

SLURRY WALLS AS AN INTEGRAL PART OF UNDERGROUND TRANSPORTATION STRUCTURES

November 1981
Final Report



Prepared for



U.S. Department of Transportation

Federal Highway Administration

Offices of Research & Development
Structures and Applied Mechanics Division
Washington, D.C. 20590

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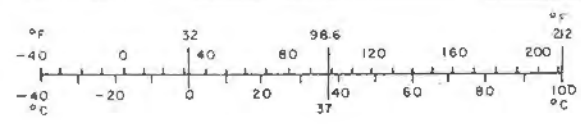
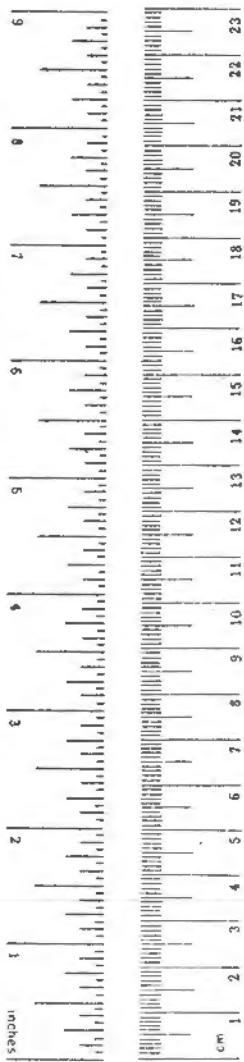
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
LENGTH				
in	inches	2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
AREA				
in ²	square inches	6.5	square centimeters	cm ²
ft ²	square feet	0.09	square meters	m ²
yd ²	square yards	0.8	square meters	m ²
mi ²	square miles	2.6	square kilometers	km ²
	acres	0.4	hectares	ha
MASS (weight)				
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
VOLUME				
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft ³	cubic feet	0.03	cubic meters	m ³
yd ³	cubic yards	0.76	cubic meters	m ³
TEMPERATURE (exact)				
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
LENGTH				
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
AREA				
cm ²	square centimeters	0.16	square inches	in ²
m ²	square meters	1.2	square yards	yd ²
km ²	square kilometers	0.4	square miles	mi ²
ha	hectares (10,000 m ²)	2.5	acres	
MASS (weight)				
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	st
VOLUME				
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m ³	cubic meters	35	cubic feet	ft ³
m ³	cubic meters	1.3	cubic yards	yd ³
TEMPERATURE (exact)				
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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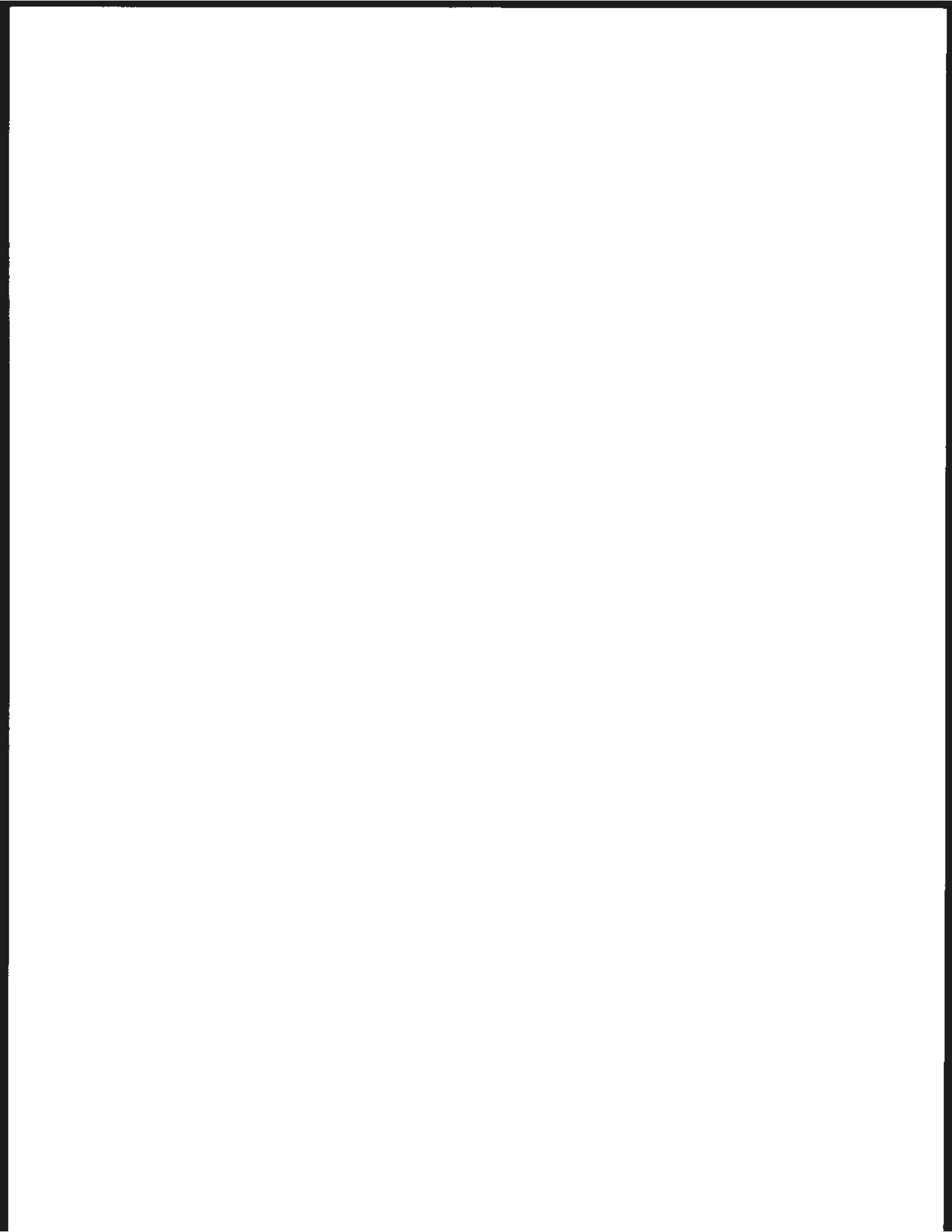
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PART I. SLURRY WALL: GENERAL

CHAPTER 1

PREVIEW OF SLURRY WALL CONSTRUCTION

1.1 Historical Development

The use of soil-laden fluid in earth excavation dates back to the turn of this century. In 1901, a 984 ft (300 m) deep hole was dug through soft soil formations off the Gulf Coast of the U. S. using this technique. The fluid, known as the "mud," was said to have been produced by driving cattle through a pit. Later, the method was used in drilling oil wells and a major advancement of this technique was the introduction of bentonite as an admixture to the drilling fluid in the U. S. in 1929. The present day concept of the application of soil-laden fluids in civil engineering structures was first conceived by C. Veder of Milan, Italy in 1938.

Although the first fluid-supported trench was tested in 1948 in Italy, the first large scale use of the fluid trench technique was undertaken in the United States in 1948 at Terminal Island, Long Beach, California, where construction involved a trench excavated to a depth of 45 ft (13.7 m) having a nominal width of 2.7 ft (0.8 m), and a length of about 12,000 ft. (3659 m). The trench surrounded the Southern California Edison Power Plant which was located in a zone of intense settlement associated with petroleum mining in the surrounding areas. The main function of the cut-off wall was to isolate the plant site from the surrounding areas so that the ground water level within the wall could be kept at a constant level, thus reducing the expected settlement around the plant. This construction involved only a "slurry trench," since no concrete was actually poured. The trench was excavated and backfilled with bentonite and soil mixture, which was combined with the ground water to form an impervious diaphragm.

The first of the "slurry wall" technique whereby an actual concrete wall was built in a fluid-supported trench took place almost simultaneously in 1950 at two different sites in Italy. The two sites, the Santa Maria Dam in Dolomites, Italy, and the compensation basin and intake structures across the Volturno River for an electric company near Venafro, Italy, required impervious cutoff walls. At the first site, the wall was constructed underneath the main embankment of the dam extending some 130 ft (40 m) in deposits of sand and gravels with some boulders. The second site involved the construction of a wall approximately 115 ft (35 m) deep through pervious alluvial layers of soil.

In addition to the expected seepage control provided by the walls at these two sites, the following important features were noted:

- a. The walls were strong enough to carry loads without excessive lateral deformations.
- b. Construction was performed through treacherous soil deposits, but the walls were embedded firmly into the underlying bedrock.
- c. The excavations were performed successfully and rapidly with specially developed trenching equipment.
- d. The wall was placed within a close tolerance.

- e. The slurry wall technique was economically feasible. The alternatives using either sheet-piling and timbering or open-cut operations were rejected because of costs.

After its successful application in projects in Italy, the slurry wall construction method spread to other European countries and eventually around the world. Construction took place in the area of hydroelectric works and dams, followed by the subsequent realization that slurry walls could be used in the areas of deep excavation for building foundations, roadways, and transportation facilities involving tunneling. By the early 1960's, the slurry wall technique had been used in a variety of construction projects in such countries as France, England, the United States, Canada, Japan, Venezuela, and the British Crown Colony of Hong Kong.

1.2 Evolution of Slurry Wall Panels

Prior to 1934, slurry walls were built in terms of a series of piles constructed in slurry supported bore-holes. The piles were placed so that each was in contact with the next and, therefore, the structures became known as "tangent pile" walls.

Effort in providing a watertight joint between the piles, necessary for ground water control, led to the following modifications of the tangent pile system. A sequence of piles was placed with a spacing a few inches less than the pile diameter. After concrete was set, holes were drilled between the initial piles, and a concave arc was cut into each initial pile. Then concrete was placed into the holes to form the intermediate piles. This method was known as "secant piling." Further development led to an interlocking pile system in which initial piles were spaced further apart, and the soil between had to be excavated with digging tools rather than by boring. In today's practice, the initial piles have been eliminated, and the entire wall is excavated in progressive design lengths, resulting in a very economical construction. This is referred to as an interlocking panel arrangement.

Interlocking panel slurry walls can be classified into two main categories:

- a. The wall panels are cast-in-place.
- b. The wall panels are precast and assembled in the trench.

Cast-in-place walls are generally constructed by one of two methods. The first method involves excavating a panel trench, inserting circular end pipes whose diameters match the width at each end of the trench, placing a reinforced steel cage in the trench, and then filling the trench with tremie concrete. This method uses a joint system usually referred to as an "end pipe" system. The end pipes are removed shortly after the concrete begins to set so that the next panel can be cast. Figure 1 shows this panel casting method, and a further discussion of the end pipe joint is given in Chapter 4.3.

The second method of cast-in-place slurry walls is commonly known as the "steel beam and concrete" technique, in which structural steel H-beams are used in lieu of the end pipes. Figures 2a and 2b show plan views of slurry wall using the end pipes and steel beams, respectively. It is interesting to note that in the steel beam method the H-beams were left in the wall during the construction and serve as a shear-connector. For the end pipe method, the continuity between the panels is not easily achieved and can be done only at a great cost.

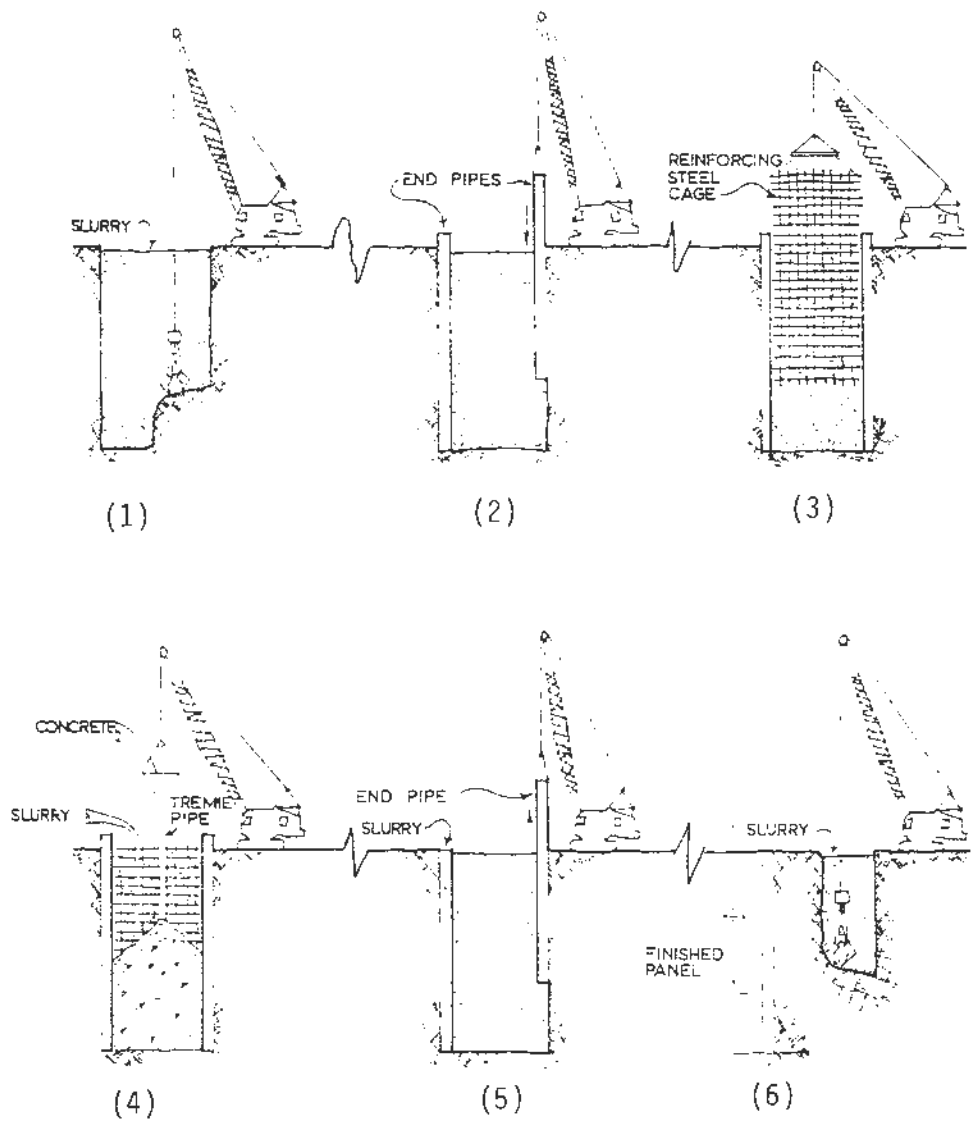
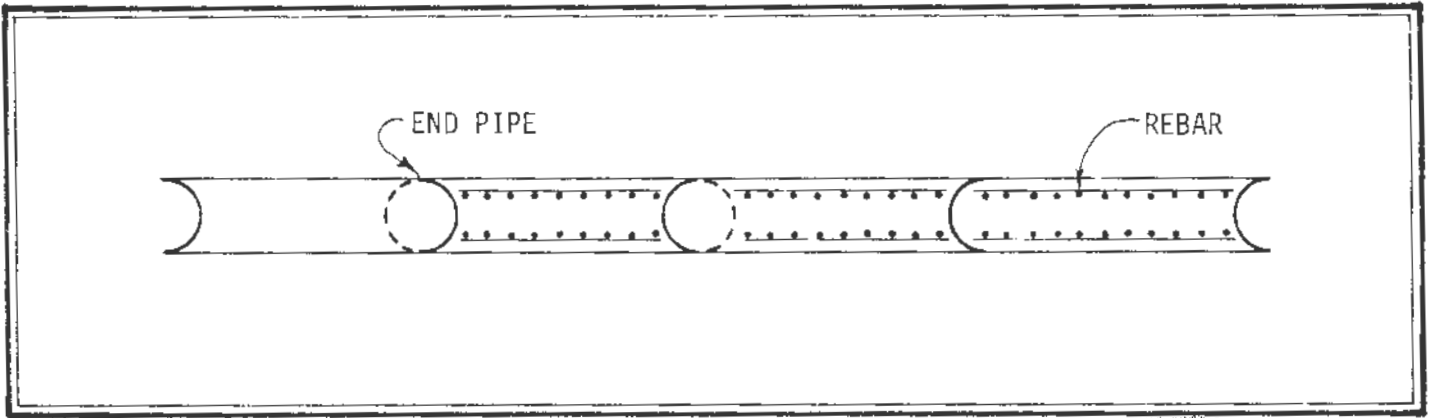
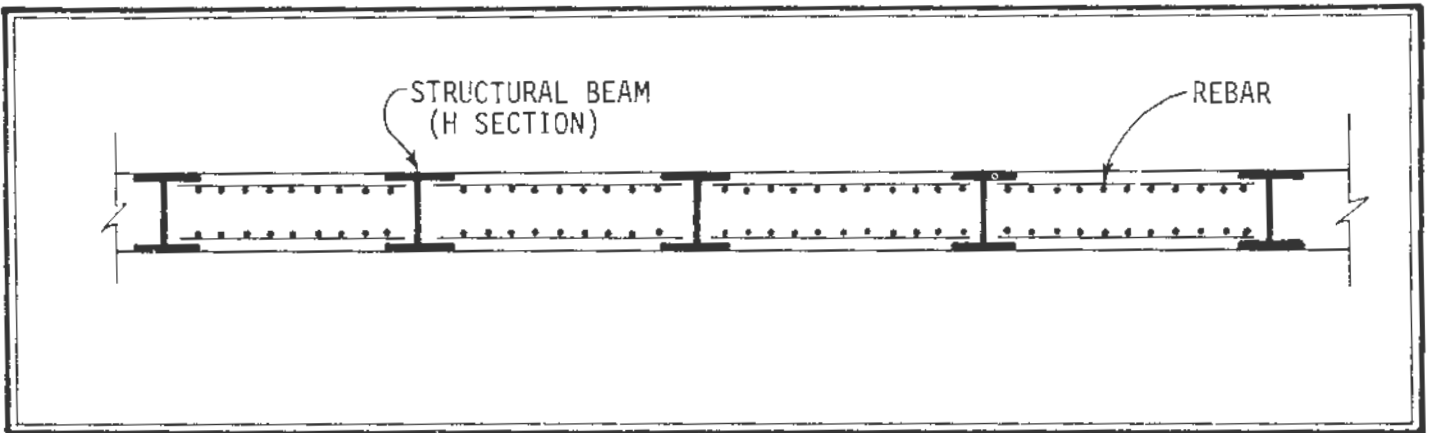


