures may permit use of a broader range of soil type in construction of long-span structures.

10. Long-span buried structures represent an excellent example of the best use of soil-structure interaction. The metal is used very efficiently by carrying the soil load in ring compression. Future design with the aid of finite element models may be expected to result in further improvements in economy and efficiency.

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DESIGN, CONSTRUCTION AND PERFORMANCE OF A CELLULAR COFFERDAM IN DEEP WATER

by

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Abstract. A description is provided of a permanent steel, sheet pile cellular cofferdam designed to accommodate a maximum water depth of approximately 80 ft. (24.4 m). The cofferdam is among the first to be constructed in the United States which, because of expected interlock tensions, required the use of high strength steel sheet piles and extruded wye connections. Vibratory probe compaction of the cell and connecting arc fill was performed to prohibit earthquake liquefaction, to minimize settlements and to obtain sufficient weight for cofferdam stability.

The cofferdam was instrumented with vibratory wire strain gauges to monitor interlock tensions in two cells and inclinometers to measure sheet pile deflections in five cells. Optical survey measurements were also made to measure cell movements. Performance data are presented and discussed for conditions after cell and arc filling, after fill densification and both during and after cofferdam basin unwatering.

Introduction

Extensive waterfront and shore based facilities are being constructed at the Naval Submarine Base, Bangor, Bremerton, Washington in conneciton with development of a new submarine refit and crew training facility for the Trident class submarine. The development includes an off-shore graving drydock. In order to build the drydock at the required offshore location, it was necessary that a cofferdam be designed to accommodate a maximum water depth of approximately 80 ft (24.4 m) and be suitable to be incorporated into the final drydock configuration. The cofferdam is among the first to be constructed in the United States which, because of expected interlock tensions, required the use of high strength steel sheet piles and extruded wye connections. The cofferdam was unique in many respects and its successful construction represents an advancement in the state of the art for cofferdam design and construction.

The purpose of this paper is to:

- Provide a summary of the cofferdam design and the construction procedures and sequences.
- Provide and discuss instrumentation data on cofferdam performance obtained during construction.

Facility Description

The drydock was located offshore, as shown in Figure 1, in order to permit the continued shallow water migration of salmon fingerlings. The drydock and two immediately adjacent pile supported piers form the three-sided Trident Refit Delta.

Refit Pier No. 1 and the South Trestle were constructed initially. The cofferdam was then constructed concurrently with Refit Pier No. 2. The drydock was constructed immediately following completion of the cofferdam. This scheduling accelerated the construction but imposed severe constraints on the cofferdam design by limiting the area available for positioning the structure relative to the future drydock and the existing pier. Additionally, the offshore location resulted in very deep cofferdam water depths.

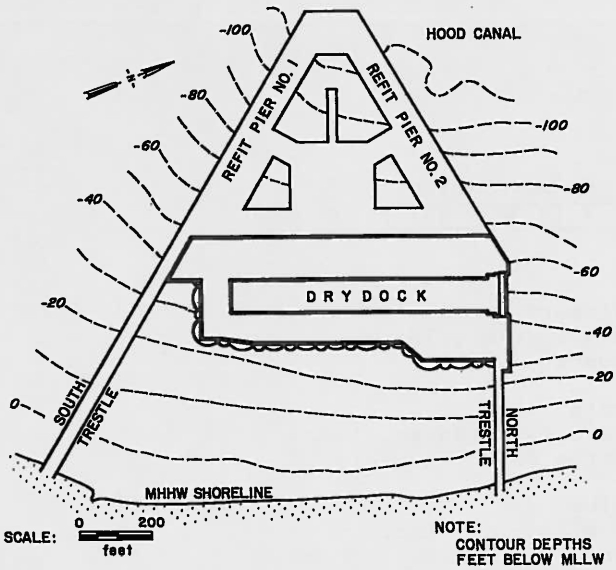

Figure 1 Location Plan (1 ft = 0.305 m)

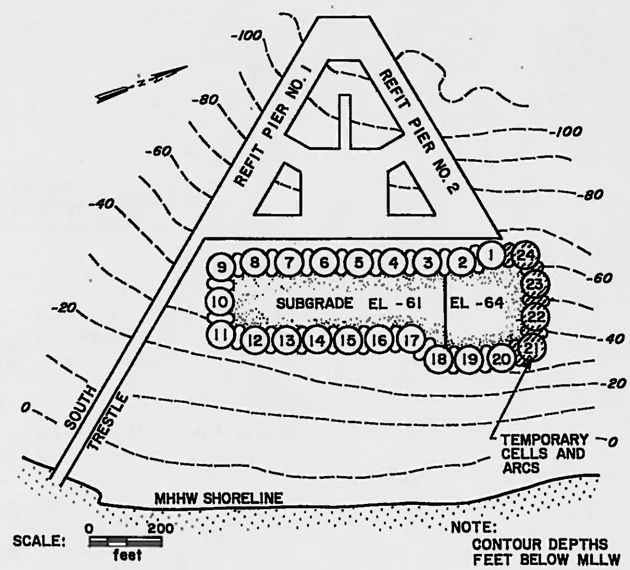

Figure 2 Cofferdam Plan (1 ft = 0.305 m)

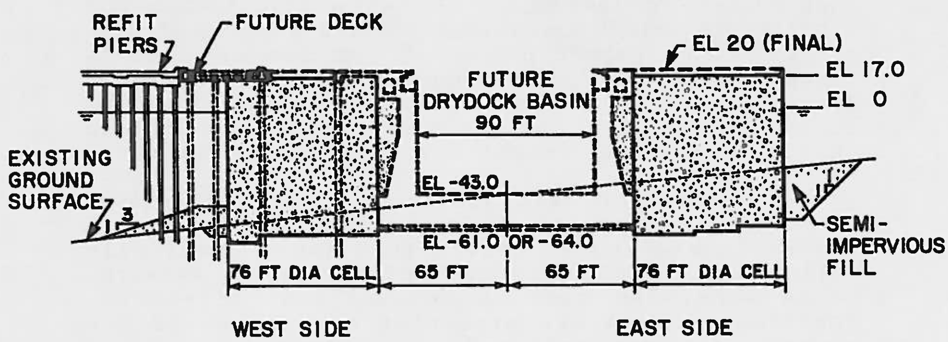

Figure 3 Typical Section (1 ft = 0.305 m)

Figure 4 Aerial Photograph of Unwatered Cofferdam

A plan and typical section of the cofferdam are shown in Figure 2 and Figure 3, respectively. Figure 4 is an aerial photograph of the structure after unwatering.

The cofferdam fill surface was at El. 17 ft (6.2m) during construction to provide freeboard against EHW at El. 14.6 ft (4.5 m). Project datum is El. 0 at MLLW. The top of the drydock floor is at El. -43 ft (-13.1 m) which is 49.4 ft (15.1 m) below MTL. The gravity design of the drydock required a floor thickness of from 16 ft (4.9 m) to 19 ft (5.8 m). This thickness, when combined with an 18-in. (0.46 m) gravel subbase and 6-in. (0.15 m) concrete working mat constructed below the floor, resulted in a final excavation subgrade of El. -61 (-18.6 m) to El. -64 (-19.5 m). The maximum depth is, therefore, 78.6 ft (24.0 m) below EHW.

The cofferdam basin is approximately 774 ft (236 m) long and has a minimum width of 130 ft (40 m). The cofferdam consists of 24 circular cells, each 75.83 ft (23.11 m) in diameter, and 24 connecting arcs. The cells are spaced from 83.55 ft (25.47 m) to 93.57 ft (28.52 m) on center to obtain the required geometry to achieve closure. Cells 1 through 20 and the associated connecting arcs are permanent. The remaining cells and connecting arcs are to be removed at the end of drydock construction. All sheet piles are PSX-32 and the connecting elements are 40-degree extruded wyes by United States Steel Corporation. The plan of a representative cell and arc is shown in Figure 5, and the details at the 40 degree extruded wye are shown in Figure 6. The manufacturer's rated interlock strength is 28 kips per lineal inch (4900 kN/m). Each cell contains 172 sheets and four wye sections. The connecting arcs each contain from 29 to 35 sheets depending on the cell location and spacing. The sheet lengths vary from a minimum of 83 ft (25.3 m) at Cell 11 to a maximum of 103 ft (31.4 m) at Cell 1.

Subsurface Conditions

A generalized profile of subsurface soil conditions, in a direction perpendicular to the shoreline, is shown in Figure 7. An alluvial deposit of loose to medium compact sands and gravels forms the uppermost soil stratum underlying the Hood Canal. At the cofferdam site, this deposit averages about 10 ft (3 m) in thickness. A glacial till stratum consisting principally of a very compact gray, silty, coarse to fine sand to sandy silt underlies the alluvium. The glacial till varies from about 20 ft (6 m) to 40 ft (12 m) in thickness. Underlying the till is a very compact, glacially overconsolidated deposit of interbedded sands and gravels

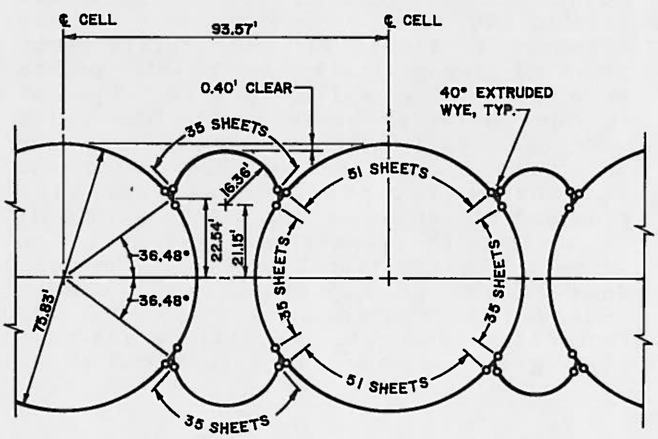

Figure 5 Cell and Connecting Arc
(1 ft = 0.305 m)

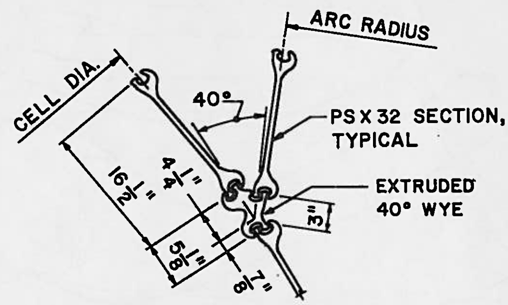

Figure 6 40 Degree Extruded Wye
(1 in. = 25.4 mm)

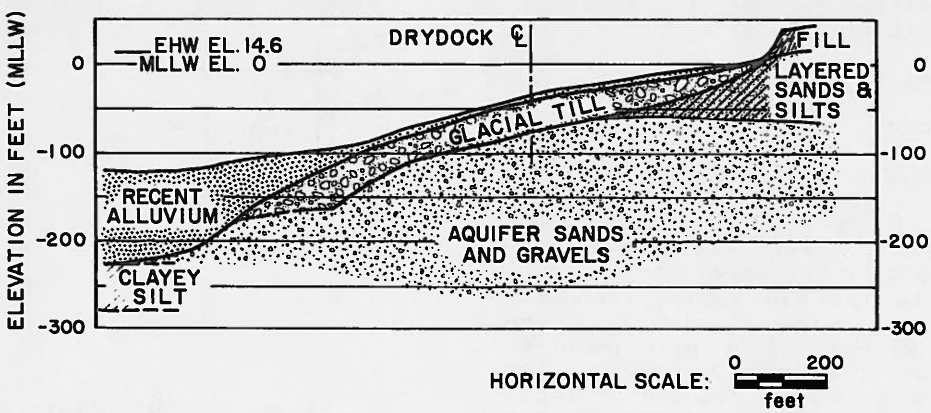

Figure 7 Generalized Soil Profile Perpendicular to Shoreline (1 ft = 0.305 m)

with silt layers, herein termed the aquifer sands and gravels. Available data indicate that the aquifer varies from about 200 to 250 ft (61 to 76 m) in thickness. Specific information on soils at greater depths is not known. It has been estimated that bedrock in the site area is at about El. -2000 ft (-610 m).[1]

An unusual site condition encountered was the discovery of high artesian pressures in the aquifer sands and gravels. Shown in Figure 8 are the piezometric elevations measured by piezometers installed both onshore and offshore. In the vicinity of the cofferdam, the piezometric level ranged from El. 30 ft (9 m) to El. 40 ft (12 m) which is approximately 25 ft (7 m) to 35 ft (10 m) above MTL. This condition imposed a need for two pumped well systems to control the aquifer piezometric levels for site dredging and subsequent cofferdam and drydock construction. The first system was installed on shore to facilitate site dredging and cofferdam construction. A second system installed at the cofferdam was subsequently used during and after cofferdam basin unwatering. This is discussed further in another paper.[2]

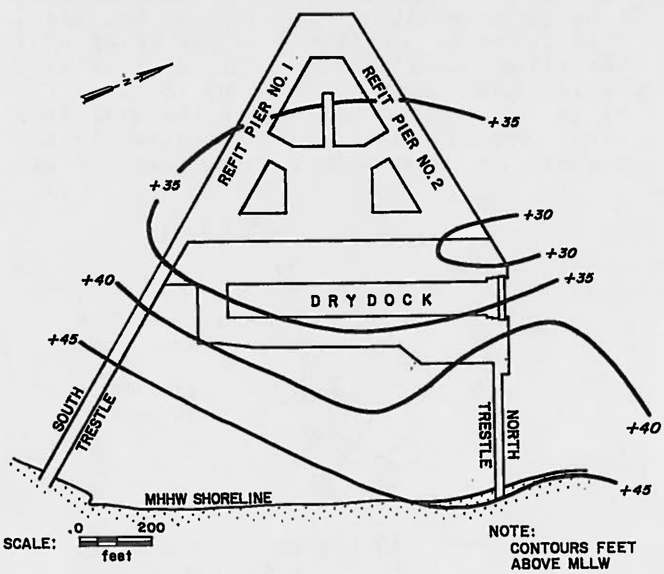

Figure 8 Aquifter Piezometer Conditions (1 ft = 0.305 m)

Cofferdam Design

Analysis Methods

The cofferdam was generally designed in accordance with the procedures of Design Manual DM-7.[3] However, the maximum interlock tensions caused by the cell and connecting arc were calculated in accordance with the TVA secant formula.[4] Because the very dense glacial till and the expected minimal sheet pile penetrations, the cofferdam was assumed to act similarly to a cofferdam founded on rock. The point of maximum interlock tension was assumed to occur at one-quarter the cofferdam height above the subgrade level, H/4. The reasonableness of this assumption is discussed later.