Figure 1. Steps in Constructing Cast-In-Place Panels



a. Plan View Of Slurry Wall Using End Pipe Method To Form Panel Joint



b. Plan View Of Steel Beam And Concrete Slurry Wall

Figure 2. Plan Views Of Typical Slurry Wall Cast-In-Place Interlocking Panels

In precast panels there must be enough room for panel fabrication near the construction site. It is generally preferable to cast the panels on the site since their great size and weight make transporting them difficult. It should also be noted that the finished face of the precast slurry wall will be smooth, and it may be somewhat easier to control the location of block-outs for floor and ceiling slab connections. Also, during lifting operations, care must be exercised so that panels are not bent, thereby avoiding the induction of any tension in the panel which would cause cracks. Figure 3 depicts two ways of using precast panels.

The methods employed in producing each of these walls differ only in trench geometry and casting procedures. This report will focus on the cast-in-place, interlocking panel slurry wall. Further information on other wall types can be obtained from the references presented or through contact with contractors who provide such wall systems.

1.3 Recent Trends

Slurry wall construction has found great applicability in congested urban areas, historically sensitive neighborhoods where there is a desire to preserve traditional structures, and in countries where other ground support systems were costly due to lack of material such as wood. Thus, most of the technological advancements in construction methodology and equipment over the past 30 years have taken place in Europe and Japan where populations, urban congestion, and, to some extent, lack of raw materials have forced the development and use of alternative and economical construction techniques.

These techniques instigated the development of special trenching and drilling equipment, new design approaches, and greater wall applicability. Wall widths and depths have increased to meet new design requirements. While the first wall excavations were limited to less than 150 ft (46 m) in depth, new equipment allows walls to exceed this level. In Manicouagan-3 Dam at Quebec, Canada, a slurry wall, made of 24 in. diameter pile group as part of a secant pile wall, was used to seal the foundation of a 350 ft high earth dam to stop water seepage. Each pile was drilled to a depth of 430 ft below the water level which set a new record. It was an impressive accomplishment since the drilled hole was only a maximum one inch out-of-plumb for the entire length, and the tolerance of the drilled depth was only six inches.*

The development of the slurry wall technique is considered one of the most significant advances in construction methods in recent years. In addition to offering an excellent barrier against horizontal ground water seepage, properly braced slurry walls reduce ground movement, as well as the settlement of structures in the soil mass adjacent to the support wall. These factors often provide tremendous savings in time and money by reducing the need for site dewatering and often eliminating, or at least minimizing, the need for underpinning structures adjacent to the construction excavation. Other side benefits are realized when employing slurry wall construction in urban areas. For example, in cut-and-cover tunneling used for subway construction, the slurry wall technique can minimize traffic disruption, and considerably reduce noise and vibration in the construction corridor. Also, where slurry walls have been

*Reported in Engineering News Record, January 6, 1972.

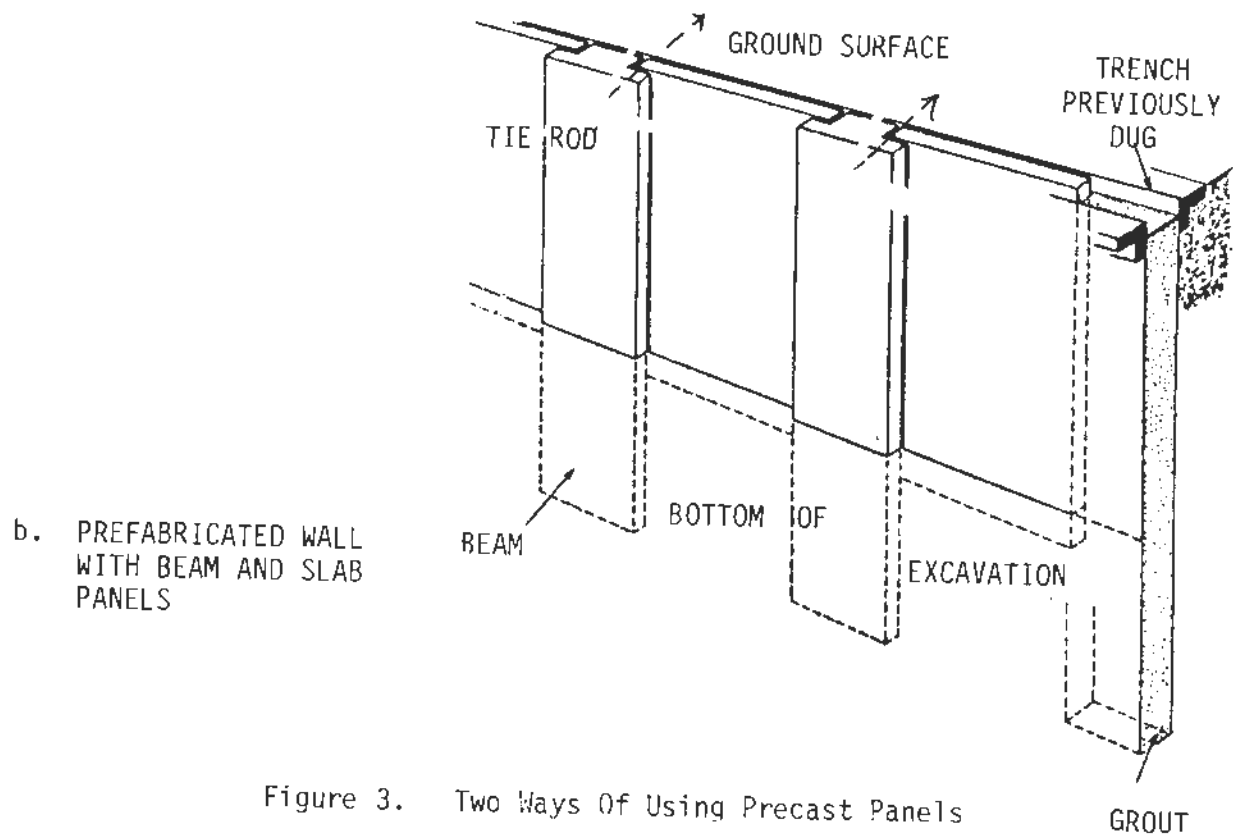
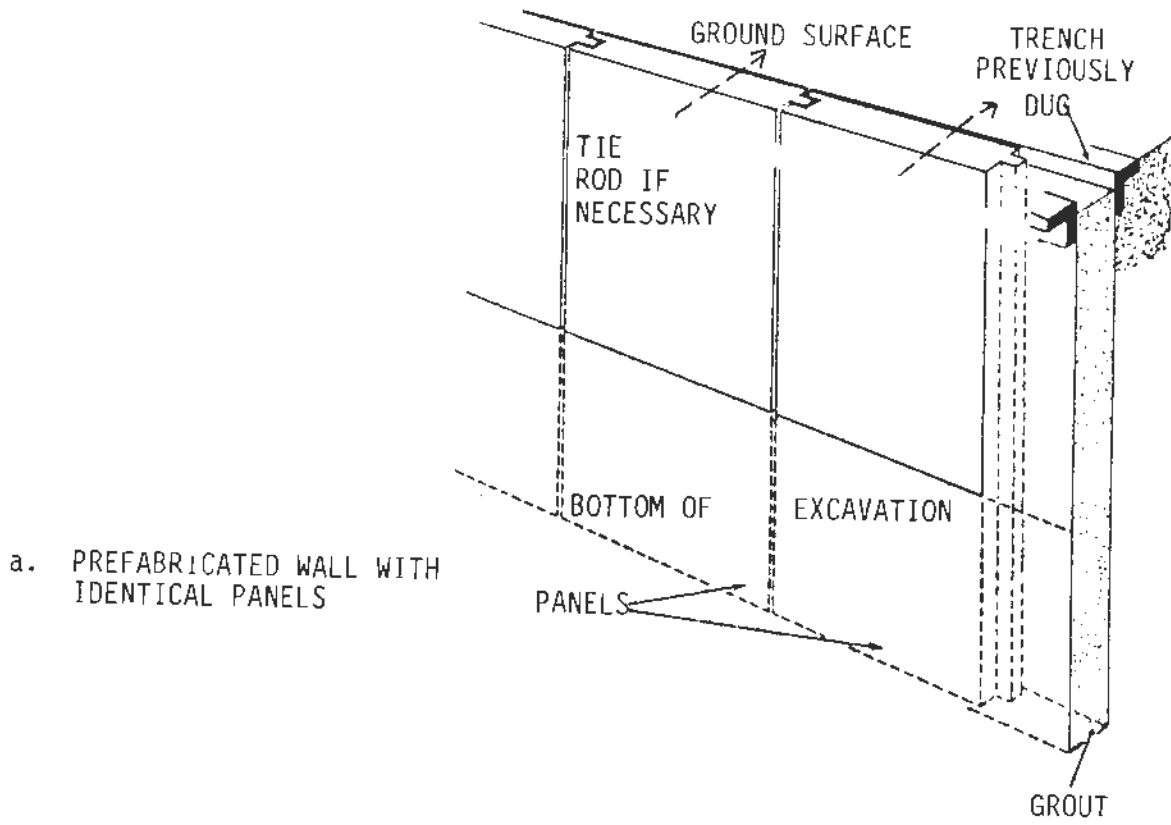


Figure 3. Two Ways Of Using Precast Panels

used as part of the final structure in subway work, they have reduced the excavation width and right of way.

Probably one of the most attractive factors in the use of slurry wall construction is its economic advantages in comparison with other construction techniques. In some cases, this has meant more than a 25 percent cost savings over the bid of the nearest competitor. For instance, the Army Corps of Engr. employed slurry walls as an impervious cutoff wall, which permitted dry foundation excavation for a large pumping station in Memphis, Tennessee. They saved a million dollars (more than 25%) in construction costs utilizing this method.* The wall also protected the completed structure from high uplift pressures. In another instance, actual bid documents showed that the slurry wall technique saved more than \$2 million in underpinning costs on the construction of the Washington Metropolitan Area Transit Authority (WMATA) Federal Center, S.W., Subway Station in Washington, D.C.**

With the ever increasing development of large metropolitan areas in the United States, the use of slurry walls for deep foundation work, as well as cut-and-cover tunneling projects, provides a viable alternative to conventional construction practices. Although slurry wall construction is not new to the United States, the technique is neither widely known nor used. As with most innovative concepts in any industry, acceptance is hindered by a reluctance to deviate from well known, standard methods. This attitude, however, is changing because of the complex nature of construction endeavors in congested urban areas. On larger projects, such as the subway construction projects which have been completed or are currently underway in areas other than Washington, D.C., (such as San Francisco, California; New York, New York; Baltimore, Maryland; and Atlanta, Georgia) the alternative of using slurry walls along with conventional techniques has been investigated. In fact, the Massachusetts Bay Transit Authority (MBTA) Red Line Extension in Boston, Massachusetts, calls for extensive use of slurry walls as an integral part of the permanent subway structure because of requirements to minimize ground disturbance in historically sensitive neighborhoods such as Harvard Square.

Research efforts continue to advance the state-of-the-art of slurry wall construction. There is still much to learn. The Federal Highway Administration of the U.S. Department of Transportation is involved in this research and is presently sponsoring detailed instrumentation efforts to monitor wall construction and performance along with the ground movement and building settlements involved in the construction of Boston's Harvard Square Subway Station. As designers and contractors understand and become more knowledgeable about the slurry wall process, its implementation as a permanent part of transportation facilities (or structures) should become commonplace.

* Reported in Engineering News Record, November 4, 1971, p.32.

** Based on final bid documents - lowest bidder, 1972.

CHAPTER 2

PHYSICAL, ENVIRONMENTAL AND ECONOMIC PROFILES

2.1 Physical Profile

Rapid transit systems, automobile parking garages and highway underpasses are examples of underground transportation-related facilities in which slurry walls have been used as permanent structures (see Figure 4). These structures are usually constructed in congested metropolitan areas where land costs are high and functional space is at a premium. For such sites, rigid slurry walls often provide a viable construction alternative to flexible systems of steel sheet piling or soldier piling and lagging.

Another physical advantage of slurry walls is that, with proper construction procedures, the walls can be used in a wide variety of soil conditions. Walls can be and have been successfully fabricated in sandy granular soils, in clayey soils, and even in soils with gravel or boulders. The case studies at the end of Part 1 of this report demonstrate this fact. In Case 1 soil conditions in San Francisco, California, consisted of sand and silty clay. In Case 2 the old shoreline of Mission Bay had a naturally high ground water table. In Case 3, the placement of a wall in a zone of plastic clay was required in Washington, D.C. In Case 4, soil consisting of coarse sand and small gravel was encountered in the Baltimore, Maryland.

For nontransportation-related projects, slurry walls have been used for deep foundations of buildings located adjacent to harbor operations having extremely high water pressures, e.g., the New World Centre in Hong Kong (see Figure 5). Slurry walls have also been used when construction of the walls has had to proceed through solid rock pockets or large boulders. All of these cases illustrate the point that it is possible to construct slurry wall panels under a variety of soil conditions.

The construction process of slurry wall panels allows their width, depth, and length to be adjusted in order to meet specific site design parameters. Panel width and depth are determined by: (1) site conditions; (2) structural requirements; and (3) design criteria. The length of the panels is dictated by: (1) width and depth; (2) presence of ground water and soil conditions; (3) surcharge loads; and (4) practical field considerations.

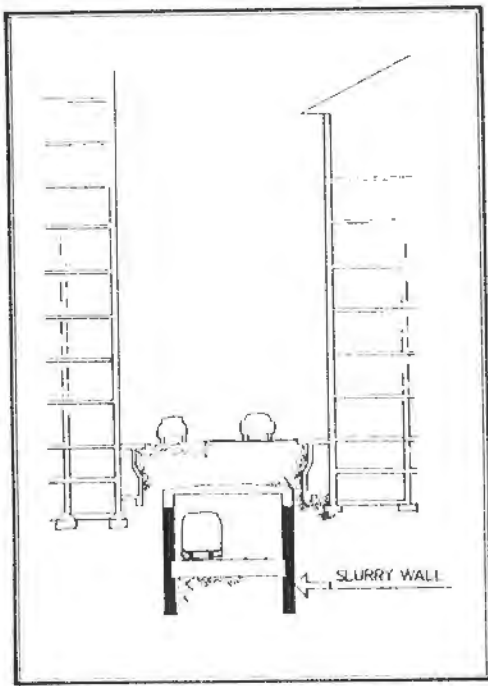
Designers and contractors have also employed panels of various geometric configurations to meet special design criteria of any particular sites. Figure 6 shows, in cross sections, a variety of geometrical shapes that have been employed in sites throughout the world. Figures 7a and 7b show actual panel shapes being fabricated in the field.

The following sections provide some advantages of slurry walls in terms of engineering, environmental and economic standpoints.

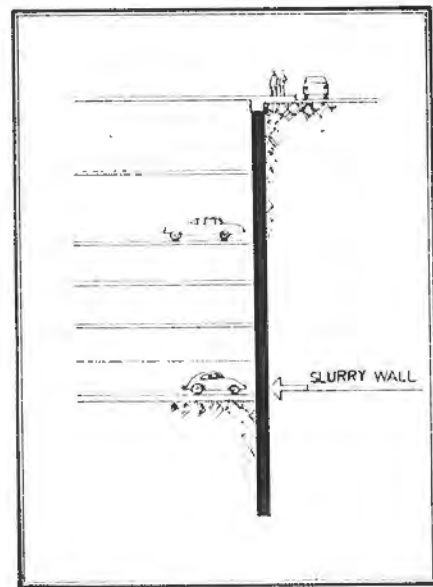
2.2 Environmental Profile

A. Minimal Vibrational Disturbance

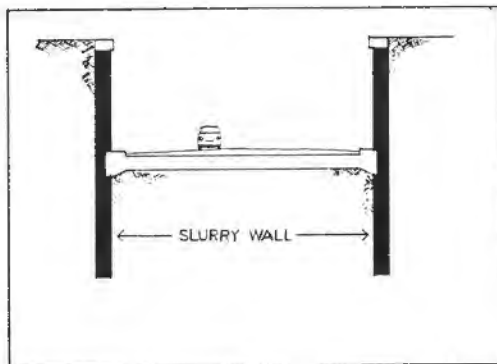
The installation of steel sheet pile walls or soldier pile and lagging



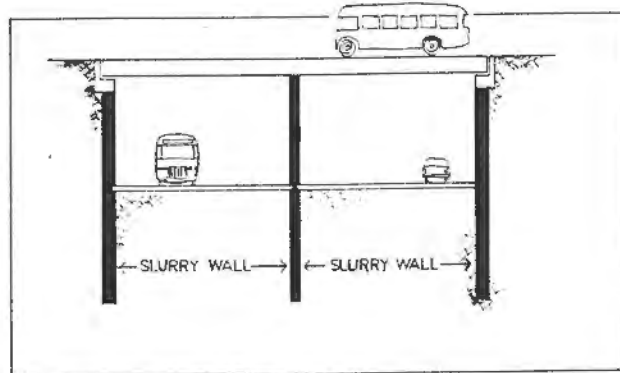
a. SUBWAYS



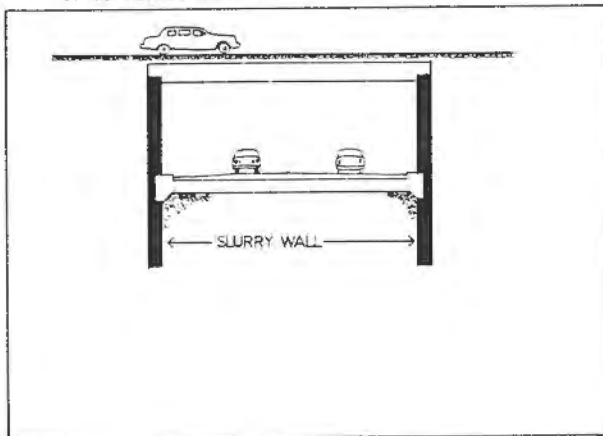
b. UNDERGROUND PARKING FACILITIES



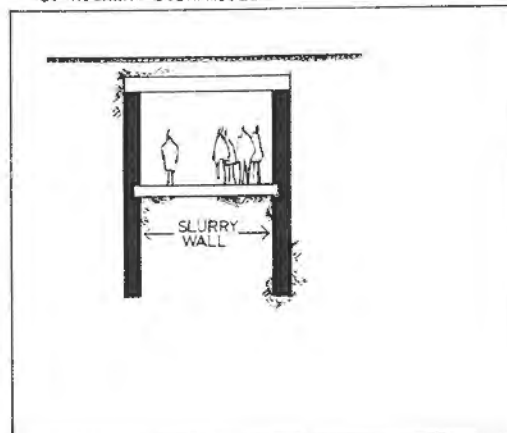
c. DEPRESSED HIGHWAYS



d. HIGHWAY OVERPASSES

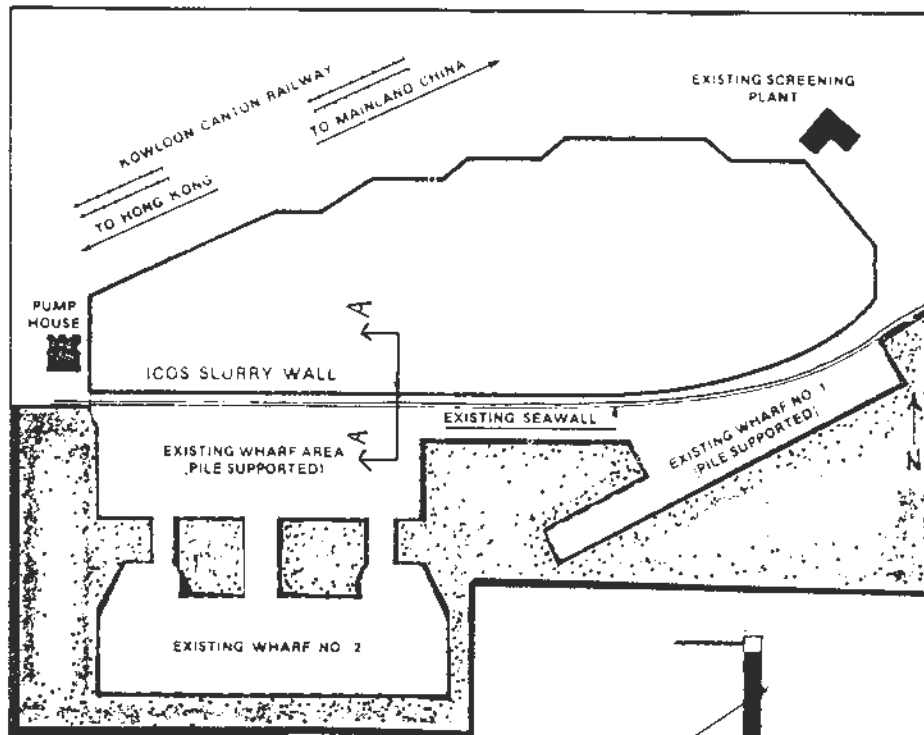


e. UNDERGROUND HIGHWAYS

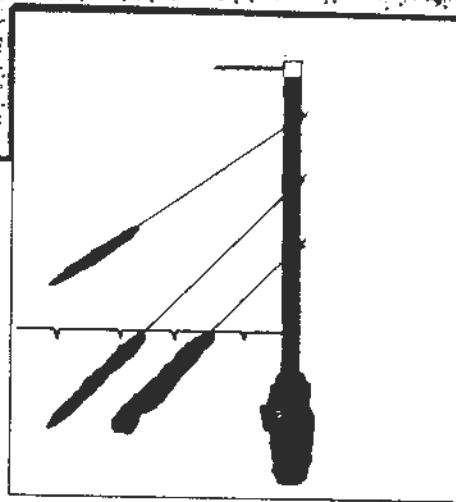


f. UNDERGROUND PEDESTRIAN WALKWAYS

Figure 4. Transportation Related Applications Of Slurry Walls



Plan



Section A-A

BACKGROUND INFORMATION

DATA ON SLURRY WALLS
 AREA 125,000 SF
 THICKNESS 30" & 36"
 MAXIMUM DEPTH 70'

DATA ON TIEBACKS
 NUMBER 1200
 LOAD CAPACITY 50T & 75T IN SOIL
 120 & 160 IN ROCK

Figure 5. New World Centre, Hong Kong

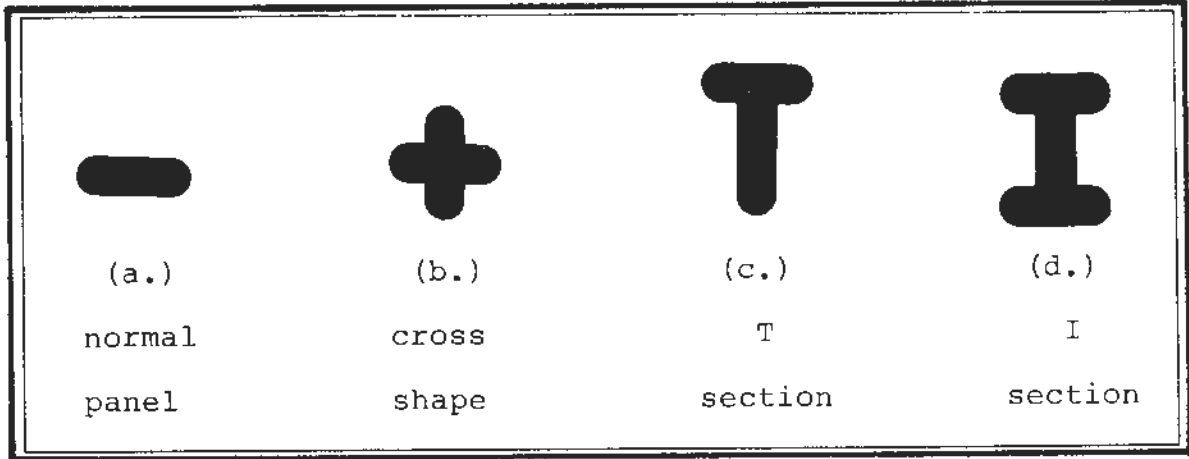


Figure 6. Cross-Sections Of Typical Load Bearing Elements



a



b

- a - Excavation I-shaped
- b - Guidewalls for T-Shaped Panel

Figure 7. Various Panel Shapes
Fabrication On Site

Photos Courtesy ICOS Corporation
of America

walls requires that wall elements be driven into the soil prior to excavation. This pile driving operation can induce considerable vibration into the soil and into existing structures surrounding the point of application, thereby causing extensive damage to the properties in the construction site area.

Slurry wall trenching operations, on the other hand, produce little vibration in the soil mass except when percussion or blasting operations can be conducted adjacent to existing structures with minimal vibration transfer as compared with shocks from pile driving.

B. Minimal Noise Disturbance

Another direct result of pile driving is noise. The irritating noise of a jackhammer is eliminated in slurry wall operations.

An interesting example of this fact was witnessed during the subway construction project in Baltimore, Maryland. Slurry wall construction efforts for the Charles Center Subway Station (Case 4, Chapter 5) are restricted to nighttime. Noise level restrictions have been placed on operations in the vicinity of Lord Baltimore Hotel. The maximum noise level allowed is 70 db after the hour of 10 p.m. A microphone was installed on an upper floor window of the hotel to monitor construction noise levels in attempts to insure compliance with the 70 db maximum. To date noise levels have been kept within limits.