Cell interlock tensions obtained in accordance with DM-7 assume a sloped phreatic surface across the width of the cells resulting from weep hole drainage of the fill during and following cofferdam basin unwatering. Initial calculations indicated excessive interlock tensions would develop for this fill drainage condition. In order to achieve tolerable tensions for the high strength sheet piling, it was determined that the cell and connecting arc fill water levels would have to be lowered by pumped dewatering. Analyses disclosed that water levels had to be lowered to or below El. -45 ft (-13.7 m) and maintained essentially flat across the full widths of the cells and arcs commencing at a time near the completion of basin unwatering and continuing until completion of the drydock floor. El. -45 ft (-13.7 m) is only 16 to 19 ft (4.9 to 5.8 m) above the excavation subgrade.

Fill Dewatering System

The dewatering system selected consisted of two wells within each cell (three in the corner Cells 9, 11, 21 and 24) and one within each arc, with the wells being 6-in. (0.15 m) diameter slotted PVC pipe. The wells were located as shown in Figure 9. Submersible pumps operated automatically by on-off probes were used. The wells located adjacent to the cofferdam basin within the cells were not required to be pumped but were provided to function as observation and standby wells. Weep holes were also provided at and below El. -49 ft (-14.9 m) to monitor the dewatering system and to serve as an additional backup. The well design made it imperative that dredge residue be removed from the cells and connecting arcs prior to filling and that filling be performed in such a manner as

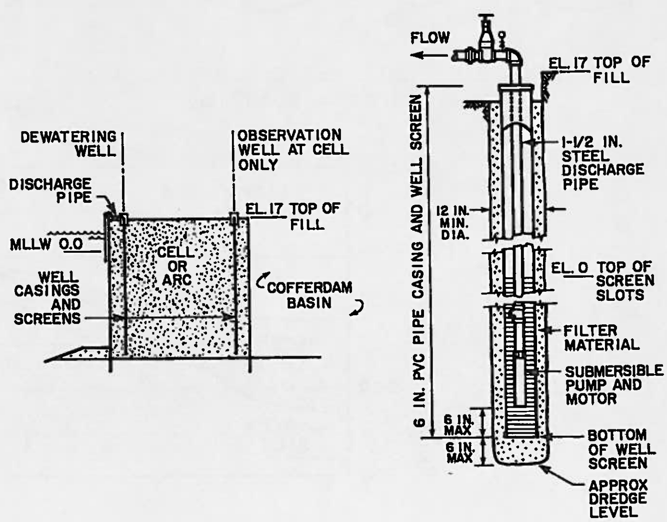

Figure 9 Fill Dewatering Wells (1 ft = 0.305 m)

to produce a clean, permeable soil within the lower levels near the dredge line.

Seismic Design

The submarine base is located in a moderately active seismic area. Geologic studies disclosed that no active faults were present within the site area which would require design for ground rupture. However, because most of the cofferdam will be permanent, consideration of vibratory ground motion was required. Specifically, it was necessary to prohibit excessive interlock tensions due to liquefaction of the fill and to limit cyclically induced settlements. Studies using the results of cyclic triaxial tests on potential borrow soils and published field records on liquefaction occurrences disclosed that a relative density of approximately 75 percent would be necessary to prevent liquefaction. Such a relative density could not be achieved by placing of uncompacted fill in-the-wet.

Studies disclosed that compaction by vibratory probe procedures would be the only feasible method to achieve the required density. The method selected consisted of filling the cells and arcs to above water level and then compacting the fill, full depth with equipment operating from the fill surface. Compaction depths ranged from 70 ft (21.3 m) to 90 ft (27.4 m). The effects that compaction would have on interlock tensions was a major consideration. Compaction of a cellular bulkhead had been performed previously, but no precedent existed for a cofferdam which would subsequently be unwatered.[5]

Choice of Cell Fill

Soil for use as fill in the cells and connecting arcs had to meet the following design requirements:

- High unit weight to provide stability to sliding and overturning after basin unwatering.
- High internal friction to minimize lateral earth pressures and interlock tensions.
- Limited percentage of gravel to permit full depth penetration of the probes for vibratory probe compaction.
- Relatively low percentage of fines to facilitate probe compaction and to permit rapid fill dewatering during cofferdam basin unwatering.

The fill specified was a well-graded, gravelly, coarse to fine sand having a 2-in. (50.8 mm) maximum stone size, from 15 to 45 percent passing a No. 40 sieve and 10 percent maximum passing a No. 200 sieve.

Cofferdam Instrumentation for Construction Monitoring

The effects that fill densification would have on lateral earth pressures and hence interlock tensions could not be estimated with confidence during design. Additionally, the pumped dewatering of the cofferdam fill was a key factor in limiting predicted interlock tensions within the very deep structure. In view of these unique aspects and the permanent nature of a major portion of the structure, geotechnical instrumentation was utilized to monitor the cofferdam performance.

Instrumentation consisting of vibratory wire strain gauges to monitor interlock tensions and inclinometers to measure sheet deflections was installed at the locations shown in Figure 10. Also, optical survey measurements were made at reference points established on the tops of selected sheets in each cell.

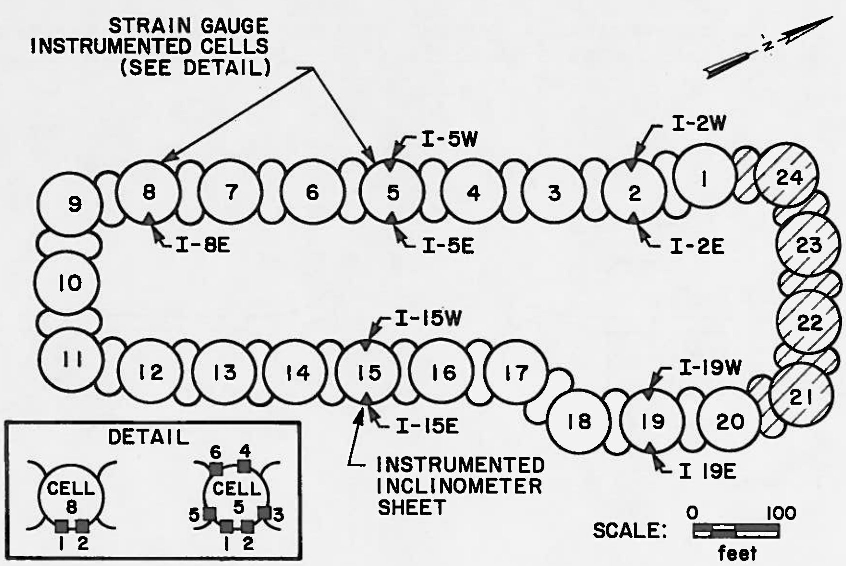

Figure 10 Locations of Instrumentation (1 ft = 0.305 m)

A total of 112 strain gauges were installed on two sheets at Cell 8 and six sheets at Cell 5. Sheets 5-1, 5-2, 8-1 and 8-2 were located at corresponding positions on the basin sides of the cells. Sheet piles 5-3 and 5-5 were the second sheets from the basin wyes within the common walls. Sheet 5-4 was located near the center of the Hood Canal side of the cell and Sheet 5-6 was the first sheet outboard of the wye on the Canal side.

The gauges were installed at four levels, as shown in Figure 11a, except for Sheets 5-4 and 5-6, where they were provided at only the lower three and two levels, respectively. The gauges were placed on the sheets before sheet pile threading and driving at levels such that, after driving, the gauges were at approximately El. 0,-20 ft (-6 m), -40 ft (-12 m) and -60 ft (-18 m). This placed the gauges near the dredge level and at intervals of approximately one-quarter the cofferdam height above. Additional details are shown in Figure 12a.

Inclinometers were installed at the centers of the basin and Hood Canal sides of Cells 2, 5, 15 and 19 and the basin side of Cell 8. Installation details are shown in Figures 11b and 12b.

Horizontal and vertical sheet movements for each cell were monitored using optical survey points at the tops of two sheets, one each at the center of the basin and Hood Canal cell segments.

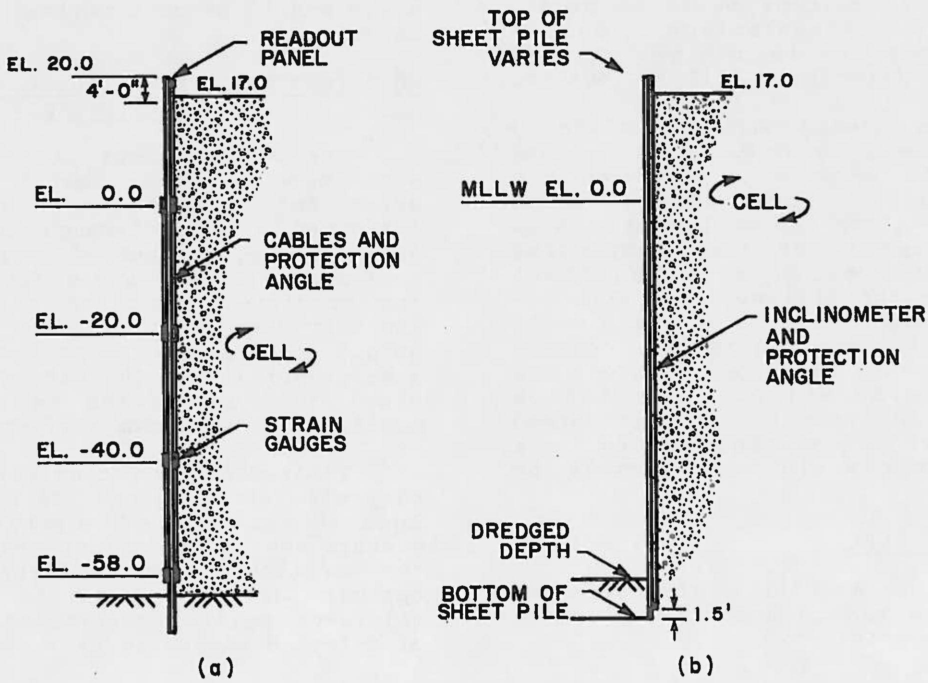

Figure 11 Instrumentation - Typical Sections (a) Strain Gauge Sheet (b) Inclinometer Sheet (1 ft = 0.305 m, 1 in. = 25.4 mm)

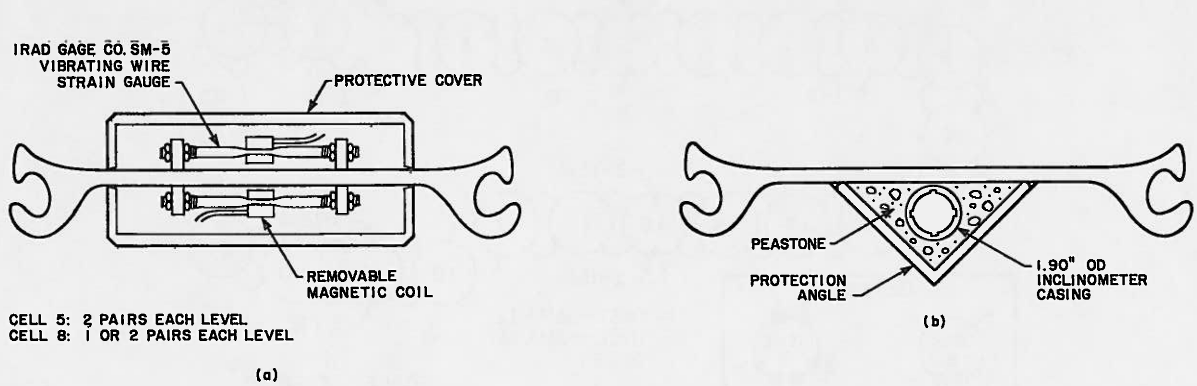

Figure 12 Instrumentation - Typical Details (a) Strain Gauges (b) Inclinometers (1 in. = 25.4 mm)

Construction Considerations

Site Preparation

The drydock floor subgrade is up to 30 ft (9 m) below the bottom grade of the canal and as much as 25 ft (8 m) below the surface of the dense glacial till. Positioning the cofferdam cells immediately adjacent to the drydock floor subgrade (Figure 3) required installing the sheet piling on the basin side of the cofferdam to below subgrade level.

Because of the density of the glacial till, it was believed that the sheet piles could not be driven into this layer more than a few feet without danger of driving the piles out of interlock. Accordingly, dredging was performed within the limits of the cells and connecting arcs prior to cofferdam construction as shown in Figure 13. Due to environmental considerations, the glacial till was excavated in-the-dry from within the drydock floor subgrade area following basin unwatering.

Cell Construction

The cell sheets were set around two 10 ft (3.1 m) high rigid circular steel templates approximately 74.7 ft (22.8 m) in diameter. The templates were positioned by four H-pile studs driven into the foundation soils. The distance between templates varied from 10 ft (3.1 m) to 30 ft (9.3 m), with the greater spacing being used at the deeper cells. The cell design radius was achieved at the upper template by placing wood blocks between the template and the sheets. No attempt was made to block the sheets at the lower template.

Sheet piles were set commencing at each of the four connecting wyes. Four closure points per cell were thus made, one near the center of each cell segment between wyes. Closure was similarly made near the central portion of each connecting arc. Two arc sheets were threaded to the wyes prior to driving the wyes or the cell sheets.

Sheets were driven with a MKT 10B3 single acting hammer having a rated energy of 13,100 ft-lbs (17.8 kN·m). Driving was terminated at 6 ft (1.8 m) or more below dredge level if five blows per inch (25.4 mm) was obtained on a pair of sheets. However, driving was stopped at less than 6 ft (1.8 m) if 10 blows per inch (25.4 mm) was reached on a pair of sheets. Sheets typically had at least 2 ft (0.6 m) of driven penetration with most sheets being driven about 4 ft (1.2 m).

Filling and Densification

The cells were mucked out with a smooth-edged clamshell bucket. A diver inspection was made to verify that an essentially firm, clean bottom existed prior to filling. All fill placed below water was deposited by lowering the bucket to the surface of the previously placed fill before opening. The cells were filled prior to placement of fill within adjacent arcs.