Residents in the area of the Massachusetts Bay Transit Authority's (MBTA) Red Line Extension Project in Cambridge, Massachusetts were also concerned with noise levels. This concern was one of the major factors which was instrumental in the decision-making process which eventually chose slurry walls for earth support and final structure support. See Cases 7, 8, and 9 in the section on Domestic Case Studies.)

C. Minimal Traffic Disruption

Initial trench excavation and concrete pouring operations require a minimum use of street surface for most slurry wall construction projects. This means that a normal traffic pattern can be maintained through the construction corridor requiring the closing of one or possibly two traffic lanes during the project. Once slurry walls are completed to the point where excavation can commence, several construction options are available. Chapter 7 discusses several construction methods which can be employed to prevent traffic disruption.

Figure 8 shows a portion of slurry wall construction underway in the commercial area of downtown Caracas, Venezuela. Figure 8a shows traffic being maintained during the construction of guidewalls. Figure 8b shows a slurry trench drilling rig in operation while a nearby sidewalk cafe remains open for business.

Figure 9 shows a busy commercial area in downtown Baltimore, Maryland, where slurry wall trenching operations are underway for the BMTA's Charles Center Station. Note that both pedestrian and vehicular traffic are allowed to move freely through the commercial area during daylight hours.



(a) Traffic Maintained Through Construction Site where Guide Walls are Being Made



(b) Slurry Wall Trenching Machine Operating Adjacent to Sidewalk Cafe. Open for Business.

Figure 8. Slurry Wall Construction On Subway In Caracas, Venezuela

Figure 9. Slurry Wall Construction In
Downtown Baltimore, Maryland. (BMTA's
Charles Center Subway Station.)



Figure 10. Slurry Wall Construction
In Harvard Square, Cambridge, Mass.
(MBTA'S Red Line Extension)



Figure 10 shows traffic continuing through the Harvard Square area in Cambridge, Massachusetts, adjacent to slurry wall operations for the MBTA's Harvard Square Station. Also, automobile traffic is being maintained through this busy intersection while pedestrian sidewalks remain open.

With all the advantages listed above, there are still several additional environmental factors which need to be considered on a job site. For example, after continued recycling of the slurry mix, it begins to lose its effectiveness due to deterioration and contamination. When it reaches a critical level, fluid from the circulation cycle must be both removed and disposed. Since the spent slurry fluid cannot be dumped into public drainage facilities, it must, consequently, be transported by tank truck to appropriate land fill sites.

When construction of the slurry wall is adjacent to an existing structure, precautions must be taken to protect the face of this structure from damage incurred by crane and grab bucket operations, as well as by the splashing of slurry that often occurs during trenching operations. Figures 11a and 11b demonstrate some of the possible mishaps which can occur, namely, cracked glass and splattered building facade.

The waterproofing of basement walls must also be considered when working directly adjacent to existing structures which have basement levels. Such waterproofing during the trenching operations is essential to prevent slurry fluid from leaking into the structures through existing basement wall cracks.

During the trench excavation for the slurry walls, it is necessary to relocate utilities away from the area. This operation requires advanced planning and lead time to allow proper relocation efforts to be carried out. In instances where, due to size or other ramifications, it is impossible to relocate an existing utility, provisions must be made to support the utilities during excavation and build a shroud around them during concrete pouring.

In subway construction a slurry wall might obstruct the flow of ground water and cause damage to surrounding structures. To prevent this the design of the slurry wall should provide for the proper drainage of ground water.

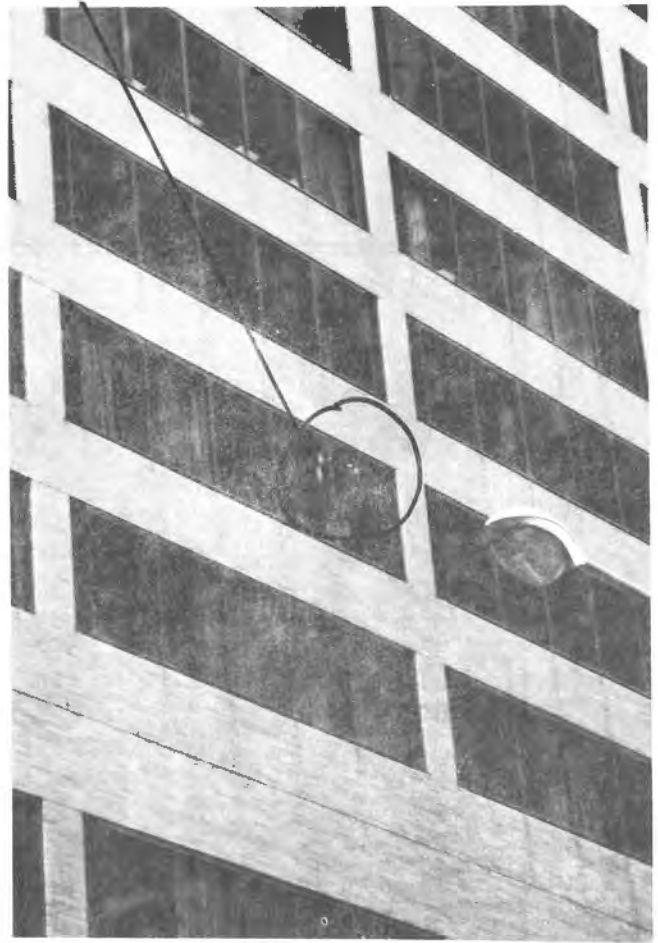
2.3 Economic Profile

The use of slurry walls as permanent structures, and, to a lesser extent, as temporary structures, can provide economic benefits both in terms of cost and construction time. Table 1 summarizes how a slurry wall, as a rigid wall system, minimizes several normal construction operations that can become quite costly, depending upon the job and site requirements.

The cost savings that can be realized were dramatically demonstrated in the construction of the Washington Metropolitan Area Transit Authority's Federal Center, S.W. Subway Station in Washington, D.C. (See Case 3 in Chapter 5.) On this particular project, the job was put out for bids allowing contractors a choice between the conventional flexible wall system and a rigid slurry wall system as temporary ground support during excavation.

For the flexible wall system, underpinning requirements for the foundation of adjacent, heavy buildings were determined by the designer. Figure 12 shows typical underpinning sections which called for 45, 30, and 90-ton piles

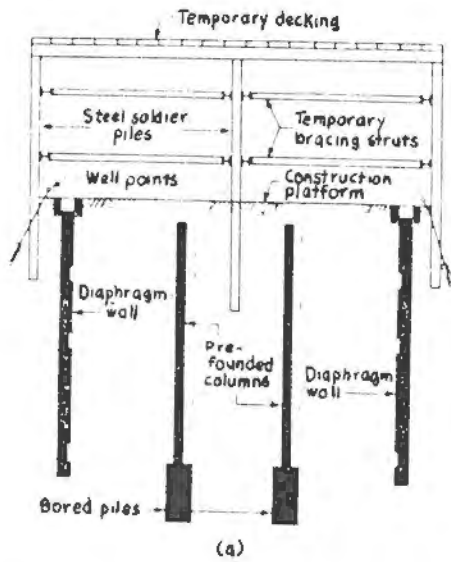
CRACKED GLASS



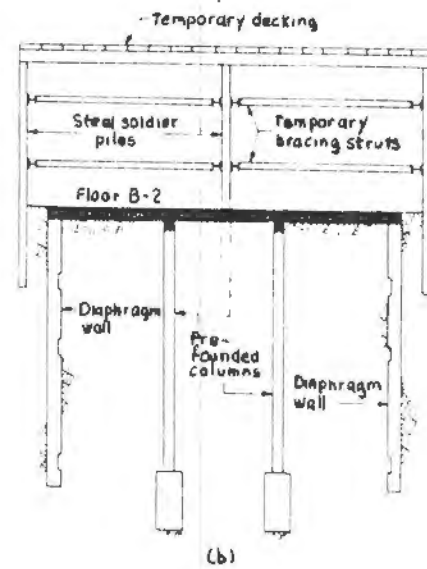
(a) Slurry Splashed Onto Face and Windows of Building From Grab bucket During Trenching Operations.

(b) Windows Cracked by Cable Suspending Grab bucket From Crane During Trenching Operations.

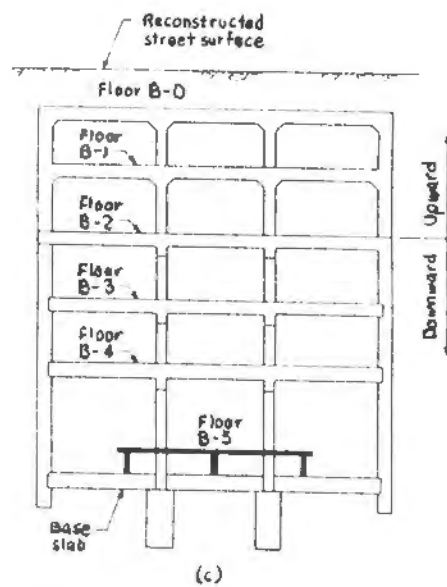
Figure 11. Construction Mishaps



Installation Of Soldier Piles
And Construction Of Diaphragm Walls



Installation Of Intermediate
Floor Level



UPWARD AND DOWNWARD CONSTRUCTION

Figure 13. Construction Of A Deep Multilevel Section By Simultaneous
Downward And Upward Process

Table 3. Profiles Of Advantages And Considerations In Slurry Wall Use

PHYSICAL PROFILE	ENVIRONMENTAL ASPECTS	COSTS
<p>Rigid Structural Wall:</p> <ul style="list-style-type: none"> - support vehical loads - restrain horizontal loads <p>Minimize:</p> <ul style="list-style-type: none"> - ground movement - ground settlement <p>Maximize:</p> <ul style="list-style-type: none"> - site dimensions <p>Reduce or eliminate:</p> <ul style="list-style-type: none"> - dewatering - underpinning <p>Applicable in:</p> <ul style="list-style-type: none"> - diverse soil conditions <p>Variable Panel:</p> <ul style="list-style-type: none"> - dimensions - geometry 	<p>Minimal:</p> <ul style="list-style-type: none"> - vibration disturbance - noise disturbance - traffic disruption <ul style="list-style-type: none"> - building protection from splashing & equipment damage - basement waterproofing - groundwater damming - utility relocation - slurry disposal 	<p>Cost savings due to:</p> <ul style="list-style-type: none"> - reduced dewatering - reduced building protection & support (underpinning) - non-redundant wall construction (labor & material) <p>Time savings due to:</p> <ul style="list-style-type: none"> - construction procedures <ul style="list-style-type: none"> - under-the-roof construction - simultaneous up & down construction - material ordering & delivery - total project duration

CHAPTER 3

DESIGN CONSIDERATIONS

3.1 Soil Loads and the Soil-Structure Interactions

Soil pressure for braced retaining structures is complicated and ever changing as excavation proceeds and braces are placed. At the onset of excavation, installation of braces at the top of the wall is effective in preventing the local translation of the wall. As excavation continues to the next bracing level, the wall tends to pivot about the upper bracing point, a process repeated throughout the excavation (see Figure 14a). For this reason, the soil pressure behind the wall does not obey the simple linear law as given in elementary text, but assumes a complicated distribution, with zero on the upper and lower ends while having a high plateau in the middle as depicted in Figure 14b. For convenience, it is recommended that the complicated distribution be approximated by a trapezoidal distribution as shown in Figure 14c, or simply a rectangular distribution as shown in Figure 14d. The maximum value of soil pressure in either case is $0.8 K_a w h$.

In addition to the lateral pressure of the soil, the designer must also consider loads due to ground water and local surface surcharge. Both must be treated as live loads since ground water levels can fluctuate and surcharge can be removed. In designing a slurry wall, the worst combination of all loads must be considered. Figure 15 shows a possible combination of loads in the wall design.

3.2 Loading Condition

To date slurry walls in the United States have been mainly used as temporary earth support structures for deep foundation and for cut-and-cover work. In principle, they can be designed as an integral part of the permanent structure. Under such a condition, the wall must serve a dual role: acting first as an earth support system during excavation and second, as a load-bearing structure after construction.

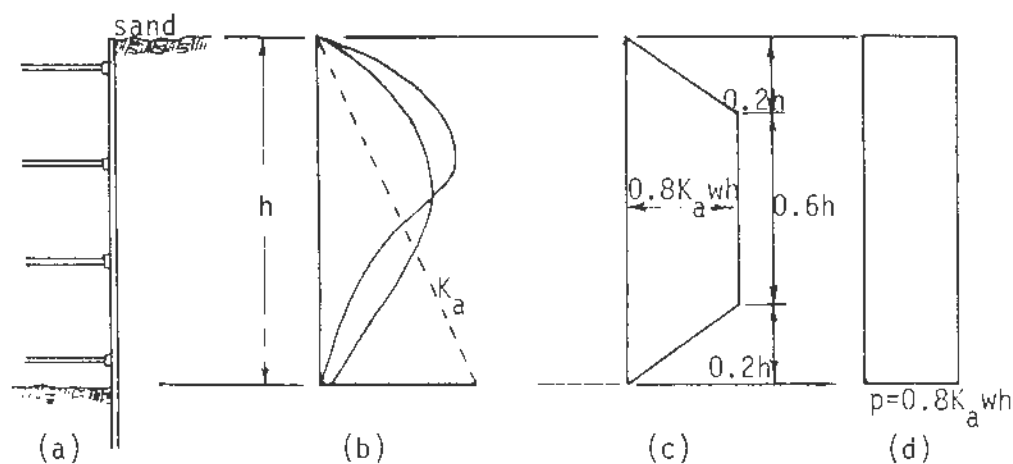


Figure 14. Braced Wall Retaining Sand. (a) May Have To Resist Pressures Of The Type Shown In (b). (c) Trapezoidal Pressure Distribution (d) Rectangular Pressure Distribution

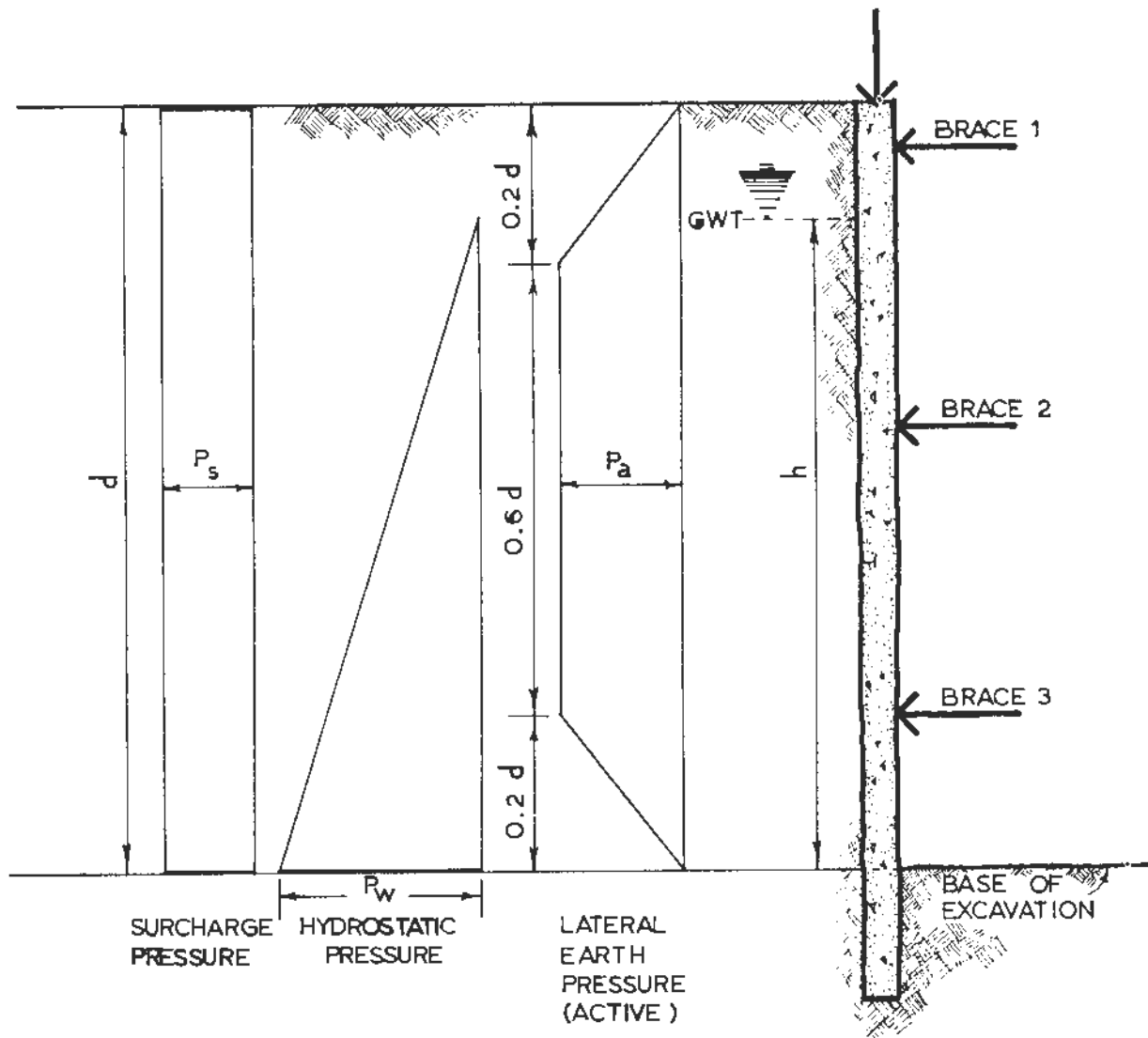


Figure 15. Load Factors In Slurry Wall Design

The design of such a permanent structure presents a loading criterion comprised of many facets. Besides considering lateral earth pressures and vertical loads, particular attention must be given to the sequence of applying and removing braces during and after excavation, as well as an analysis of a soil structure interaction.

a. Load on Slurry Wall

A slurry wall designed to be a temporary structure during construction must function like a cofferdam. The initial loading of the wall will consist of lateral earth loads and short term vehicle loads. The factors contributing to lateral loads are lateral soil pressures, hydrostatic pressure from the ground water, surcharge loads from existing structures adjacent to the excavation, and loads from construction equipment. Vertical loads can result from construction equipment, temporary decking (if used), traffic, and uplift pressures on the wall due to an imbalance of the hydrostatic head in the ground water.

As excavation of the wall proceeds, bracings are usually installed to prevent wall deflection due to the influence of lateral earth pressures. In order to control movement in the adjacent soil mass, the wall is braced at certain excavation levels to minimize wall deflection. The designer must consider the reaction of the wall during the sequence of brace installation as well as during the sequence of brace removal. Introducing a brace after the excavation has proceeded to a given depth, provides control over wall deflection. However, as excavation proceeds, the addition of each level of bracing changes the shear and moment within the wall. Careful consideration must be given to wall stress upon the introduction and removal of each level of bracing. It is important to ensure that the wall can satisfactorily handle each loading condition.

Other vertical loads on the wall are produced by the action of the adjacent soil. Settlement of the soil or the release of an overburden pressure in a overconsolidated clay stratum can cause a downward or upward pressure on the wall. Also, the friction between the wall and soil should be calculated and used when considering vertical loads on the wall.

b. Load on Station Structure

As a permanent structure, the final slurry wall must support floors and roofs forming a frame, either monolithically or by hinges.

Since the slurry wall will remain in place for the life of the structure, it will be required to carry the earth pressure that develops after construction disturbances cease and equilibrium is restored. This means that the "at rest" soil pressure will replace the "active" soil pressure that prevailed during the construction phase. We remark in passing that the active pressure includes the effect of friction along the sliding plane of the soil wedge, while in the "at rest" condition, this friction factor no longer exists. It follows that the "at rest" pressure is higher than the "active" pressure. Figure 16 illustrates ranges of values for pressure coefficients in different soil types.

A typical loading diagram for a tunnel cross section is shown in Figure 17.

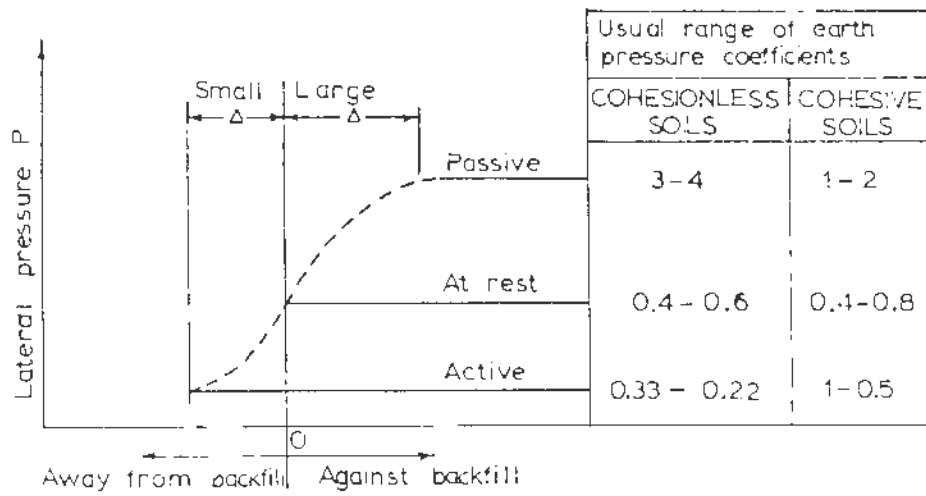


Figure 16. Illustration Of Active And Passive Pressures With Usual Range Of Values For Cohesionless And Cohesive Soil

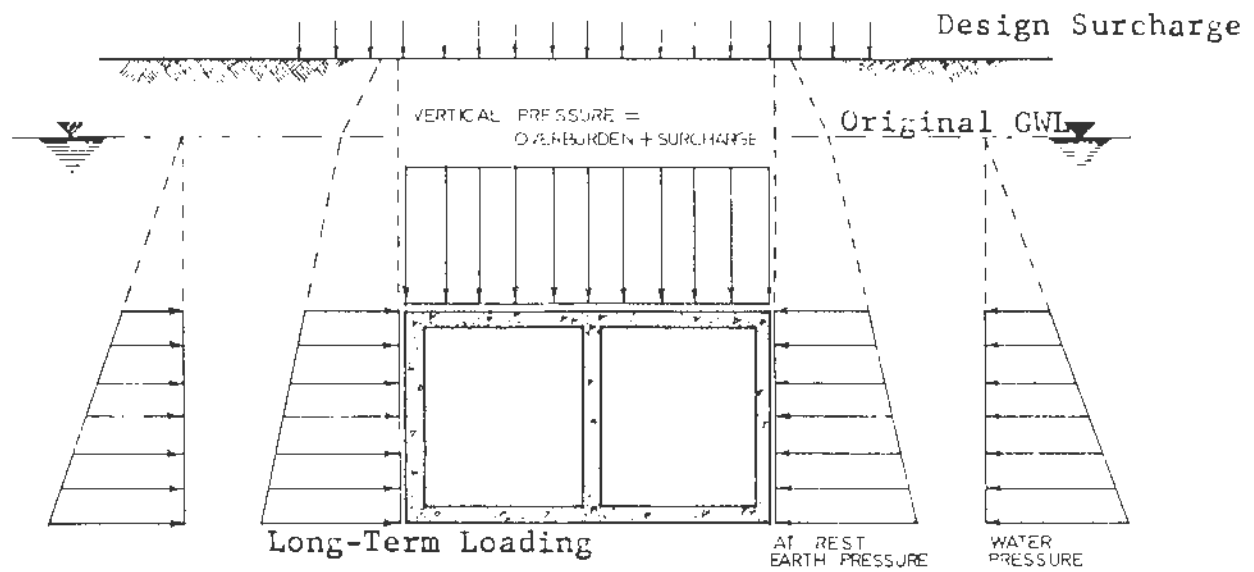


Figure 17. Typical Loading Conditions For A Tunnel Cross-Section

3.3 Basic Approaches to Station Design

There are two basic approaches to station design. The first is a frame analysis approach in which the slurry walls serve as walls for the tunnel cross section. There is no need for an additional separate vertical member in the frame. The roof and floor slabs are connected directly to the slurry wall which then becomes an integral part of the final structure, as shown in Figure 18. A block-out area is left in the face of the slurry wall so that after excavation, the rebars in the wall will be accessible in order to connect slab reinforcements.

The designer must decide on suitable connections between the completed slurry wall and floor slabs that can be tied directly into the reinforcing steel of the floor. The behavior of the entire system depends on the choice of using hinged, partially hinged or fixed joints. The slurry wall method requires insertion of a tremie pipe down into the trench between the rebars of the reinforcing cage of the wall. Often, a full moment transmitting joint requires very close placement of reinforcing bars, making it impossible to insert a tremie pipe in place. A heavier rebar is not a solution since it cannot be bent properly or practically within the dimensions of the wall. Also, for larger bars, there may not be adequate room to provide a proper lapping with the rebars in the floor slab necessary to develop full stress transfer. The designer, therefore, must give careful consideration to the design of the joint. Sometimes a partial moment transmitting joint or even a hinged joint is the solution.

A second approach to station design is known as "composite design". After excavation, a separate box section is built within the slurry wall as shown in Figure 19. The box section is connected to the slurry wall in this approach by embedding rebars throughout the vertical wall of the box section. The amount and distribution of rebars remain an open question and is basically left to the discretion of the designer. It is a difficult and controversial subject deserving extensive research effort.

Connections for either the frame analysis or the composite design approaches may be further complicated by a common requirement as specified in the design criteria of many subway systems which calls for the finished structure to accommodate asymmetrical loading. This is shown in Figure 20. This rationale attempts to insure the integrity of the tunnel cross section in the event that earth loads on one side of the structure may not develop as predicted in the design calculations, or in cases where earth loads are relieved during future excavation at the adjacent site. Such criteria are having decisive effects on the final design of subway projects currently underway in the United States.

Another important factor in the slurry wall design is the bracing configuration. From soil data and the basic structural requirements of the proposed building, the designer can determine the total required wall depth and, subsequently, choose appropriate locations for wall support. The wall is considered as a continuous beam so that the bracing load reactions can be determined by the three moment theorem. The moments in the beam can be readily determined by integrating the shear diagram. Deflections can then be calculated by the moment area method. The maximum moment will dictate the thickness of the wall, and the required steel reinforcement will correspond to the resulting moment envelope. If a steel beam and concrete wall are being considered (see Figure 2), the H-beams can act as a main structural element to which bracing can butt, thereby offering

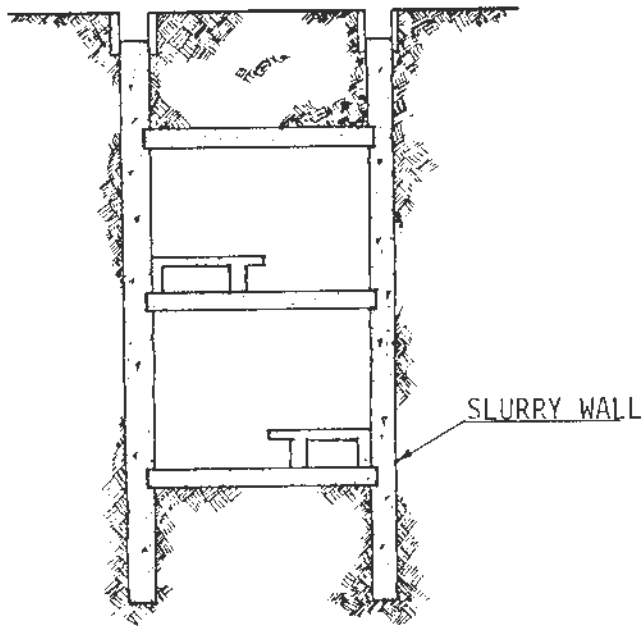


Figure 18. Slurry Walls As Direct Load Bearing Elements

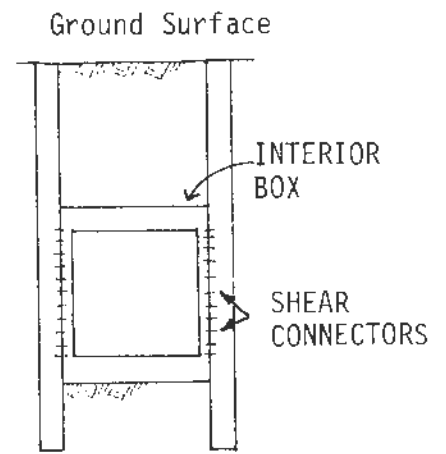


Figure 19. Composite Design

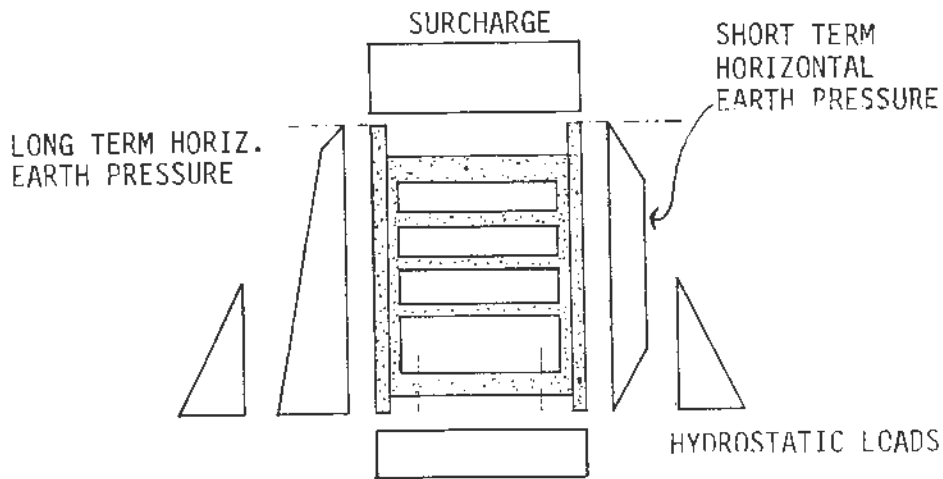


Figure 20. Asymmetrical Loading Used For Slurry Wall Design
(See Chapter 8)

resistance to flexural stress. In such a wall, reinforcing steel requirements for the concrete will be reduced, and, in some cases, where the wall is being used for temporary earth support and the panel lengths are small, reinforcing steel will be eliminated.