Fill compaction was performed with a vibratory probe manufactured by Toyomenka (America), Inc. The probe was 22 in. (559 mm) in diameter and approximately 100 ft (31 m) long. Penetration of the probe and compaction was achieved by a combination of vertical and horizontal vibrations generated by vertically and horizontally vibratory pile drivers, located at the top and tip of the probe respectively. Initial penetration of the probe was aided by jetting with two, 2-in. (51 mm) diameter

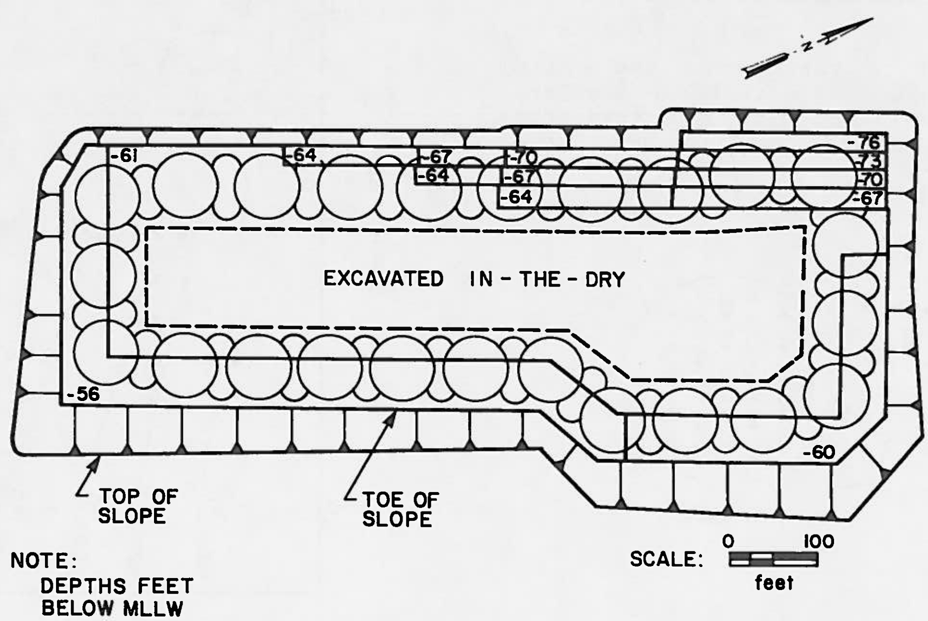

Figure 13 Dredging Plan (1 ft = 0.305 m)

jet pipes which discharged vertically. Compaction was performed by consecutively raising the probe 6 ft (1.8 m), holding it momentarily, and then reinserting it 3 ft (0.9 m), until the probe was withdrawn to the surface. As compaction proceeded, fill was added by placing additional soil around the probe.

Following a trial compaction program at Cell 8 involving 19 probes which tested various center to center probe spacings, a procedure was established to compact the remaining cells. Ten additional probes were subsequently required to complete compaction at Cell 8.

The next six cells were compacted using 31 probes located at about a 12 ft (3.7 m) center to center probe spacing. This initial spacing was found to provide a level of compaction in excess of that required. Nineteen probes were used within the remaining 17 cells, at about 15 ft (4.6 m) spacings. The sequence of compaction for each pattern is shown in Figure 14. Compaction results were measured by the standard penetration test with the results correlated to relative density in accordance with Gibbs and Holtz.[6] Typical results are shown in Figure 15.

Compaction in the connecting arcs was conducted following compaction of the adjacent cells. The number of probes within each arc varied depending on the area but typically ranged from four to six.

Cell Fill Dewatering

The wells for dewatering the cell and connecting arc fill were installed following fill compaction. Both pumps within each cell were used during basin unwatering. Intermittent pumping from only one well per cell maintained adequate drawdown thereafter.

Measurements taken when the wells were not pumping showed that the dewatering system maintained hydraulic gradients across the cells, ranging from near zero to less than 5 ft (1.5 m).

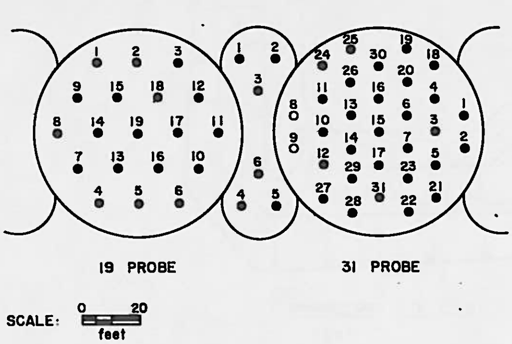

Figure 14. Vibratory Probe Patterns (1 ft = 0.305 m)

Cofferdam Basin Unwatering

The basin was lowered at a rate no greater than 5 ft (1.5 m) per 24-hour period. At each 10 ft (3 m) increment below El. 0, the basin water level was held constant for a day while the cofferdam performance monitoring instrumentation was read, a diver inspection was made of the interior and exterior perimeter of the cofferdam, and the data were reviewed to evaluate cofferdam performance.

The basin level was maintained at El. -20 ft (-6 m) for a three week period while the cells were proof tested. The proof loading was obtained by temporarily raising the water level in a cell and connecting arc to El. 0. Typically two to three cells and the adjacent connecting arcs were proof tested at one time. The estimated maximum interlock tension under this loading was comparable to that expected during the final unwatering stage.

Interlock Tensions During Cell Construction and Compaction

Interlock tensions measured after filling and compaction are shown in Figure 16 and Figure 17 for Cells 8 and 5, respectively. The following comments apply:

- Cell 8 was the cell at which trial compaction operations to establish production probe spacings took

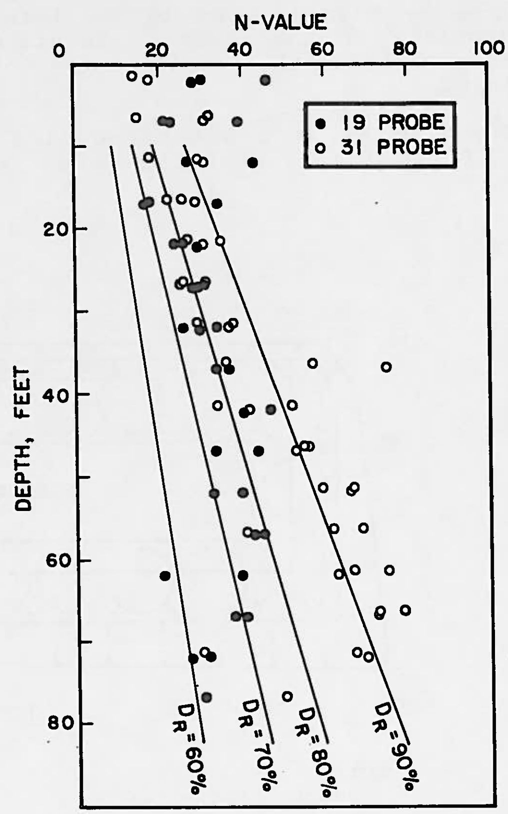

Figure 15 Compaction Results (1 ft = 0.305 m)

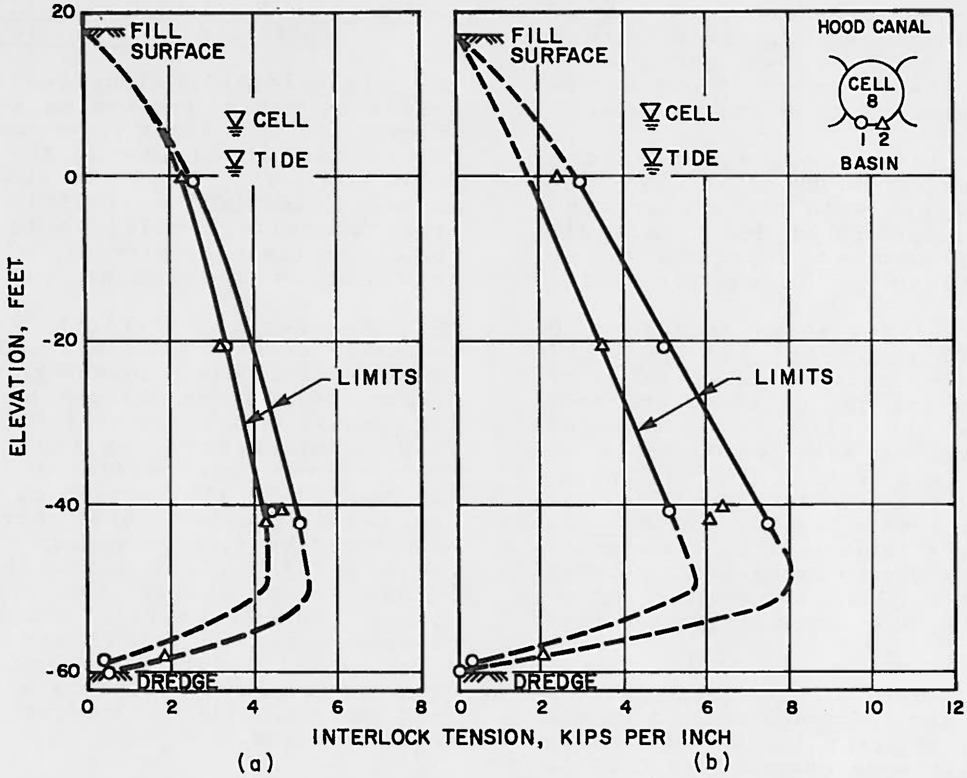

Figure 16 Cell 8 Interlock Tensions (a) After Filling (b) After Compaction (1 kip/in. = 175 kN/m, 1 ft = 0.305 m)

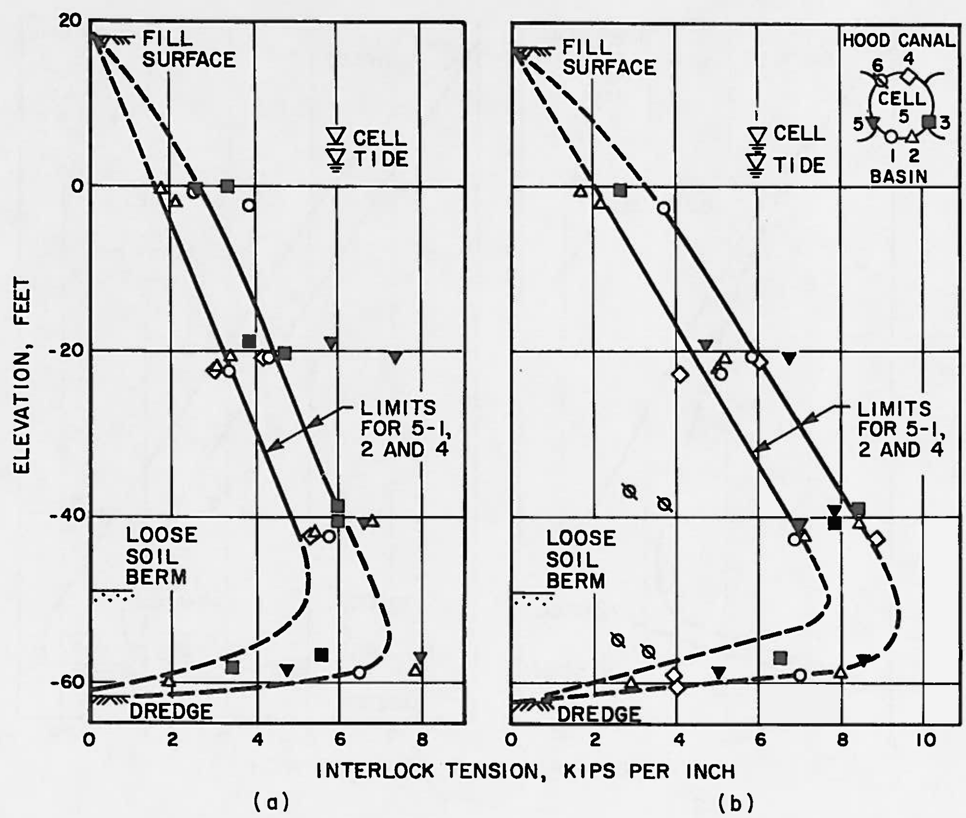

Figure 17 Cell 5 Interlock Tensions (a) After Filling (b) After Compaction (1 kip/in. = 175 kN/m, 1 ft = 0.305 m)

place. As such, the cell compaction was somewhat atypical with a total of 29 probes being made versus the 19 and 31 probe arrays utilized at the other cells.

- During tide cycles, interlock tensions varied by up to 1 kip/in. (175 kN/m), with the higher tensions occurring at low tide. All data presented herein are for conditions at about low tide.

The limit lines shown in Figure 16 and Figure 17 represent the authors' estimate of the probable total range of interlock tensions on the basin and Hood Canal sides of the cells. The causes of the abnormally low measured tensions at Sheet 5-6 are not known. Cell 5 consistently exhibited higher tensions than Cell 8. Reasons for this are not certain but possible factors include differing compaction patterns and minor variations in cell fill gradation. Also sheet pile embedments at Cell 8 were somewhat greater than at Cell 5.

Small differences in tensions between the sheets on the cofferdam basin side and those located adjacent to the wyes along the common wall were observed at Cell 5. It is seen that compaction typically caused an increase of from near 0 to about 2 kips/in. (350 kN/m).

Interlock Tensions During and Following Cofferdam Basin Unwatering

Interlock tensions measured subsequent to basin unwatering are shown in Figure 18. The limit lines shown for Cell 5 are for the sheets on the basin side (5-1 and 5-2). The limit lines for Cells 5 and 8 indicate a relationship between the two cells similar to that observed prior to basin unwatering, with the Cell 5 tensions continuing to be higher.

Changes in interlock tension which occurred in Cell 5 at El. -60 ft (-18 m) subsequent to basin lowering, are shown in Figure 18. These changes are attributed to removal of a loose cell fill berm which had accummulated against the basin side of the cell to El. -49 ft (-15 m). At Sheet 5-2, the tension at the El. -60 ft (-18 m) upper level strain gauges increased by 1.65 kips/in. (290 kN/m) to a value of 10.2 kips/in. (1,790 kN/m). Other lesser changes occurred as indicated. The relatively small increases observed at Sheets 5-3 and 5-5 suggest that significant resisting frictional forces had developed between the sheets and the cell fill along the common walls near the wyes.