If the slurry walls are relatively close together, as is often the case with cut-and-cover tunneling operations, the employment of pipes or rolled structural sections for the cross-lot bracing is generally recommended. Care must be exercised so that, during excavation, the inward deflection of the wall, caused by earth pressures, is kept to a minimum. To achieve this the bracing may be prestressed to a desired load level. Due to the localized nature of the bracing force, whether prestressed or not, the designer should provide enough supplementary reinforcement near the bracing points to tolerate the proper distribution of high bearing stresses over a large wall area.

As excavation proceeds downward, additional levels of bracing should be installed. To safeguard the wall, it is important to install each brace in its designated position as soon as possible after the earth is removed from that area. Since this bracing installation can obstruct excavation operations, it may be necessary for the excavating equipment to maneuver around them.

As an alternative to those bracing procedures which may interfere with the excavating procedures, tiebacks have often been employed for wall support. Tiebacks are prestressed and extend downward at an angle of less than 45 degrees on the outside wall being excavated. They are usually anchored in bedrock or in a firm stratum. Similar devices, known as ground anchors may also be employed. Ground anchors are also prestressed and extend away from the wall at an angle approaching 90 degrees. Both tiebacks and ground anchors depend upon soil friction as an anchoring medium.

Aside from allowing an unobstructed work area between the slurry walls, tiebacks offer other advantages. When cross-lot bracing is impractical, due to the distance between walls, tieback supports remove the necessity of constructing a temporary shoring system. In addition, tiebacks can be installed as permanent wall supports, provided they are properly installed and protected against soil corrosion.

3.4 Construction Considerations

It is not uncommon that a structural design, which seems totally workable on the drawing board, encounters difficulties during construction. Slurry walls are no exception and may be susceptible to considerable complications. Because the wall is cast in a trench under a bentonite fluid, the contractor cannot visually examine the concrete while pouring. Thus, anomalies which range from air pockets in the concrete to the misalignment of a wall can recur.

The designer must have a working knowledge of the construction process in order to appreciate many of the factors that will directly influence the implementation of the design. In the course of concrete placement operations, the construction process requires that a firmly supported, reinforcing cage be placed in the trench. During tremie pouring operations, some movement of the cage is unavoidable. It is possible, therefore, that the reinforcing steel in the area of the slab connection could be misaligned from the joint area by as much as a plus or minus 6 in. The designer must be aware of this practical difficulty and take cognizance of this possibility in detailing.

Other incidents, completely beyond the control of the designer, can arise during construction. For example, if the expected earth loads suddenly give way behind the wall, bracing loads may cause a passive failure of the soil causing the wall to crack. Thus, wall movement must be carefully monitored during and after construction so that corrective action can be initiated immediately if such a situation develops.

The depth to which a slurry wall must be extended also deserves careful consideration. Usually, the wall must extend to an impervious soil stratum, if possible, to bedrock. If the bedrock is too deep, the designer must be sure that the impervious soil in which the wall is to be embedded is extensive enough to prevent seepage. A localized impervious layer above a sandy stratum can lead to "piping" and heaving at the downstream side (see Figure 21).

The recommendation that the slurry wall may be terminated a short distance below the bedrock should not be taken literally. It is considered bad practice to terminate the slurry wall construction where fissures abound in some of the bedrock, as shown in Figure 22a. Although it is advisable to pressure grout the fissures after excavation is completed, such pressure grouting is not reliable in preventing impending shear failure. If possible, the slurry wall should be extended into the solid rock for appropriate embedment below the limit of excavation as shown in Figure 22b. If, for economic reasons, this is not practical the embedment should be at least extended a considerable distance into the bedrock.

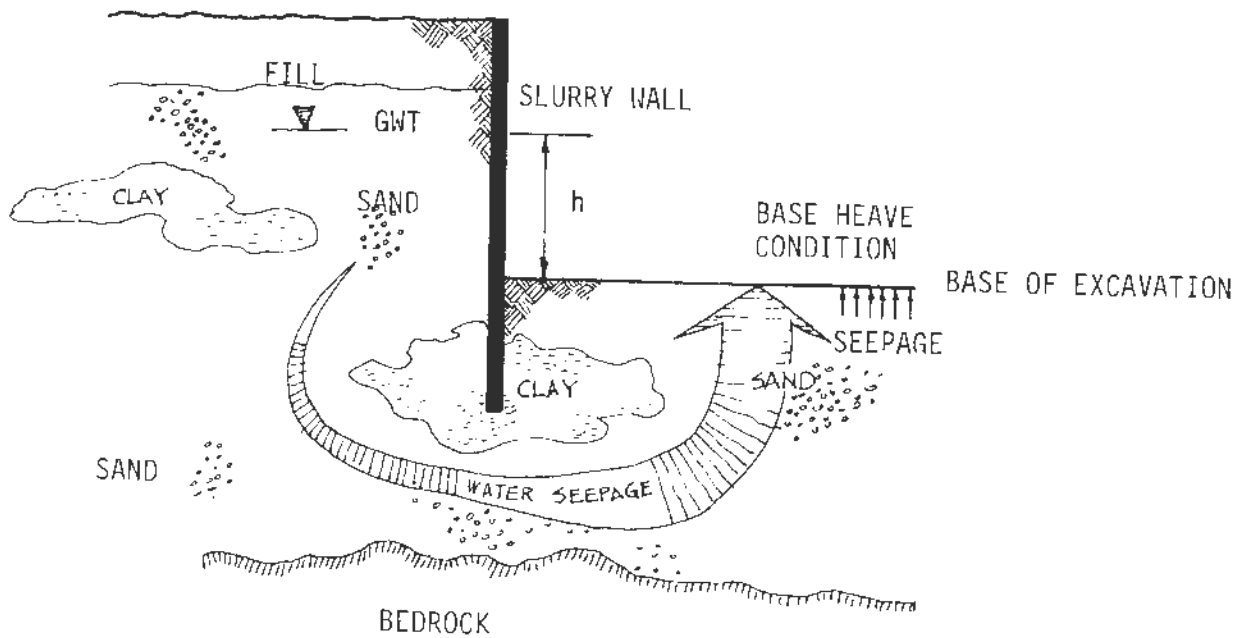


Figure 21. Excavation Base Seepage And Possible Base Heave Due To Unbalanced Hydrostatic Head (h). Caused By Slurry Wall Embedment In Impervious Stratum

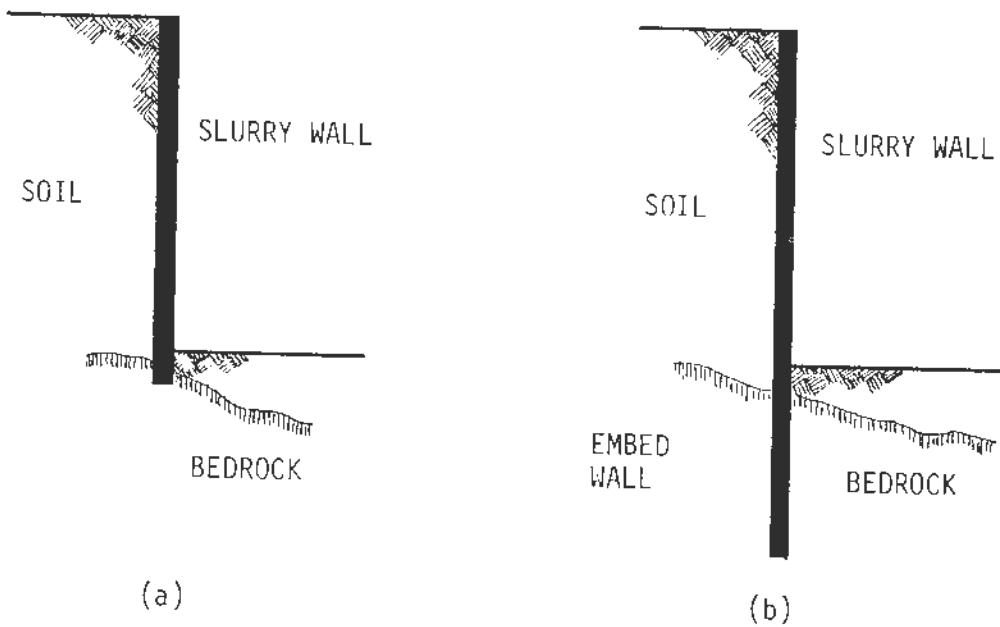


Figure 22. Slurry Wall Embedment In Rock

CHAPTER 4

BASIC CONSTRUCTION PROCEDURE

4.1 Introduction

The construction of a cast-in-place continuous slurry wall is rather simple, straightforward and can be summarized in four steps.

The first step is the construction of guide walls which serve as guides for trenching equipment and help to prevent any cave-in of soils near the surface of the trench which might be caused by the weight and movement of excavation equipment.

The second step is the excavation of the trench for the slurry wall. The excavation is carried out progressively in panel lengths.

The third step involves the placement of a reinforcing steel cage in the trench, as needed.

The fourth and final step is the placement of concrete by the tremie method whereby the concrete is carefully deposited through a pipe extended below the slurry in the trench.

After the wall is completed the excavation of the spaces between the walls may begin. During the excavation, bracing and tiebacks may be used where necessary.

4.2 Guide Walls

At the outset of the wall construction, a pre-trench is dug to allow a pair of guide walls to be built along its length. The wall is so named because it is used to control the alignment and orientation of the trench excavation so that the excavation equipment is guided to dig a trench true and plumb. The secondary purpose of the guide walls is to support the soil several ft below grade against the surcharge load from the heavy equipment. There are two types of guide walls: one for use in a firm ground, as shown in Figure 23a and the other for use in soft soil or landfill, as shown in Figure 23b. The only difference between these two types is the presence of apron slabs in the latter for distribution of the equipment weight to a wider area of the soil. The guide walls should be slightly wider than the width of the excavating equipment, e.g., the clamshell, to allow for easy movement. The recommended clearance is about two inches. This minimum distance is maintained by placing a brace between the guide walls which also assists in stabilizing the embankment.

The guide walls are usually 6 to 12 in thick, 3 to 6 ft deep and lightly reinforced with steel. They usually extend below the grade level, ending approximately 10 ft above the ground water level in order to permit adequate trench stabilization against excess hydrostatic pressure. Since it is recommended that the slurry level should be at least 5 to 6 ft above the ground water table, this practice keeps the unsupported soil depth between the guide wall and slurry to a minimum. In the locations where the ground water table is too high to permit the recommended slurry level, the natural ground level may have to be backfilled such that guide walls can be built above grade. This is necessary to avoid the even more costly dewatering procedure.

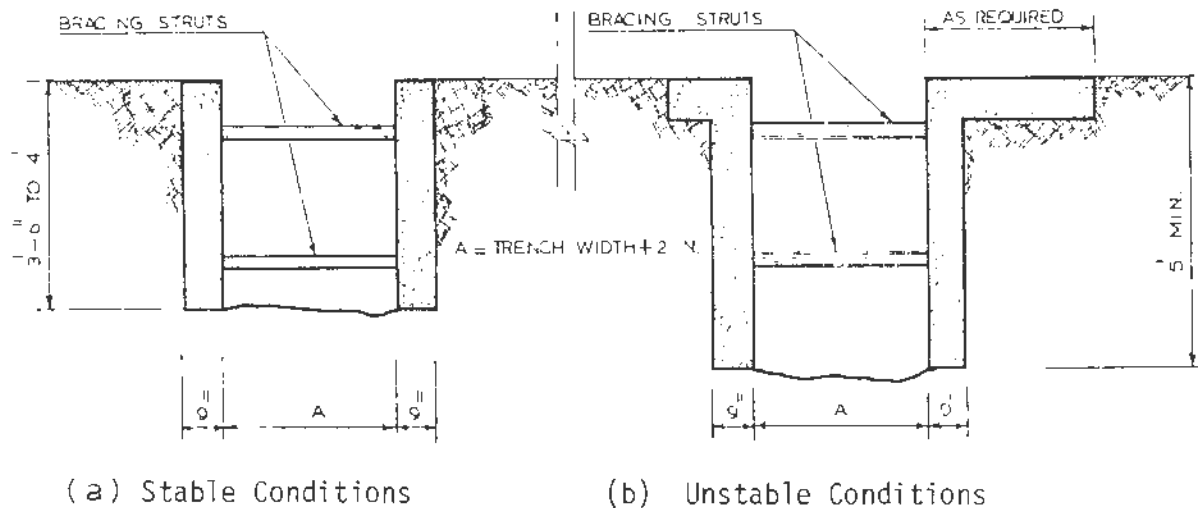


Figure 23. Guide Walls

(1"=2.5cm; 1 ft=0.3m)

The interior face of the guide wall is used to provide a smooth surface for the excavation equipment to slide through; the exterior face can be formed by soil as excavated in pretrenching. The guide walls, being temporary structures, are usually removed after the slurry wall is in place. The interior guide walls will be removed during the excavation process. The exterior guide walls can be left permanently at the discretion of the builder. On the other hand, the exterior guide walls can be omitted at the discretion of the designer. This may be necessary when the slurry wall must be built too close to an adjacent building to permit the construction of a guide wall on that side. Precautionary measures must be implemented so that foundation failure will not occur. Some of these measures are grouting the footing and waterproofing the foundation wall, etc.

4.3 Trench Excavation

Trenching operations are executed in predetermined lengths referred to as panels. Panel length is dictated by such factors as soil stability and equipment size. During the excavation of each panel, a slurry fluid is pumped into the trench to provide side wall stability and lubrication during drilling operations.

a. Trenching Equipment

During the 34-year history of slurry wall construction, excavation equipment has been vastly improved in order to meet the ever increasing challenge of greater depths and difficult types of soil. Earliest equipment consisted of

mechanical diggers which could excavate up to 100 ft deep. Later, power drilling units extended the applicable depth to as much as 400 ft.