Following unwatering, some differences were observed between the Cell 5 interlock tensions at the sheets on the basin side and those on the common wall

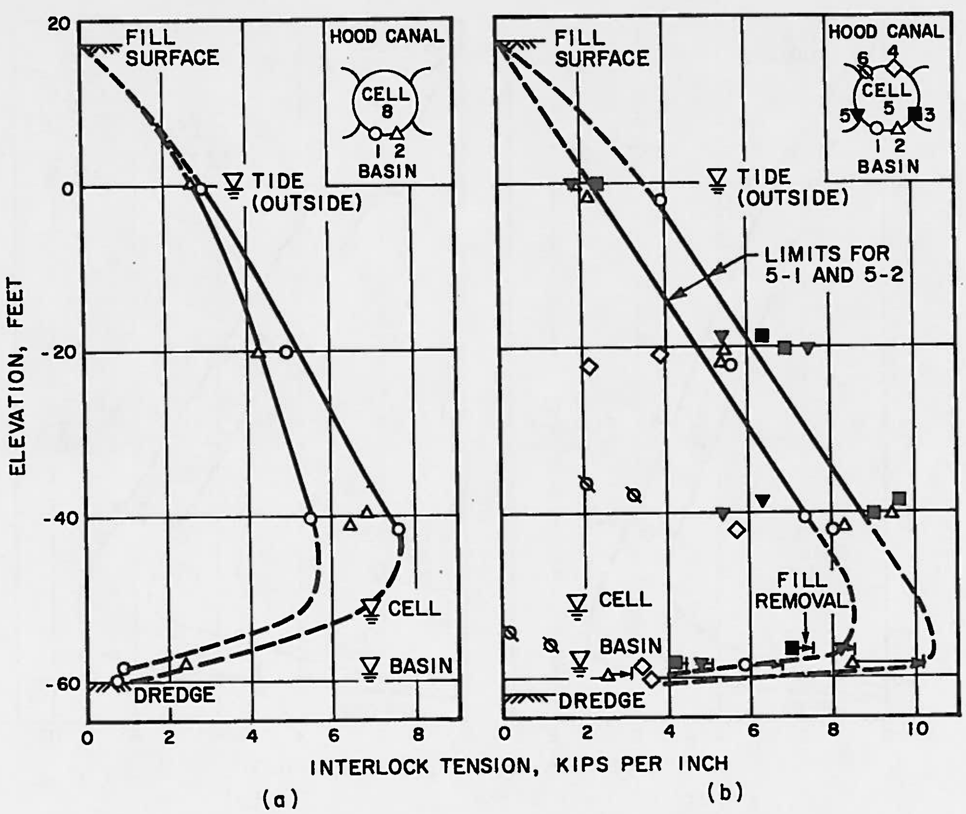

Figure 18 Interlock Tensions After Basin Unwatering (a) Cell 8 (b) Cell 5 (1 kip/in. = 175 kN/m, ft = 0.305 m)

side of the wyes. At the El. 0 level, the common wall tensions were essentially identical to the lower limit of the tensions on the basin side. At the El. -20 ft (-6 m) level, the average of the common wall tensions was about 20 percent higher than the average of the basin side sheets. At the El. -40 ft (-12 m) level, a significant difference in measured tensions existed between the two common wall sheets. However, the average for the common wall sheets was essentially equal to the average for the basin side sheets.

When the cofferdam basin and cell fill water levels were lowered, the measured interlock tensions at the cell sheets on the basin side increased whereas the tensions on the Hood Canal side decreased. Lowering of the water levels increased the lateral pressure on the basin side of the cell. However, the lowering of the cell water level decreased the lateral pressure on the Hood Canal side. The direction of the measured changes is compatible with the expected behavior.

Sheet Pile Profiles and Movements

Inclinometer profiles for Cell 5 are shown in Figure 19. These data are typical of those for the other inclinometers. Since the inclinometer casings were not installed until after cell filling, the as-driven sheet profiles and the sheet movements during cell filling are not known. The data show the sheet profiles relative to the design locations which are referenced to the cell centerline.

It is seen that the sheets were significantly out of plumb. Data from the nine inclinometers revealed that the instrumented sheets ranged from 12.7 to 54.5 in. (323 to 1,380 mm) out of plumb, top to bottom. The data consistently showed that the tops of the sheets were outside the design locations and the bottoms were inside. Compaction of the fill resulted in movement of the Hood Canal and basin sides of the cell, away from the cell centerline, producing a somewhat oval shape. The data have been plotted on the assumption that the sheet pile tips were fixed.

Because of the degree of out-of-plumbness measured at the inclinometer sheets, extensive diver plumb line and dredge line surveys were made at each cell. These surveys disclosed that at the dredge line, the sheets near the wyes were typically outside the design radius whereas those midway between the wyes were inside the design radial location. This is believed to be the result

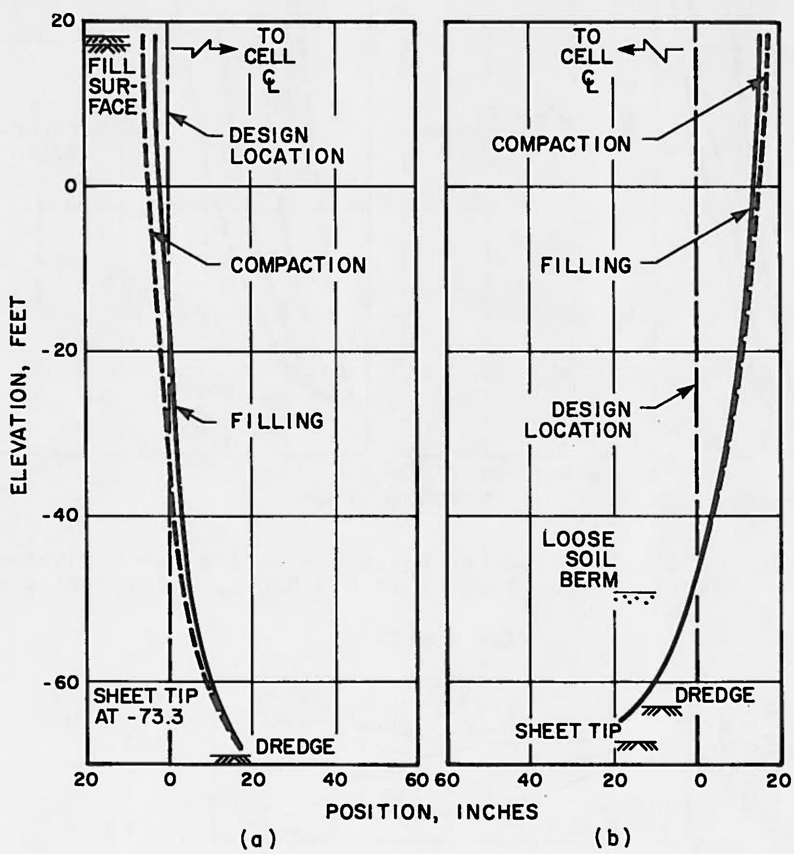

Figure 19 Cell 5 Inclinometer Profiles After Filling and Compaction
(a) I-5W (b) I-5E (1 ft = 0.305 m, 1 in. = 25.4 mm)

of the cell construction procedures wherein blocking was only used at the upper template level. The sheets set initially near the wye were positioned with the bottom outside the radial line and the remaining sheets had to be toed in to complete closure. All inclinometers were located on sheets that were near the closure points, where the sheet tips were inside the design radius.

Movements of the Cell 5 inclinometers during unwatering are shown in Figure 20. The displacements shown are incremental movements for the various stages of unwatering from the sheet pile positions which existed when the basin was at El. 5.3 ft (1.6 m). Movements resulting from proof testing the cell at a basin level of El. -20 ft (-6 m) are included. The causes of the movements during proof testing are not fully understood, but are believed to be, at least partially, the result of creep movements which occurred during the three week period that the basin was at El. -20 ft (-6 m).

Total movements measured at the top of the cell were very small, with the movements of both inclinometers being less than 3 in. (76 mm) during unwatering to El. -59 ft (-18 m). The data indicate a substantially rigid body rotation of the cell with a slight bulging of the basin side.

Optical survey measurements of the movements of the tops of the west side cells as a function of cofferdam basin level are shown in Figure 21. The data shown are for the reference points on the basin sides of the cells. The largest total movement at the end of unwatering was slightly less than 4 in. (102 mm) at Cell 2. Movements of the east side cells were generally equal to, or less than, those for the deeper west side cells.

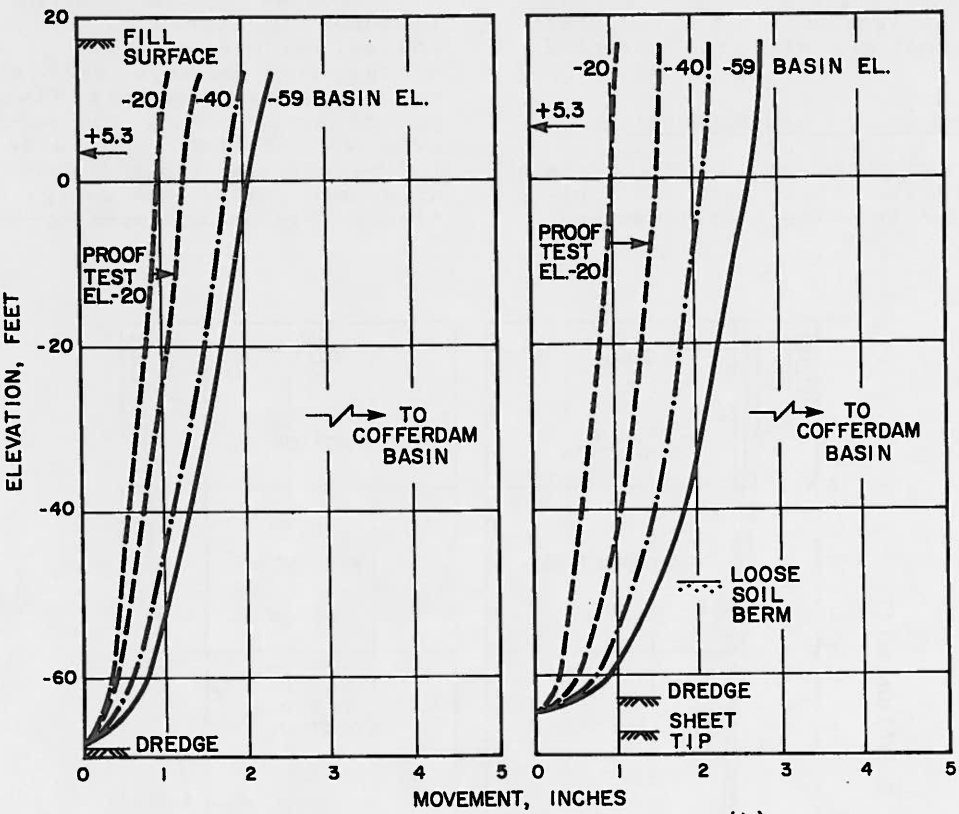

Figure 20 Cell 5 Inclinometer Movements During Basin Unwatering (a) I-5W (b) I-5E (1 ft = 0.305 m, 1 in. = 25.4 mm)

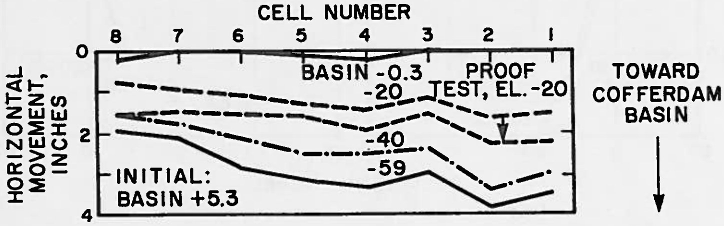

Figure 21 Inward Movements Tops of Cells During Basin Unwatering (1 in. = 25.4 mm)

Discussion of Performance Data

Interlock Tensions

The authors' interpretation of the cofferdam data is as follows:

- For the basin side sheets, the interlock tensions both before basin unwatering (Figures 16 and 17) and after (Figure 18) indicate that maximum tensions occurred at approximately 10 ft (3 m) above the dredge line. This compares to the common design assumption of maximum interlock tension at the H/4 point, which is approximately 20 ft (6 m) above the dredge level for this cofferdam.

- For the common wall sheets near the wyes, the after filling and after compaction data indicate that the maximum tension also occurred at approximately 10 ft (3 m) above the dredge level. However, after unwatering, the point of maximum tension is at a higher elevation. This is believed to be the result of the restraint provided by the connecting arc fill, which limited deformations of the common wall sheets near the bottom of the cell during basin unwatering.

- Values of K_h shown in Figure 22 have been calculated based on in-place soil densities and the limit lines of measured interlock tensions indicated in Figures 16, 17 and 18. After filling, Figure 22a, K_h values for the loose fill were relatively constant with depth in both cells, ranging from a low of about 0.2 to a high of about 0.4. After compaction, Figure 22b, K_h values increased with depth and ranged from a low of about 0.2 at El. 0 to a high in Cell 5 of 0.46 at the El. -40 ft (-12 m) level. Conversely after basin unwatering, Figure 22c, K_h values decreased with depth, ranging from 0.3 to 0.45 at the El. 0 level and from 0.23 to 0.36 at the El. -40 ft (-12 m) level.

  For the three conditions, the K_h values for the after-filling condition are believed to be the most representative values of the "true" lateral earth pressures within the cells because of the absence of prestress effects in the sheets from compaction operations.

- The data for Cell 5 show that the interlock tensions at the common wall sheets next to the wyes were from zero to 20 percent higher than measured on the basin side By comparison, the TVA secant formula for the Cell 5 geometry indicates that the common wall tensions near the wye should be about 50 percent greater than for the basin side sheets.

Cofferdam Movements

The inward movements of the tops of the cells during basin unwatering ranged from 1.5 to 4.0 in. (38 to 100 mm), which is 0.16 to 0.42 percent of the height. This range of movements is significantly less than what would generally be expected for cofferdams of this height. The reduced deflections are considered to be principally the result of the quality of the fill and the compaction operations which increased the fill density and shear modulus.

It has been reported that following basin unwatering, maximum bulging of the cells occurs at approximately H/4 above the dredge level.[7,8] From these observations it was concluded that the point of maximum tension likewise occurred at H/4. The inclinometer data in Figures 19 and 20, in conjunction with the interlock tension data, do not appear to support these observations. It is believed that the high quality of the fill and the densification may have contributed to this occurrence.

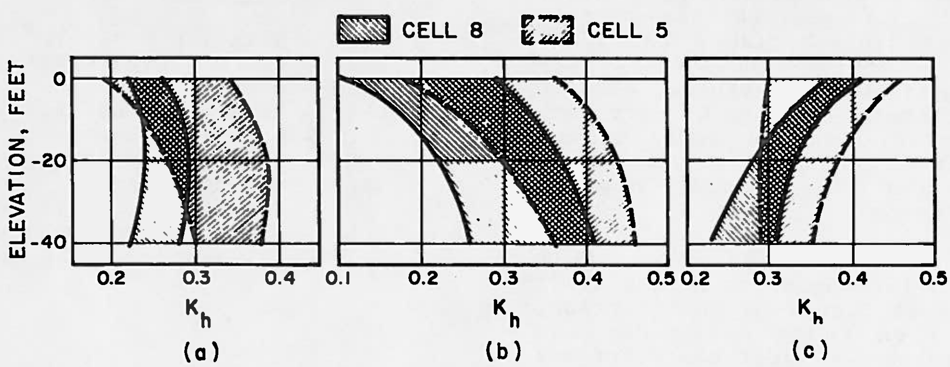

Figure 22 Range of Lateral Earth Pressure Coefficients Cells 5 and 8 (a) After Filling (b) After Compaction (c) After Basin Unwatering (1ft = 0.305 m)

Conclusions

The following principal conclusions are made:

1. High strength sheets and extruded wyes can be successfully utilized to carry high interlock tensions and facilitate construction of steel sheet pile cellular cofferdams suitable for resisting 80 ft (24 m) water depths.

2. The data show that substantial interlock tensions developed near the dredge level. Throughout construction, the maximum interlock tension is believed to have occurred at approximately 10 ft (3 m) above the dredge level on the basin side sheets. The maximum interlock tensions for the common wall sheets appeared to also develop at the same elevation prior to basin unwatering but may have existed at a higher elevation following unwatering.

3. Lateral earth pressure coefficients can be variable with depth depending on the construction procedures. For compacted, well graded sand and gravel fill, K_h values as high as about 0.45 developed.

4. For the 40 degree extruded wye connection, the data indicate that the common wall tension near the wye is not significantly higher than the tension in the basin side sheet piles.

5. The quality and density of the cell fill have a significant influence on the magnitude of cell movements during cofferdam basin unwatering.

Acknowledgements

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Notations

D_R	- Relative Density
EHW	- Extreme High Water
El.	- Elevation
MLLW	- Mean Lower Low Water
MTL	- Mean Tide Level
H	- Cofferdam Height
K_h	- Lateral Earth Pressure Coefficient
N	- Standard Penetration Resistance

PRACTICAL UNDERPINNING OPERATIONS

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Abstract - Factors affecting the necessity, design and choice of an underpinning system are presented. Typical methods are described noting the advantages and disadvantages of each. Failure of each system are described - noting the cause of the failure and steps that should be taken to preclude a repetition of same.

Introduction

Underpinning is the art of altering an existing foundation to provide increased depth or increased bearing capacity.

Massive structures such as cathedrals and castles settled and were underpinned during the middle ages. Due to the lack of subsurface information however, many of these operations resulted in failure. Modern day cities with their skyscrapers and the deep foundations associated with them spawned the modern art of underpinning. The construction of subway and large underground sewerage systems led to the development of new methods to cope with the great variety of sub-soil conditions encountered. As the cities grew, more and more structures were erected on marginal sites that previously were bypassed.

Conditions Requiring Underpinning

The need for underpinning may be due to new construction adjacent to a building or to the inadequacy of the existing foundation to support the present loadings, as evidenced by settlement, or to support additional loadings to be imposed.

This paper will examine underpinning requirements due to new construction. The methods described however are identical with those used to halt settlement or to increase foundations bearing capacity. In most cases underpinning is required when an adjoining excavation goes deeper than an existing foundation.

The criteria most commonly used requires underpinning if an influence line taken from the edge of the new excavation at subgrade falls below the existing footing, see Figure 1. The slope of this influence line may vary from a 1 on 2 in water bearing stratas to almost vertical in rocklike formations. The most generally used influence line being a 1 on 1.

Factors Determining the Depth and Type of Underpinning Systems to be Used

1. Local geology is of prime importance. The characteristics of the various stratas underlying the existing foundation must be determined. This is best accomplished by exploratory soil sampling with accurate analysis by a soils laboratory. The presence of boulders, water bearing stratas, plastic clays, etc. all play a part in the selection of the underpinning method to be used. Unlike the Middle Ages, we have the capacity to reasonably predict the behavior of various soil stratas under load. This has resulted in the elimination of many underpinning failures that otherwise would have taken place.

2. The type of structure also may dictate the underpinning method to be used. A structure must be evaluated prior to any undermining of its foundation. The ability of a building to transfer loadings from one part of the structure to another is the basic premise behind all underpinning operations. This arching ability is apparent to all who have seen a building being demolished.

The building to be underpinned must be inspected with all cracks or other signs of distress noted. The foundation loadings must also be determined. This often necessitates a thorough inspection; measurement of wall thickness, floor to ceiling heights etc., with loads computed on the basis of these measurements.

3. The depth of the adjoining excavation and location of the new perimeter walls also determine the minimum depth and location of the underpinning system. It is important to note that the underpinning must be as deep as an abutting excavation and often must extend many feet below this point. Should the soil at subgrade of the new construction be inadequate to sustain the underpinning loads, the underpinning

must extend to a deeper strata.

Concrete Pit Underpinning Installations

The simplest and most common method of underpinning is the excavation and concreting of hand dug pits under the existing foundation.

General excavation is first completed to approximately one foot above the bottom of the footing to be underpinned. The procedure then is as follows: (Figure 2)

1. Hand excavate a pit adjacent to the footings (called an approach pit) extending about four or five feet in depth, Figure 2a. The sides of the pit are prevented from caving in by the placement of horizontal wood sheeting. The sheeting normally used is 2" x 8" lumber with spacers between the individual rings. The spacers are necessary to permit the skilled underpinner to backfill tightly between the boards and the earth. One of the requisites of this method is the prevention of ground loss from under the remaining portions of the footing not yet underpinned.

2. Once the approach pit has been completed,a drift is made under the existing footing to the depth of the approach pit, Figure 2b. This area is sheeted as noted previously.

The bottom of the footing is then cleaned of all debris and soil to insure good bearing against the completed underpinning.

Continue the pit excavation to subgrade. In some cases it is advantageous to bell the bottom of the pit to enlarge the bearing area and thereby increase the carrying capacity of the pier.

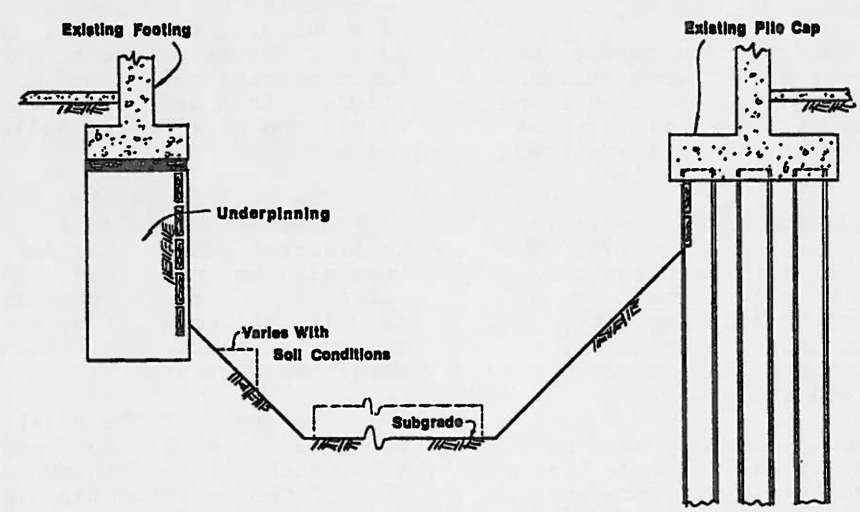
3. The pit is then filled with concrete to within two or three inches of the underside of the existing footing, Figure 2c.

After the concrete has set a minimum of sixteen hours, the two or three inch void is filled with drypack consisting of one part cement, one part sand and just enough water so that the mixture will retain its shape when molded by hand. The drypack is rammed into place using a 2" x 4" and an eight pound mall. The loads have now been transferred from the existing footing to the underpinning.

If the underpinning pier is bearing against soil with the same properties as the existing footing then the bearing area of the underpinning must be equal to that of the footing. In most cases, however, this is not the case. The bearing value of a soil usually increases with depth. This then permits the use of intermittent pits, in lieu of continuous underpinning under wall footings, Figure 3. Horizontal 3" sheeting is attached to the side pit boards as shown providing the required bank protection of the area between the pits.

Pit Sheeting

Due to the arching action of the soil, pressure on the sheeting boards in nominally sized underpinning pits up to 7' x 7' does not increase with depth. It is therefore possible to use 2" sheeting in pit sizes to 5' x 5' and 3" sheeting in larger sized pits. Pits with a side dimension



INFLUENCE LINE FOR EXTENT OF UNDERPNG

Figure 1

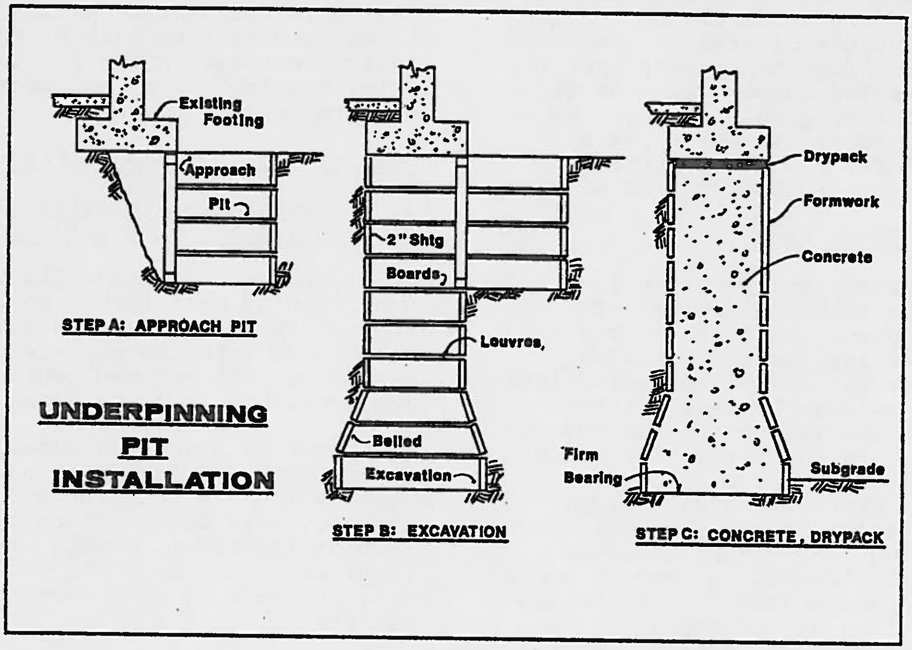


Figure 2

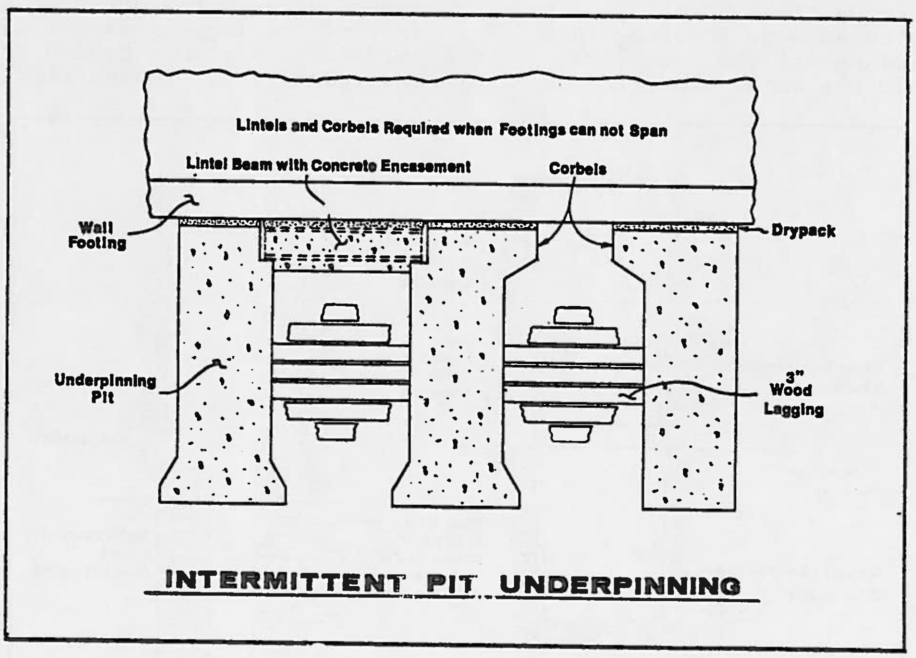


Figure 3

greater than seven feet usually require the use of intermediate bracing. Cost records have shown that 3' x 4' underpinning pits have proven to be the most practical size. The use of treated lumber in unnecessary. After many years even though the untreated lumber does rot it still retains its original volume. The strength of the lumber is lost, but at that point in time, its main function is to prevent loss of ground; this it does.

Pit Excavation Procedures

The excavation, taking place in a constricted area prevents the use of mechanized equipment. All the excavation is removed by skilled craftsmen using manual methods. The soil is shoveled into small pails or caisson buckets. These buckets are then hauled to the top and deposited adjacent to the work. The underpinner at the bottom of the pit decides how deep he can excavate in each lift prior to the placing of the protective 2"x 8" wood sheeting. Should he excavate too far, prior to lagging, a cave-in could occur with an accompanying loss of ground from other portions of the footing. In dry granular material it is often necessary to cut the depth of the sheeting boards from 8" to 4" to prevent such cave-ins. The site must also be dewatered prior to the start of an underpinning pit operation to permit the excavation to be carried out in the dry. There are cases, however when water is encountered directly overlying an impervious strata. If the water bearing strata is shallow, it is possible to drive vertical steel sheeting or tongue and grooved wood sheeting into this strata, sealing off the water, see Figure 4. Should the water condition be excessive or dewatering impractical at the time the underpinning system is to be installed, other underpinning alternates are available and should be used. It is sometimes necessary to jack or drive piling inside an underpinning pit to penetrate a water bearing strata and provide adequate bearing capacity.

Special Applications of Pit Underpinning

1. Underpinning to serve as perimeter wall of adjoining construction.

Often due to space limitations, the underpinning must serve two purposes. Support the existing structure and be incorporated into the new structure as a perimeter wall or load bearing element. The procedure is as follows: (see Figure 5)

Over excavate the underpinning pit by approximately 6 inches to permit setting a plywood form to coincide with the face of the perimeter wall. Place all necessary reinforcing steel, dowels, waterstops if required, etc. Concrete pit and transfer footing loads to the underpinning by dry-packing.