Today, the most popular equipment used for slurry wall trenching operations is the clamshell-type grab bucket. It is most effective for soft to medium hard, loose or cohesive soil. As long as boulders and other obstructions are not encountered, clamshell rigs can easily excavate trenches up to 200 ft in depth.

Clamshell grab buckets are available in various types and sizes. These buckets can be either mechanically or hydraulically operated (see Figure 24). Both the mechanical and hydraulic buckets can be suspended from the boom of an overhead crane by cables or by a Kelly Bar.* The size of the clamshell bucket depends upon the desired thickness of the slurry wall. Buckets have widths which vary from 18 to 60 inches; the most common and practical ones usually have widths between 24 and 36 inches. Variation in the weight and capacity of these buckets will depend upon soil parameters and excavation schedules. In 1972 a specially designed clamshell bucket which was 20 ft high, 32 inches wide, and weighed more than 9 tons was employed in trench excavating operations for a section of the WMATA Federal Center, S.W., Subway Station in Washington, D.C. This bucket had a capacity of 1.5 cubic yards. Today, grab buckets this size and larger (over 11 tons) are commonly employed.

The clamshell grab bucket is ineffective when it encounters an area of hard packed soil or a boulder which exceeds its width or length. In these instances, percussion tools or drills must be utilized. Excavation can continue only after heavy chiseling, rotary drilling equipment, or blasting are employed to reduce obstructions to pieces small enough for the clamshell to handle.

For soil that contains many boulders or is continuously dense and hard-packed, as well as for sites where the trench will extend to depths exceeding 200 ft, the excavation is primarily carried out with percussion and/or drilling equipment.

b. Panel Arrangements

As previously mentioned, trenching operations are carried out in predetermined lengths, referred to as panels. These panel lengths are usually controlled both by the project design requirements and by the type of equipment being used. By making several passes with the excavation equipment, the panel can be increased to the desired length. Panel lengths are generally between seven to ten ft.

The main factors which determine the panel length for any given job are:

- (1) Width and Depth of Trench. Along with the length selected for the

*Kelly Bar: Long, rigid structural member (usually an H-shaped section) to which grab bucket is attached at one end. The crane lifts and lowers the grab bucket into a trench using the Kelly Bar as a guide instead of the cable.

c. Excavating Sequence

The alternating panel construction sequence excavates and constructs every other panel, each of which has semi-cylindrical ends, either convex or concave. The first set of panels cast are known as primary panels and have convex ends as demonstrated in Figures 25a (Panels A and B), and 25b (Panels G, C, A, B, and F). After the concrete has set in all primary panels, the spaces between these panels are then cast in concrete thereby forming a complete wall. Because they are constructed after the first set of panels, these panels are called secondary panels and have concave ends as demonstrated in Figures 25a (Panel C), and 25b (Panels, I, E, D, and H). The cylindrical ends in the primary and secondary panels are formed by pipe segments for the purpose of serving as interlocking joints to form the continuous wall. Further discussion of these joints can be found in Section 4.4, "Reinforcement."

Figure 25c demonstrates an alternative excavation procedure in which panels are set in sequence. With this process the first panel end is convex, while the second panel end is concave. In this case Panel A is completed, and construction on Panel B must begin at the extreme right of the Panel commencing toward Panel A in order to allow the concrete in Panel A to set completely before construction on Panel B reaches it.

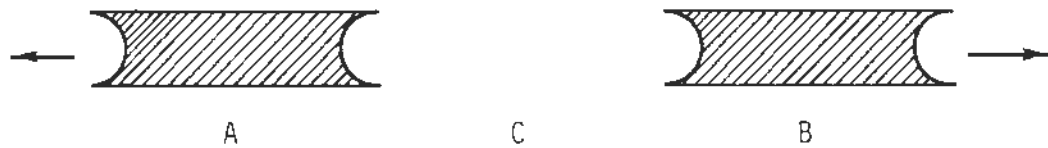
Similarly, Figure 25d demonstrates another aspect of successive panel construction. In this case Panel A is completed. Both Panels B and C are begun. However, construction on these two panels must begin at the far ends of each and commence toward Panel A. Again, the purpose is to allow Panel A to set completely before Panels B and C reach it.

d. Slurry Mixture

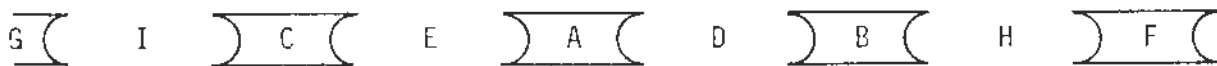
Bentonite is a granular, natural clay consisting mainly of montmorillonite and possesses pronounced thixotropic properties when mixed with water. In slurry wall construction the bentonite/water combination is the mixture most commonly used because of its ability to stabilize the open trench throughout the excavation period. When the slurry mixture is maintained near the top level of the trench, the trench can sustain its shape without elaborate shoring. Tests have proven that trench stability can be maintained for more than a month, even with surcharge loads on adjacent soil.

An interesting demonstration of slurry filled trench stability occurred in the fall of 1969 during construction of EMBARCADERO SUBWAY STATION of the BART SYSTEM in San Francisco, California. A significant earthquake (of about 6.0 on the Richter scale) centered approximately 60 miles (97 km) north of San Francisco took place. Two 15 ft (4.6 m) long, 44 in. (1,100 mm) thick, fully excavated, slurry filled, wall panel trenches were undisturbed.

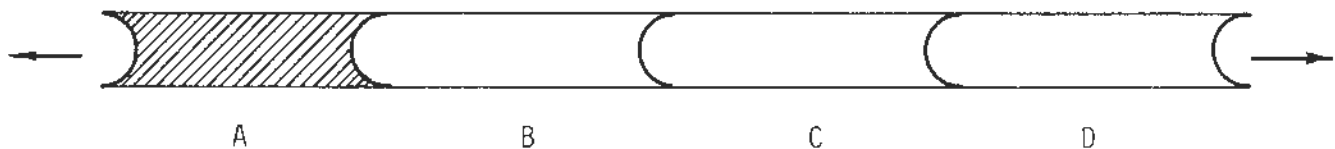
The mechanics of trench stability and the balance of soil and water pressures developed by bentonite slurry are not fully understood. However, several factors are known to effect the behavior of bentonite or trench stability and are listed below:



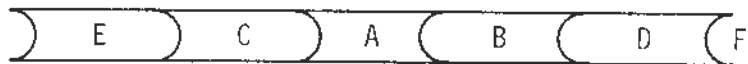
a. ALTERNATING PANEL CONSTRUCTION



b. ALTERNATING PANEL EXCAVATIONS WITH PRIMARY (A, B, C, F, G) AND SECONDARY (D, E, H, I) PANELS



c. SUCCESSIVE PANEL CONSTRUCTION



d. SUCCESSIVE PANEL SEQUENCE

Figure 25. Excavation Sequence

(1) Density of Slurry and Differential in Hydrostatic Head. The slurry has a density slightly greater than that of water. The hydrostatic pressure exerted by it and its thixotropic properties prevent the ground water from seeping through as long as the level of slurry in the trench is maintained at a sufficient depth above the ground water table. The recommended depths for granular soils and cohesive soils are 5 and 3 ft respectively. The density of the slurry can be increased by adding heavier minerals to the mixture, but the bentonite concentration should be kept the same. The higher density will be needed for locations in which the ground water table is so high that the above-mentioned depth differential cannot be implemented.

(2) Arching Action. In unstable ground conditions or areas of high surcharge, if the panel length of the trench is kept short, relative to its depth and width, there will take effect in the soil a certain amount of arching action which helps prevent collapse of the upper areas of the trench. Otherwise, trench failure may occur along a three-dimensional curved surface near the ends of the trench before the trench itself reaches a point of total collapse.

(3) Caking of Bentonite Fluid and its Effect on Improving Shearing Strength of the Soil. Bentonite, being very fine in texture, is able to penetrate deeply into the pores in the soil. The slurry fluid, under hydrostatic pressure, has been observed to penetrate several feet into granular soils such as sand and gravel formulations. After a short time has elapsed, the slurry fluid tends to jell due to the thixotropic properties of bentonite mix. The soil particles are bonded more tightly together, resulting in greater shearing strength of the soil. This behavior is augmented by an electrochemical phenomenon such that the formation of the so-called "filter cake" is prevalent along the excavated surfaces of the trench. The combination of jelling and caking keeps the trench faces stable and watertight.

At each construction site, there is usually a station for desanding and cleaning the stabilizing fluid. Following is a description of this operation:

a. As trench excavation proceeds, the slurry stabilizing fluid becomes filled with suspended soil residue from either the drilling or clamshell excavation cycles.

b. The contaminated slurry is drawn out of the trench and replaced with clean, fresh slurry, either through suction pumps or air lift systems.

c. The contaminated slurry fluid can be reused after cleaning. The foreign substances in the contaminated slurry fluid are generally coarser and heavier than the bentonite and can be easily removed by a vibrating "mud screen." The more effective way to remove contaminants from the slurry fluid is with cyclone equipment using a centrifugal force principle or through a precipitation process using chemicals with coagulates.

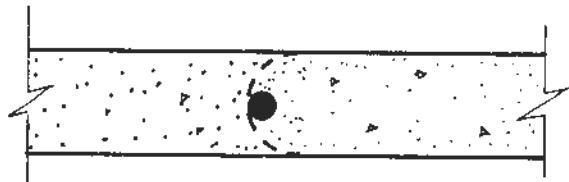
With continual exposure to contaminants, the effectiveness of the bentonite slurry gradually deteriorates. When this deterioration reaches a critical level, the fluid must be removed from the circulation cycle and disposed.

After the excavation is completed, one final cleansing of the trench bottom is performed to remove all settled residue from the slurry. The removal and the disposal of both the used bentonite and mud from the trench must take into consideration that most local ordinances prohibit the discharge of mud or slurry fluid into public drainage and/or disposal systems. Because of this, in most metropolitan areas, the mud is removed in dump trucks and spent slurry is removed in tank trucks to predesignated land fill locations.

4.4 Reinforcement

The slurry wall requires numerous, heavy reinforcements. To facilitate their placement, these reinforcements are usually fabricated into cages. Needless to say, the width of each cage should be slightly narrower than the trench. The fabrication site should be near the trench to avoid unnecessary efforts and transportation costs. Rebar detailing should be carefully done with the consideration of final fabrication of the reinforcement into a cage. Rebars should be tied together securely with minimum welding. Experience shows that residual stresses, due to welding, can be harmful. Breakage of rebars during necessary handling of the cage is not uncommon.

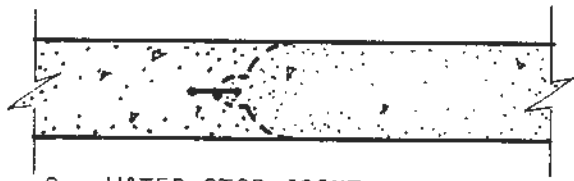
As previously stated, the most common joints used in slurry wall construction are those formed by using pipes or H-beams between panels. Plan views of these panel joints were shown in Figure 2. In an effort to insure a more water-tight joint between panels, several key waterstop joints have been used in slurry wall panels as shown in Figure 26.



A. SINGLE KEY JOINT



B. DOUBLE KEY JOINT



C. WATER STOP JOINT



D. WATER STOP JOINT
(STEEL BEAM)



E. INTERLOCKING JOINT



F. SPLIT PILE JOINT

Figure 26. Keyed And Water-Stop Joints

CHAPTER 5

DOMESTIC CASE STUDIES

Four transportation related slurry wall projects in the United States were studied in great detail. The location, ownership, and status of these projects are listed in Table 4. Also, significant findings of nine individual cases from the four studies are reported in the following pages and are summarized in Table 6.

All factual information was obtained from unpublished design documents received from consultants and by personal communications with design engineers associated with those projects.

TABLE 4. List Of Domestic Case Studies

<p>I LOCATION: San Francisco, California</p> <p>OWNER: Bay Area Rapid Transit (BART) Authority</p> <p>Case 1: Civic Center Subway Station Status: Completed 1970</p> <p>Case 2: Embarcadero Subway Station Status: Completed 1972</p>
<p>II LOCATION: Washington, D.C.</p> <p>OWNER: Washington Metropolitan Area Transit Authority (WMATA)</p> <p>Case 3: Federal Center S.W. Subway Station Status: Completed 1974</p>
<p>III LOCATION: Baltimore, Maryland</p> <p>OWNER: Baltimore Region Rapid Transit System, Mass Transit Authority (MTA)</p> <p>Case 4: Charles Center Subway Station Status: Under Construction</p> <p>Case 5: Laurens Street Subway Station Status: Under Construction</p> <p>Case 6: North Avenue Subway Station Status: Under Construction</p>
<p>IV LOCATION: Boston, Massachusetts</p> <p>OWNER: Massachusetts Bay Transit Authority (MBTA)</p> <p>Case 7: Harvard Square Subway Station Status: Under Construction</p> <p>Case 8: Davis Square Subway Station Status: Design Stage</p> <p>Case 9: Subway Line Section, Davis Square to Alewife Status: Design Stage</p>

I. Project Overview

- A. Location: Bay Area Rapid Transit (BART) District
San Francisco, California

The station is a key element of the subway section in downtown San Francisco, located on Market Street between 7th and 8th streets on the old shoreline of an arm of Mission Bay.

- B. Description: The subway system on Market Street is in two subterranean transit levels. The upper level consists of the city-owned street car system and the lower level consists of the high speed BART system. A mezzanine, located at the station entrance, comprises the third level where passenger fares and transfers are transacted.
- C. Project Status: Completed
- D. Site Plan
1. Right of Way: Distance between building lines across Market Street is 120 ft (37 m); station excavation was approximately 99 ft (30 m) wide, which left about 10 to 11 ft (3 m) right-of-way on each side. (see Figure 27).
 2. Buildings and Foundations: Nearby structures are older, urban commercial buildings up to eight stories in height. Most buildings have single story basements, and are of frame or masonry construction on a spread foundation. Some would be seriously damaged by the expected excessive settlements in conventional construction.
 3. Remarks: The general area was open to traffic during the construction period. Utilities were removed and relocated.

II. Geotechnical Information

- A. Soil Profile: A large section of downtown San Francisco was created by backfilling water areas with earth and dredged sand dunes. Figure 28 shows the subsurface conditions at the station site.
- B. Ground Water Level: A high ground water table existed with pervious soil layers.

III. Site Requirements

- A. Constraints or Conditions:
1. Placement of the station in a built-up urban area with adjacent buildings sensitive to damage by settlement.
 2. Highly compressible organic deposits underlying a large area in the vicinity of the station.