The underpinning pits are not necessary continuous and in most cases intermittent pits are used approximately 10 feet on centers. After all the pits have been concreted and building loads transferred through the underpinning; commence excavation to subgrade. The excavation should proceed in lifts not to exceed five feet and 3" horizontal lagging is placed between the underpinning pits. The face of the lagging as set should be a sufficient distance behind the face of the underpinning pit to install the required

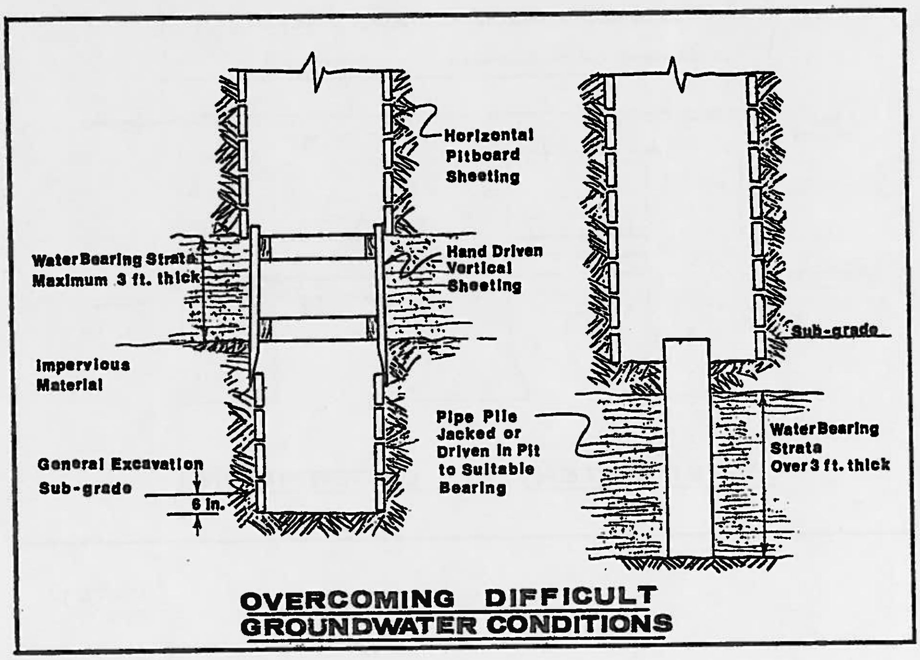


Figure 4

structural concrete wall. Once the excavation has been completed to grade, formwork is placed between the pits and the intermediate walls poured. The net result is a finished perimeter wall without encroachment into the adjoining excavation.

2. Underpinning to be undermined by new footings in adjacent excavation.

It is sometimes necessary for footings for new construction to extend past and below the footing line of an existing building. The procedure is as follows: (see Figure 6).

a. Excavate the conventional underpinning pit to 2'+ below the subgrade of the new construction.
b. Cast a two foot thick concrete mat. The top of the mat to be at the underside of the new footing.
c. Place several steel posts on top of this mat extending to about one foot above the top of the new footing. A layer of sand is then placed from the top of the mat previously poured to three inches above the theoretical top of the new footing. Fill the remainder of the pit with concrete.
d. Excavate to the new subgrade, removing the previously placed sand portion of the underpinning pit. This will now permit the placing of the new footing as required.

It is obvious that in this instance the soil pressures on that portion of the footing extending beneath the underpinning will be increased by the pressure created by the underpinning. The ground must be capable of withstanding this increased load.

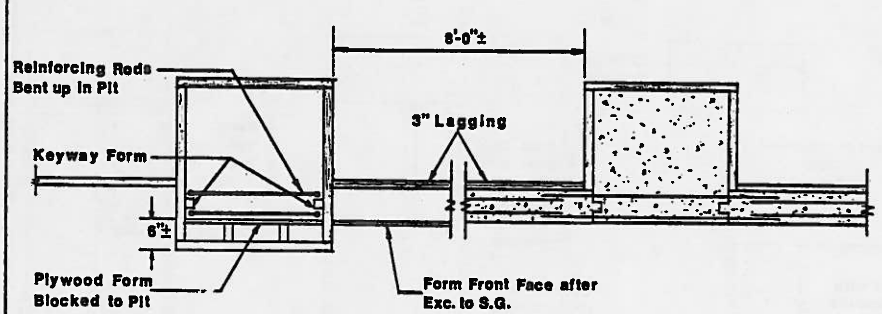
3. Underpinning of large footings with access from one side only.

Structures often require large footing areas to distribute the column loads into low strength soils. Individual footing sizes in excess of 10' x 10' are relatively common. Many projects require that the underpinning operation be completed by approaching the work from the exterior of the building only. The underpinning procedure in this special case would be as follows: (see Figure 7).

a. Excavate and pour underpinning pit "A" to within 5 ' of the underside of the existing footing.
b. Set steel posts between the top of the underpinning as poured and the underside of the footing. Transfer the footing loads into the underpinning by the use of plates and steel wedges.
c. Once the above is completed; drift over the top of pit "A" and excavate pit "B" to grade.
d. Concrete pit "B" within 3" of the underside of the existing footing. Fill this space with drypack as described previously; completing the load transfer.
e. Concrete pit "A" and drypack.
f. Repeat the above procedure for pits "C" and "D".

4. Underpinning of small isolated footings

The underpinning of this type of foundation requires a temporary shoring system



UNDERPINNING UTILIZED AS NEW PERIMETER WALL

Figure 5

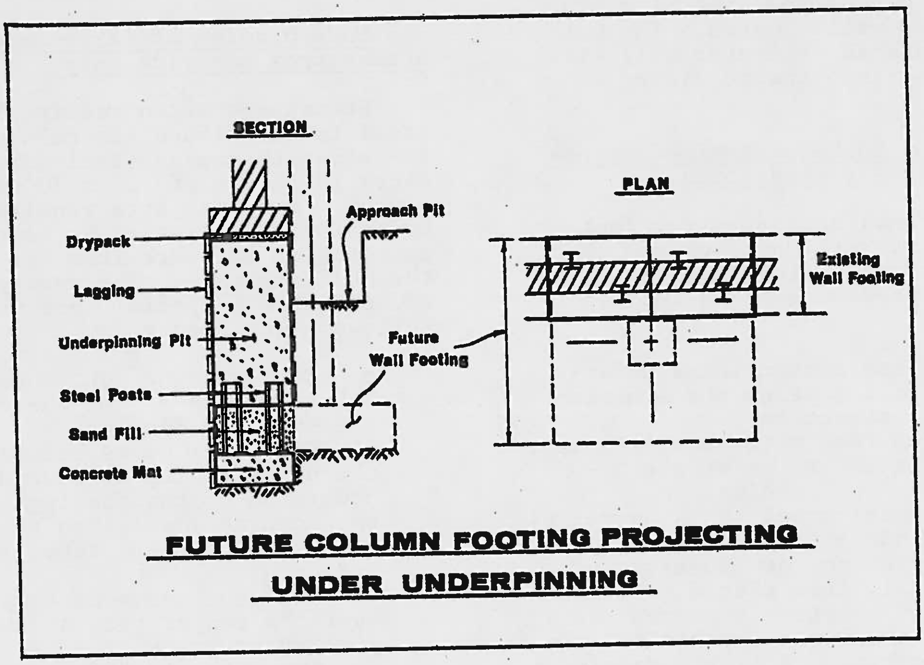


Figure 6

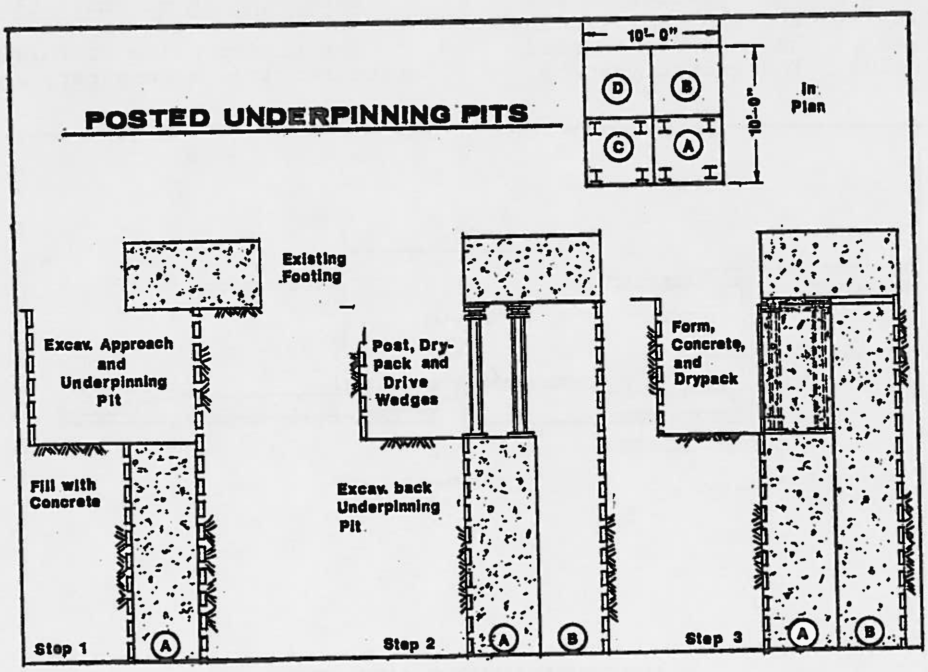


Figure 7

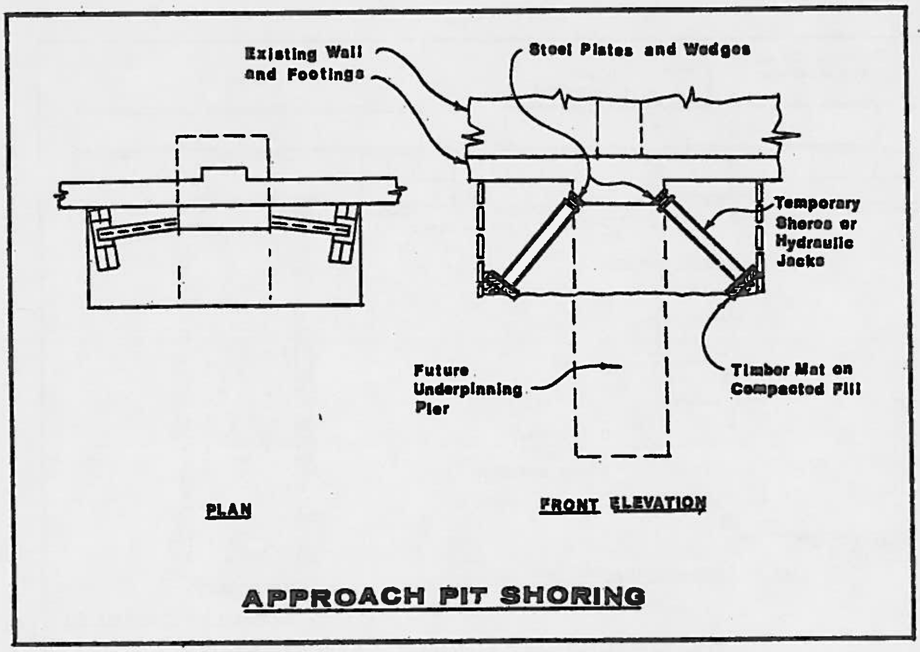


Figure 8

to support the total load prior to undermining the footing. On lightly loaded footings this support can often be realized by the use of hydraulic jacks or steel shores placed in the approach pit. (see Figure 8). Another method is as shown in Figure 9.

The top of the existing footing is exposed. Four or more holes are drilled through the footing and approximately 6" into the ground. Heavy bolts are then placed into the holes as shown and attached to the header beams. The ends of the header beam bear on mats placed beyond the influence line of the underpinning. A drift is then made at one bolt location at a time; A plate is attached to the bolt to bear against the bottom of the existing footing. Drypack is used between the plate and the footing to evenly distribute the loadings. The footing load is finally transferred to the header beams using plates and wedges. In some cases it may be necessary to use hydraulic jacks to pre-deflect the header beams. Once the temporary shoring is completed, the underpinning pit may be safely installed.

Potential Failure Factors of Concrete Pit Underpinning

1. The ground at subgrade is not adequate to sustain the imposed loads. Despite all preliminary investigations, the unexpected is often encountered in all subsurface construction. A thin layer of peat or soft clay may be encountered at subgrade and not the hardpan anticipated. Inexperienced supervision may blindly bottom out the underpinning in this inadequate strata causing settlement of the structure underpinned. Competent supervision would prevent this from occurring.

2. The wall to be underpinned is not structurally sound and has almost no arching whatsoever. This is common in old rubble wall foundations. When undermined, the wall collapses in the immediate area endangering the remaining portions of the structure. In cases such as this it is necessary to reinforce the rubble wall prior to commencing the underpinning operations. One commonly used method is to pour a skin coat of 6"± of concrete against the wall using wire mesh with reinforcing rods doweled into the masonry.

3. Pile driving or blasting may set up vibrations that adversely affect the underpinning by consolidating the soil under the pits causing settlement. In all cases where such a potential exists precautionary steps should be taken. A recess should be made in the underpinning pit to permit the placement of hydraulic jacks. Should the underpinning settle as a result of vibration, the use of hydraulic jacks between the footing and the underpinning pit would minimize any damage that would otherwise take place.

4. The lack of lateral restraint against underpinning piers in excess of 12 feet in depth, is the most frequent cause of structural damage during an underpinning operation. A rule of thumb is all pits in excess of 12 feet in depth have to be restrained either by bracing or tie-backs. In underpinning one of the vital considerations is the prevention of lateral movement. As a general rule, lateral movement of an underpinning system in the range of 1"± will cause more structural damage than

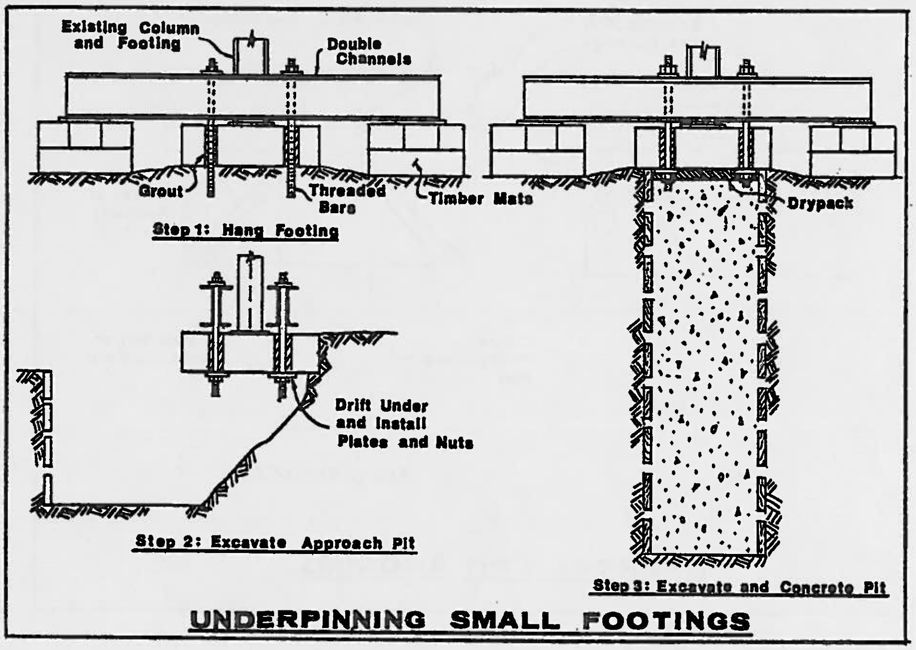


Figure 9

a settlement of 1"±.

5. Undermining of the concrete pit underpinning may take place if the soil at subgrade is not satisfactory for the new construction and the cut must be deepened several feet or more to a more suitable bearing strata. This does happen and at times it has occurred when the underpinning contractor is no longer at the site. The excavator may not question the effect the extra depth cut will have on the underpinning and excavate to the new grade causing settlement. This type of failure could be eliminated by having the structural engineer check the subgrade of the underpinning pits prior to concreting them.

Bracket Pile Underpinning

The use of bracket piles is limited to the underpinning of sidewalk vaults or exterior walls of lightly loaded buildings.

The maximum load generally used is in the range of 30 tons. A typical procedure would be to excavate and expose the top of the footing to be supported. It may be necessary to remove a portion of the footing to permit the pile to be driven as close to the wall as may be possible; decreasing the amount of eccentricity. The pile is usually driven to satisfy two criteriors; a minimum distance below the subgrade of the adjoining new construction and/or to a required resistance. A small pit is then excavated behind the pile and under the footing. The bracket is welded to the pile and wedges driven, completing the load transfer. The final step is the concrete encasement of the bracket. (see figure 10).

The installation time of this method is faster than either jacked piling or concrete pit underpinning. The unit cost per ton of supported load is usually much less than other systems. This system may be used through water bearing soils and can derive its support from stratas at considerable depths below the existing foundation.

There are however,many localities that do not permit this type of construction. It is often a requirement that all of the support system be confined within the property lines of the building being underpinned. The system may also encroach on the new construction and must then be incorporated within the perimeter foundation walls. This requires additional work in reinforcement placement and may also entail thickening the wall to provide sufficient cover at the beam locations.

The failures associated with this method are few, probably due to the light loads being supported. When failures do occur they are usually the result of not driving the soldier beam to proper resistance or insufficient welding between the bracket and the pile. Both of these cases would be eliminated with proper supervision and/or a proper design.

Pretested Jacked Pile Underpinning Installations

During the construction of the early subway systems in New York City, it was necessary to underpin buildings through a water bearing sand and silt material.

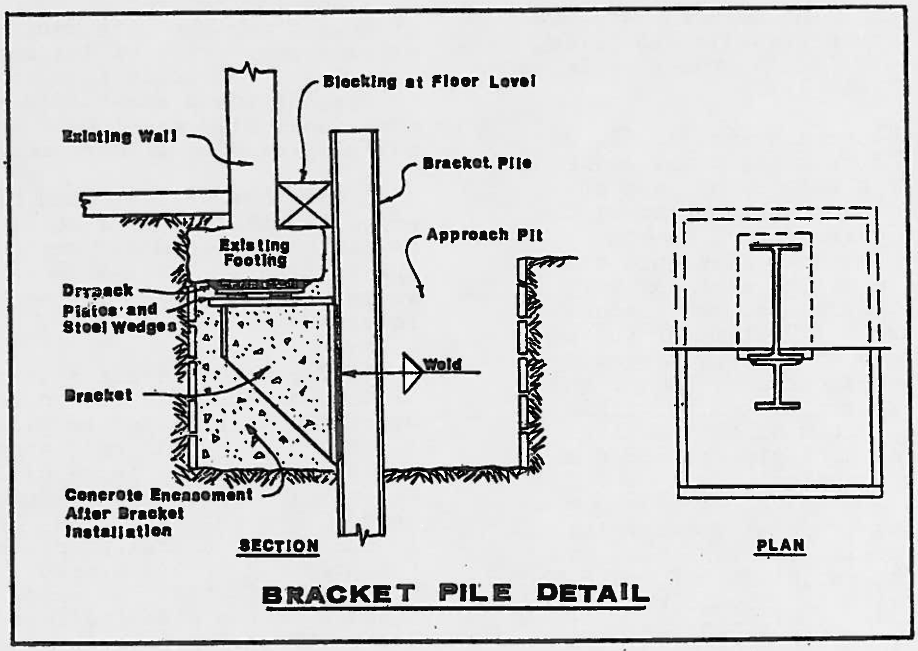


Figure 10

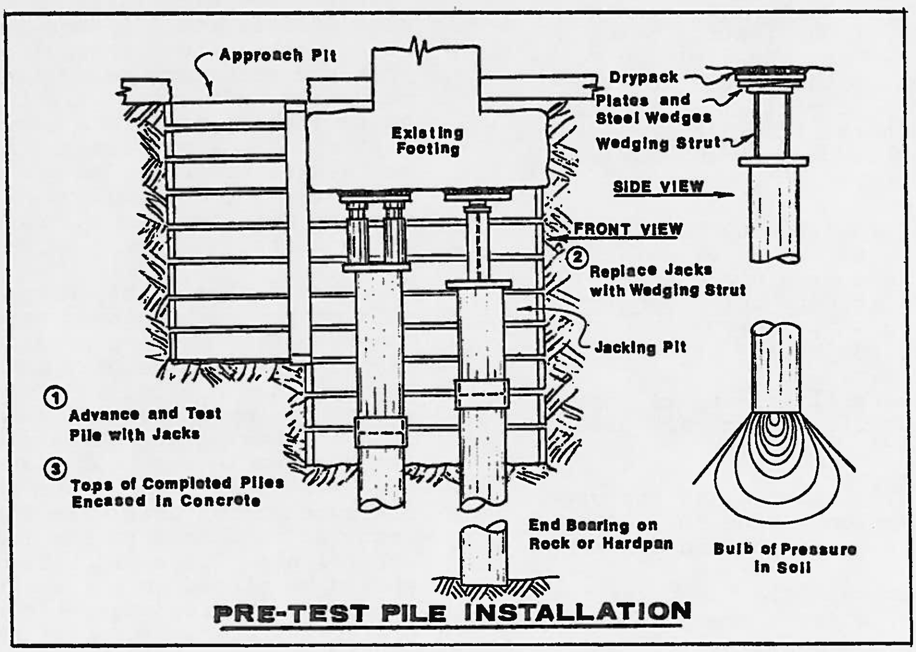


Figure 11

This material was fine grained and extremely difficult to dewater making it impossible to safely hand dig concrete underpinning pits. The solution was the application of the pretest jacked piling method. The installation procedure is as follows: (see Figure 11).

Excavate an approach and jacking pit to a minimum of 6 feet below the existing footing. Set up a section of pipe approximately 4' long on the bottom of the pit. Hydraulic jacks placed on top of the pile then thrust the pipe into the ground. The size of the pipe may vary but through the years most contractors have opted to use a 12" diameter section. The wall thickness would vary depending upon the load to be carried. After the first length has been jacked to within 6 inches of the bottom of the jacking pit, another length of pipe is added on top of the first section.

This coupling is usually accomplished with external jacking sleeves. The use of oakum caulking and bitumastic make the connection watertight. Should the jacked pile be required to resist any lateral loadings, the sleeve must be welded to both sections of pipe. An alternate method to the welded sleeve would be a full strength butt welded splice.

The second section is then jacked into the ground. Eventually the hydraulic force required to jack the pipe becomes excessive. The limiting force that can be used in the jacking operation depends on the weight of the structure and the characteristics of the footing being jacked against. Once the limiting force has been reached with no additional advance of the pipe piling, it is necessary to clean out the inside of the casing. The soil may be removed by any one of the following methods:

a. Pancake augers attached to 3/4" pipes and using universal joints.
b. Small hand or winch operated orange peel buckets built specifically for this purpose.
c. Controlled jetting or use of compressed air.
d. Continuous flight augers.
e. Churn drills as commonly used by well drillers.

After the pile has reached the predetermined elevation and/or the required capacity it is cleaned out and concreted.

The next day the pile may be pretested. Two hydraulic jacks are placed on top of the pile approximately 8" on centers and the required loading applied. The load is then maintained until no additional settlement of the pile takes place. It is common for the pile to settle in excess of 6" before the required resistance is obtained. The bearing value of the pile increases due to the bulbal pressure build up at the pile tip, (see Figure 11).

Should the pretest load be released at this time, the pile would rebound and a great proportion of its carrying capacity lost. It would then be necessary to reapply ~~the pretest~~ load resulting in additional pile settlement until the bulbal pressure was once more restored.

Once the pretest load has been properly maintained, a steel I-beam is cut to size and placed between the pile and the footing. The I-beam is securely wedged and only then is the pretest load released.

Before the pretest system was developed, it was the practice to remove the hydraulic jacks prior to placing and wedging the I-beam. At this stage the wedging could only exert a force of 10 tons $\pm$. The result was excessive building settlements. The success of the pretest method is due to the bulbal pressure build up at the pile tip. This pressure remains constant only if the load is maintained. Release of the pile loading directly affects the pile capacity and only additional settlement of the pile will increase its capability to sustain additional loadings.

The advantages of the pretest pile method includes the capability of the system to be installed through water bearing soil without the necessity of dewatering. The individual pile units are capable of supporting loads to 60 tons each in sand stratas and 100 tons or more if jacked to rock. The principal disadvantage is that the structure being jacked against must have sufficient weight so that adequate jacking reactions may be obtained without structural damage to the building being underpinned. The presence of boulders in the ground being jacked through will greatly increase the installation costs.

The failures that have occurred when this method was employed were largely due to inadequate bearing capacity of the completed piling. The pile may have been founded on a boulder in a compressible strata and not properly pretested. The load not having been maintained till no further settlement took place. There have also been cases where the jacking pressures being used were excessive and resulted in damage to the footing being underpinned. To avoid this, level marks should be placed on a structure and continuously observed during the jacking and pretesting operations. If the structure should start to raise, the jacking pressures must be lowered.

When many pretest piles are required under a footing, it is often necessary to apply the principles of group pretesting. Five piles pretested to 50 tons each do not necessarily equal a total load capacity of 250 tons. The pressure bulbs of

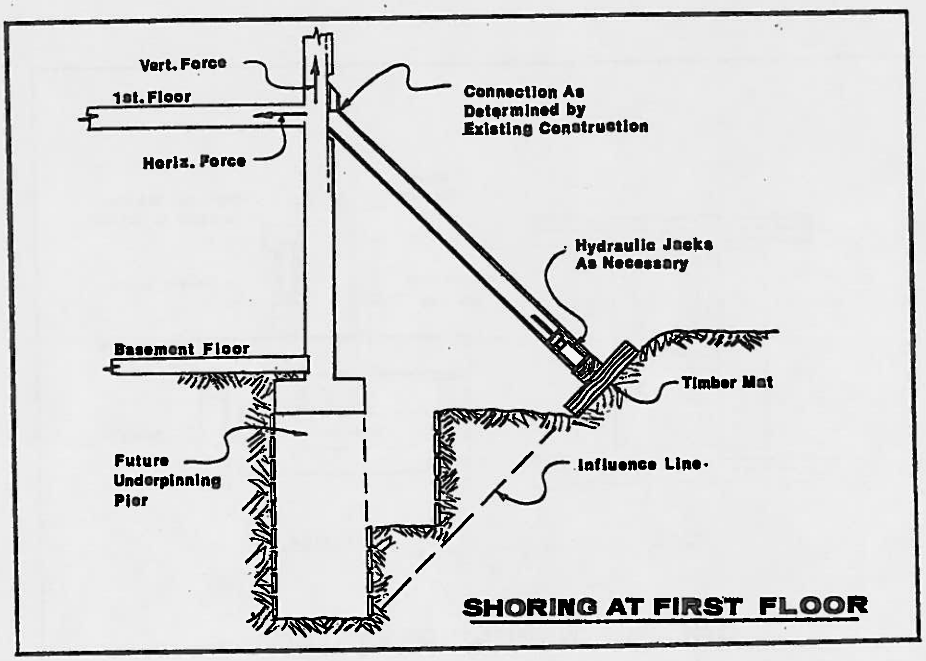


Figure 12

pile groups may overlap thereby diminishing the individual pile capacities. The correct procedure to be followed is to pretest and wedge off as many piles as possible simultaneously.

Column Shoring Systems

It is often necessary to pick up column loads directly. The loads may be too large to use the methods described previously for small isolated column footings. In other cases the existing footing may have to be completely removed and replaced with a new footing at a lower elevation.

There are two basic shoring methods commonly in use that permit the total removal of an existing column footing.

One method is the use of an inclined shore (see Figure 12). The shore is placed as near to vertical as possible taking into consideration the influence line of the excavation necessary to install the new column footing. The horizontal component of the shore must be resisted to prevent possible structural damage to the column. It is thus necessary to ensure that the point of contact between the shore and the column is made at a floor level so that the horizontal shore reaction is properly countered.

This method is mostly used when the underpinning operation may be completed quickly. Failures that have occurred were due primarily to the inadequacy or undermining of the shoring reaction mat. Should there be any question as to the adequacy of the soil beneath the mat to sustain the loading to be imposed, it is prudent to induce the loadings by using hydraulic jacks.

Another method more widely used then inclined shores would be the needle beam system (see Figure 13).

There are for all practical purposes no limitations on the column load that may be supported by a properly designed needle system. The ends of the needle beams may rest on piles, caisons, timber mats, underpinning piers or even other parts of the structure capable of carrying increased loads. The design and installation of the members that transfer the column load to the needling system are of prime importance. The columns may be composed of steel, concrete or cast-iron with each material requiring a different method to properly effect the load transfer (see Figure 14).

It is also necessary to compute the needle beam deflections under the final load transfer. The beams must be predeflected prior to severing the column connection to the existing footing. This is best done by the use of hydraulic jacks or with lighter loads, the use of plates and steel wedges could suffice.

There have been relatively few failures using this system. Due to the high loadings involved a proper needling system must be designed and supervised by an engineer experienced in this field. The failures that have occurred have been mainly due to not pre-deflecting long needle beams and upon freeing the column excessive movements have resulted.

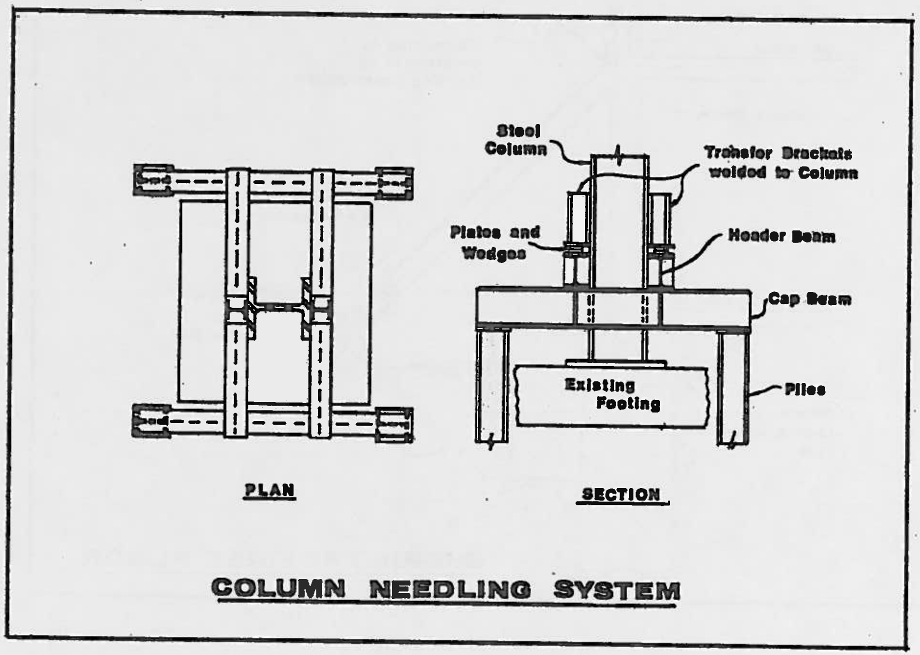


Figure 13

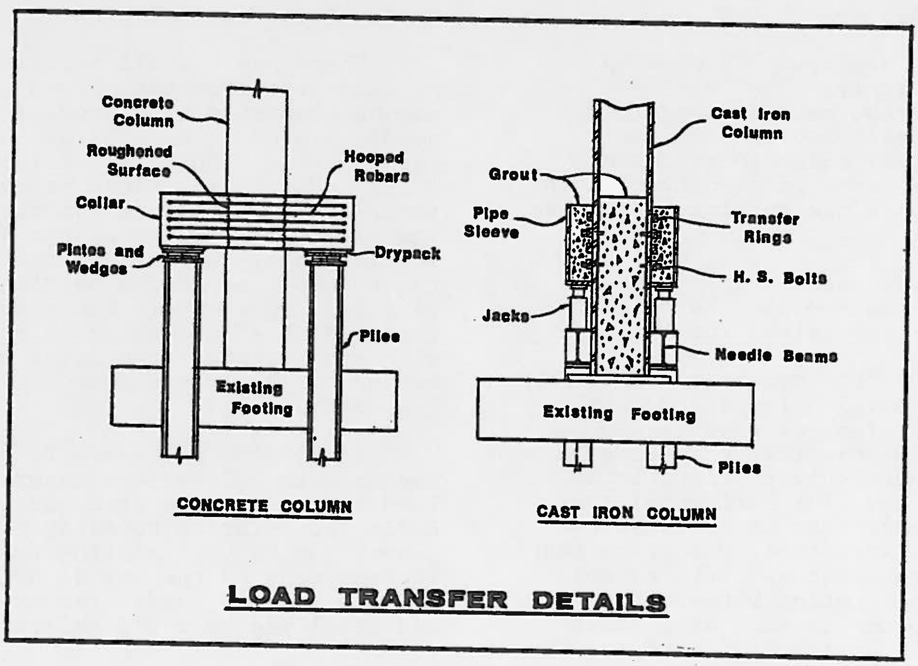


Figure 14

Conclusion

All firms that specialize in the subsurface construction are aware of the one basic truth associated with this type of work; the unexpected is to be expected. Rock occurs twenty feet above the anticipated elevation; firm impervious soils dissappear and are replaced by loose gravelly water bearing stratas. The project that started using concrete underpinning pits may be changed to a jacked pile installation to eliminate what might be a very expensive dewatering operation. The reverse is also true; the presence of boulders may result in the use of concrete pits in lieu of jacked piling. Existing wall and/or column footings may not be sized or located as shown on the contract drawings.

The above cases are not rare occurences. Rather it is rare to underpin an old structure without altering a preconceived underpinning scheme. It is imperative therefore that all underpinning be designed by engineers experienced in the field and installed by speciality underpinning contractors.

The contractor must be able to cope with the changed conditions encountered. A contractor experienced only in the use of one method may not be able to competently adjust to a different system if required.

Owners and architects always lament monies expended in underpinning installations. The results are hidden; there is no impressive facade or additional rental space resulting from such efforts.

Underpinning is of course only specified when absolutely necessary. It usually is one of the first operations that take place at the job site. The timely completion date of this phase of the work is imperative as it effects the excavation schedule and therefore the final completion date.

It is prudent therefore, that the choice of an underpinning firm not depend solely on the "low price" but greater weight be given to the contractor better equipped to cope with the unexpected.

APPENDIX

PAST PROCEEDINGS
OF THE
OHIO RIVER VALLEY SOILS SEMINARS

ORVSS I
BUILDING FOUNDATION DESIGN AND CONSTRUCTION

October 16, 1970
Lexington, Kentucky

AND WHY DO WE DO IT THIS WAY? by R. C. Deen, Assistant Director of Research, Kentucky Bureau of Highways.

A CASE STUDY - PIPE PILING FOR A DEEP FOUNDATION PROBLEM by Don Fuller, P.E., L.S., Fuller, Mossbarger and Scott Civil Engineers, Inc.

DESIGN, CONSTRUCTION AND PERFORMANCE OF SPREAD FOOTINGS ON A NON-UNIFORM FOUNDATION by Robert Lennertz, M. ASCE, The H. C. Nutting Company.

DESIGN, CONSTRUCTION AND TESTING OF DRILLED PIERS ON SHALE AND THINLY BEDDED LIMESTONE by George J. Thelen, P.E.

BUILDING FOUNDATION FAILURE RELATED TO SOIL SHRINKAGE by Aubrey D. May, Chief Engineer, Stokley and Associates.

INNOVATIONS IN FOUNDATION DESIGN AND CONSTRUCTION by D. J. Hagerty, Assistant Professor, Civil and Environmental Engineering Department, University of Louisville.

ORVSS II
EARTHWORK ENGINEERING, START TO FINISH

October 15, 1971
Louisville, Kentucky

PRELIMINARY SITE SELECTION STUDIES WITH REMOTE SENSING TECHNIQUES by David J. Barr, Department of Civil Engineering, University of Cincinnati.

SITE EVALUATION WITH REMOTE SENSING TECHNIQUES APPLIED TO SUSPECT KARST TOPOGRAPHY by Melville D. Hensey, Engineering Division, The Proctor and Gamble Company.

EXPLORATION METHODS by Dr. Vincent P. Drnevich, P.E., Assistant Professor of Civil Engineering, University of Kentucky.

THE SELECTION OF STRENGTH PARAMETERS AND THEIR USE IN THE STABILITY ANALYSIS OF EARTH ROCK EMBANKMENTS by Woody McGraw and Don Tupman, Louisville District, U.S. Army Corps of Engineers.

CONTRACTOR'S PROBLEMS by Richard Geottle.

ENVIRONMENTAL CONSIDERATIONS IN SOILS ENGINEERING by D. Joseph Hagerty, Asst. Professor of Civil Engineering, University of Louisville and Joseph L. Pavoni, Asst. Professor of Civil and Environmental Engineering, University of Louisville.

ORVSS III
LATERAL EARTH PRESSURES

October 27, 1972
Fort Mitchell, Kentucky

INSTRUMENTATION FOR STRESS AND STRAIN MEASUREMENTS IN SOILS by Stanley D. Wilson, Executive Vice-President, Shannon and Wilson, Inc.

DEEP EXCAVATION AND BUILDINGS SUPPORTED BY TIE-BACK SYSTEM by Richard Geottle, P.E., Richard Geottle, Inc., James J. Flaig, P.E., H. C. Nutting Company, Arthur J. Miller, P.E., Miller, Tallarico, McNinch, and Hoeffel Consulting Engineers, and Steven E. Schaefer, EIT, Miller, Tallarico, McNinch, and Hoeffel Consulting Engineers.

LATERAL PRESSURES AND MOVEMENTS CAUSED BY PILE DRIVING by D. Joseph Hagerty, A.M. ASCE, Assistant Professor of Civil Engineering, University of Louisville.

FACTORS TO BE CONSIDERED IN THE DESIGN OF BACKFILL FOR LARGE DIAMETER FLEXIBLE METAL CONDUITS by David C. Cowherd, M.S., P.E.

GENERALIZED SLIDING WEDGE METHOD FOR SLOPE STABILITY AND EARTH PRESSURES ANALYSIS by Dr. Vincent P. Drnevich, Assistant Professor of Civil Engineering, University of Kentucky.

LATERAL PRESSURES AND PRESTRESSED TIE-BACK WALLS by P. C. Rizzo and R. D. Ellison, E. D'Appolonia Consulting Engineers, Inc., and R. J. Shafer, Herbert F. Darling, Inc.

DESIGN AND CONSTRUCTION OF SUPPORTED TEMPORARY EXCAVATIONS IN URBAN ENVIRONMENT by Yves Lacroix and Walter T. Jackson, Woodward-Moorhouse and Associates, Inc.

ORVSS IV
GEOTECHNICS IN TRANSPORTATION ENGINEERING

October 5, 1973
Lexington, Kentucky

RETRIEVAL AND USE OF GEOTECHNICAL INFORMATION by D. Joseph Hagerty, Assistant Professor of Civil Engineering, University of Louisville, Nicholas G. Schmitt, Soils Engineer, Law Engineering Testing, William J. Pfalzer, Research Engineer Assistant, Kentucky Bureau of Highways.

IN SITU SHEAR STRENGTH PARAMETERS BY DUTCH CONE PENETRATION TESTS by C. T. Gorman, Research Engineer Assistant, Kentucky Bureau of Highways, T. C. Hopkins, Research Engineer Principal, Kentucky Bureau of Highways, and V. P. Drnevich, Associate Professor of Civil Engineering, University of Kentucky.

REINFORCED EARTH - DESIGN AND CONSTRUCTION by David S. Gedney, Regional Engineer, Region 15, Federal Highway Administration and John L. Walkinshaw, Highway Engineer, Federal Highway Administration.

APPLICATIONS OF THE FINITE ELEMENT METHOD TO GEOTECHNICS AND TRANSPORTATION ENGINEERING by Yang H. Huang, Associate Professor of Civil Engineering, University of Kentucky.

RELIABILITY ANALYSIS AND COST OPTIMIZATION OF EMBANKMENTS by Warren J. Baker, Chrysler Professor and Dean, College of Engineering, University of Detroit.

STABILITY ANALYSIS - ITS POTENTIAL AND ITS LIMITATIONS by T. H. Wu, Professor of Civil Engineering, The Ohio State University.

ORVSS V
ROCK ENGINEERING

October 18, 1974
Clarksville, Indiana

EXPLORATION AND TESTING IN ROCK by Joseph Hagerty, Civil Engineering Department, University of Louisville, and James Coulson, Tennessee Valley Authority.

A ROCK CLASSIFICATION SCHEMA by R. C. Deen, Assistant Director, Kentucky Bureau of Highways, Division of Research, C. D. Tockstein, Research Engineer Assistant, Division of Research, and M. W. Palmer, Research Engineer Assistant, Division of Research.

FOUNDATIONS ON OR IN ROCK by E. D'Appolonia, E. D'Appolonia Consulting Engineers, Inc.

ROCK FOUNDATIONS (A Panel Discussion) Comments by: C. R. Lennertz, H. C. Nutting Company, Harry Thomas, US Army Corps of Engineers, and Milton M. Greenbaum, Greenbaum and Associates.

DESIGN OF OPEN EXCAVATIONS IN ROCK by D. Ross-Brown Dames & Moore.

ROCK EXCAVATION (A Panel Discussion) The Producer's Viewpoint by J. A. Waddell, Martin Marietta Aggregates, Consideration of Effects on Surroundings by D. J. Hagerty, University of Louisville, and Geologic and Support Considerations for Rock Excavations by J. Mahar, University of Illinois.

ROCK ENGINEERING ON SOME RECENT PROJECTS by Don U. Deere, Consultant in Engineering Geology and Rock Mechanics, University of Florida.

ORVSS VI
SLOPE STABILITY & LANDSLIDES

October 17, 1975
Fort Mitchell, Kentucky

QRVSS VII
SHALES AND MINE WASTES: GEOTECHNICAL PROPERTIES, DESIGN, AND CONSTRUCTION

October 8, 1977
Lexington, Kentucky

CONVENTIONAL AND UNCONVENTIONAL APPROACHES USED TO LOCATE AND ELIMINATE HAZARDOUS MINE WASTE DAMS IN OHIO by V. V. Rajadhyaksha, Ohio Department of Natural Resources

GEOTECHNICAL OVERSIGHT NULLIFIES PROPER PROCEDURES by B. Rosen, Marks-Rosen, Inc.

WEST VIRGINIA EXPERIENCES WITH REVIEW OF COAL REFUSE DISPOSAL FACILITIES by G. Hall and M. K. Robinson, West Virginia Coal Refuse and Dam Control Section, and R. E. Smith, Woodward-Clyde Consultants

CLOSED CIRCUIT COAL REFUSE AS A STRUCTURAL FILL by D. C. Cowherd, Bowser-Morner Testing Laboratories, Inc.

DYNAMIC DESIGN CONSIDERATIONS OF LOOSE FINE COAL REFUSE by R. D. Ellison and Y. Y. Cho, E. D'Appolonia Consulting Engineers, Inc.

PROPERTIES OF COAL MINE FLOOR SHALE by R. W. Stephenson and J. D. Rockaway, University of Missouri-Rolla

ORVSS VIII
EARTH DAMS AND EMBANKMENTS
DESIGN, CONSTRUCTION, PERFORMANCE

October 14, 1977
Louisville, Kentucky

SIGNIFICANT ENGINEERING-GEOLOGY FEATURES AT DAMSITES IN FLAT-LYING SEDIMENTARY ROCKS by Alberto S. Nieto, Department of Geology, University of Illinois at Urbana-Champaign, Urbana, Illinois

SMALL DAMS - PARTICULAR PROBLEMS AND CONSIDERATIONS by Walter E. Hanson and David E. Daniels, Hanson Engineers, Inc., Springfield, Illinois

FOUNDATION SEEPAGE PROBLEMS AT WOLF CREEK DAM by Frank B. Couch, Jr., Nashville District, U.S. Army Corps of Engineers, Nashville, Tennessee

MOVEMENTS OF A NATURAL SLOPE AND AN EMBANKMENT - TWO CASE HISTORIES by E. C. Palmer, W. L. Nuzzo, Richard G. Almes, Richard Brissette, and E. D'Appolonia, E. D'Appolonia Consulting Engineers, Inc., Pittsburgh, Pennsylvania

COMMENTS ON SEISMIC STABILITY EVALUATION OF EMBANKMENT DAMS by Gonzalo Castro, Geotechnical Engineers, Inc., Winchester, Massachusetts

ORVSS IX
DEEP FOUNDATIONS

October 27, 1978
Fort Mitchell, Kentucky

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