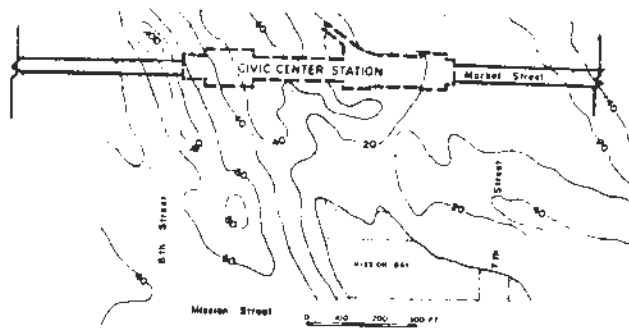


Figure 27. Plan Of Civic-Center Station Showing 1852 Topography

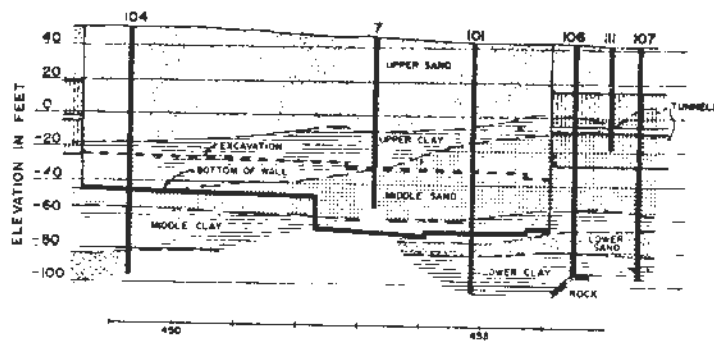


Figure 28. Subsurface Conditions - Civic Center Station

(1 ft=0.3m)

- B. Special Considerations: To avoid the expected excessive settlement, excavation was carried out without substantially lowering the water table in adjacent areas. Constructing a strong watertight wall around the site was mandatory.

IV. Design Information

- A. Design Criteria and Loading: In the absence of experimental data, lateral earth pressure criteria were developed under zero lateral strain conditions. The average values of lateral pressures were:

<u>Material</u>	<u>Average Values/Lateral Earth Pressure Coefficients</u>		<u>Hydrostatic Pressure</u>
	<u>K_0</u>	<u>K_{0d}</u>	
Sand	0.34	0.38	full
Clayey Silt	0.41	0.44	full
Combined Soils	0.38	0.41	full

where K_0 is the ratio of the effective minor principal stress to the effective major principal stress under no lateral strain conditions. $K_{0d} = \tan^2 (45^\circ - (\phi_d/2))$ and ϕ_d = the angle of friction developed at K_0 conditions.

An envelope of design brace loading was adopted using a lateral earth pressure coefficient of 0.45 with both rectangular and triangular distribution, and full hydrostatic pressure. (see Figure 29)

- B. Design Alternatives: Sheet piling was considered but was rejected because of its relatively low flexural strength, susceptibility to leakage, and difficult installation to 100 ft (30 m) depths. For similar reasons, the use of soldier beams and lagging, combined with dewatering, was also eliminated.
- C. Final Choice: A steel column and concrete type slurry wall was selected. The wall was heavily reinforced with wide-flange steel columns set in slurry-stabilized bored holes. The slot between the flanges of adjacent vertical steel sections was excavated with clam buckets. Excavation was stabilized with slurry, and concrete was tremied into the slot.

V. Results

- A. The Structure: The slurry wall belongs to the steel beam and tremie concrete type. The construction of slabs at mezzanine, middle and lower levels was carried out in an inverted manner. That is, instead of completing the excavation and then proceeding from bottom up, each slab was placed as soon as the excavation reached its level. After the concrete set, the excavation proceeded in a manner similar to the under-the-roof construction method. When excavation was finished, the bottom slab was poured in place, and the walls of the station were completed. Seepage under the station walls was controlled by extending the walls below the excavation level at least 20 ft (6 m) or into one of the clay layers. Figure 30 shows typical sections.

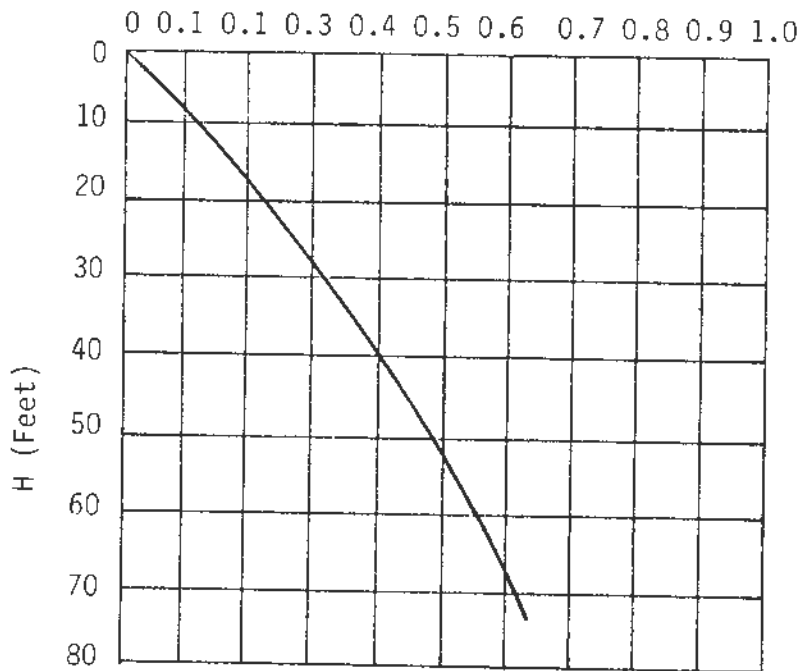
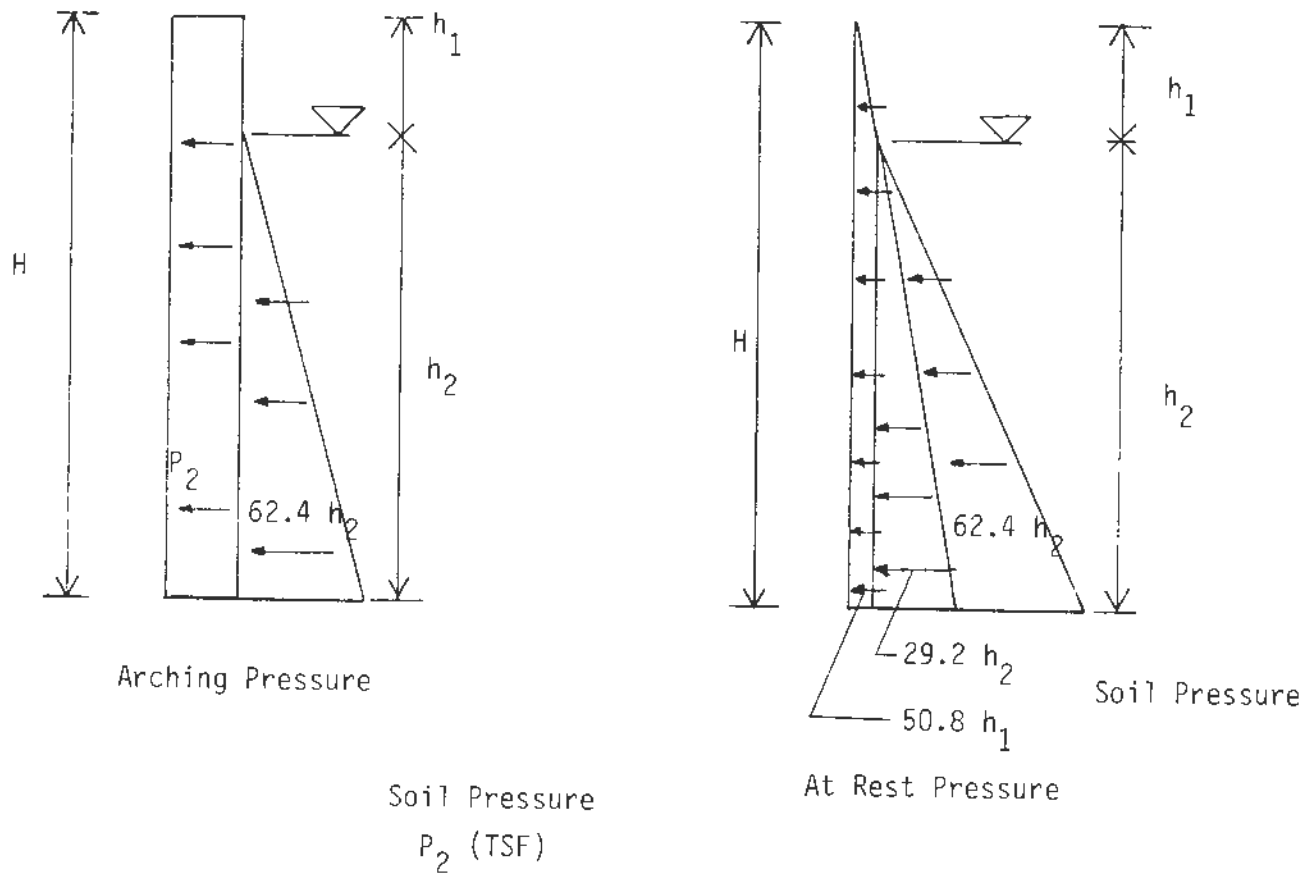


Figure 29. Design Lateral Pressure

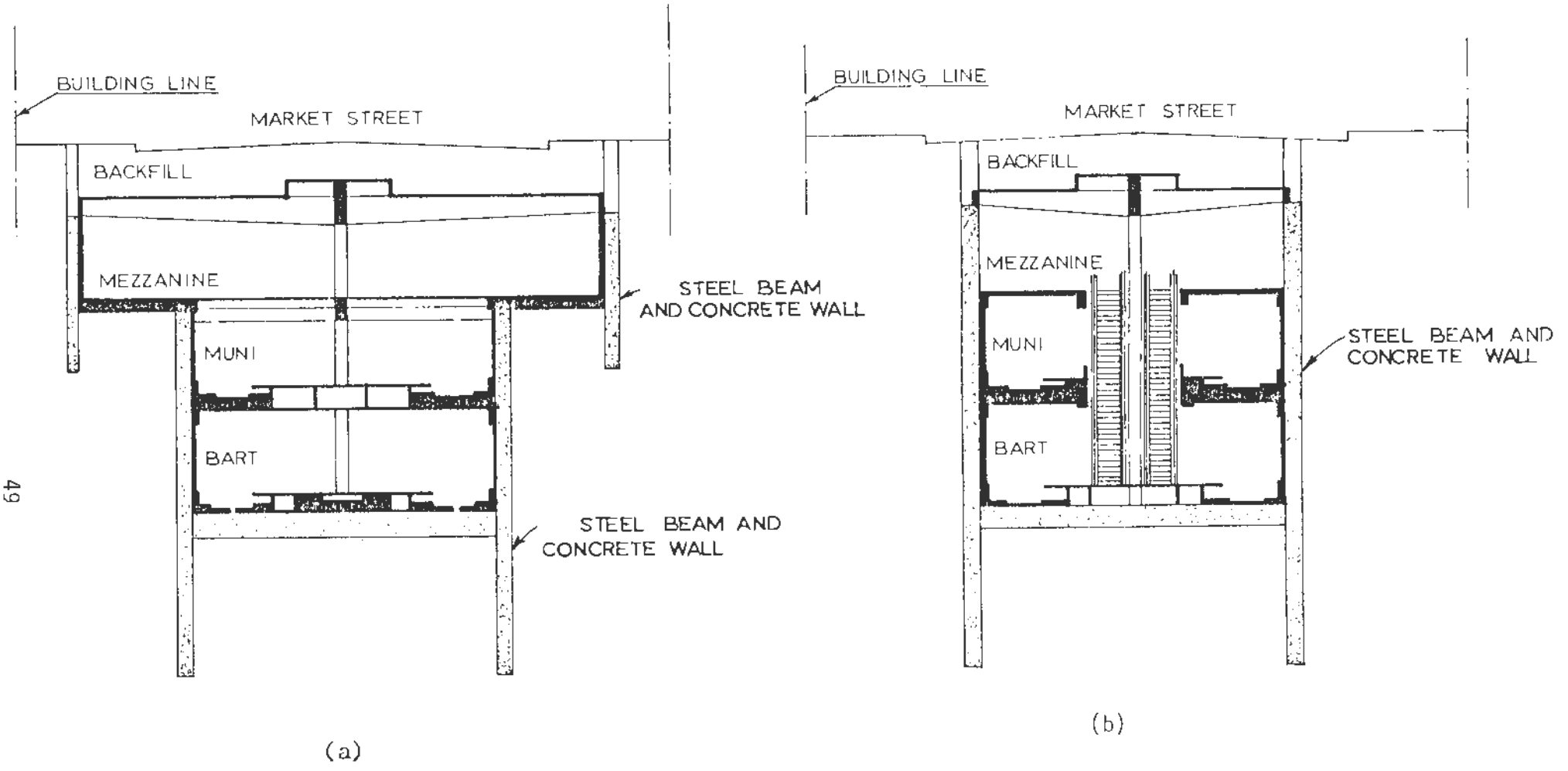


Figure 30. Typical Station Sections Of Bart Civic Center Subway Station

I. Project Overview

- A. Location: Bay Area Rapid Transit (BART) District
San Francisco, California
- B. Description: The Embarcadero Subway Station is located on Market Street in San Francisco between Bush and California Streets. The design and construction of the lower level trackways allowed the high speed BART trains to use the trackways even though the station was not completed.
- C. Project Status: Completed
- D. Site Plan:
 - 1. Right of Way: Market Street, where the construction took place, has a 76 ft roadway (23 m) with a 22 ft (7 m) sidewalk on each side. (see Figure 31).
 - 2. Building and Foundations: On both sides of the street there are buildings; most of them have five stories, some have as many as 32 floors. All buildings have basements as well as sidewalk vaults and rest on long piles.
 - 3. Congestion: Underground utility facilities form a complicated crisscross network. Extensive and permanent utility relocation effort was necessary.

Maintaining vehicular traffic (streetcars, trolley coaches, buses) during construction created an immense problem but the overhead trolley power lines and street car tracks were relocated.

II. Geotechnical Information

- A. Soil Profile: The soil conditions existing at the site are summarized in Figure 32.
- B. Ground Water Level: Ground water existed about 15 ft (5 m) below street surface at mean sea level elevation 0.

III. Site Requirements

- A. Constraints or Conditions
 - 1. Because of the presence of saturated layers of clay, the natural water table in adjacent areas had to be maintained to prevent the consolidation from taking place. Lowering the water table would cause unacceptable settlements of adjacent areas and buildings.
 - 2. Building sensitivity required particular control of lateral movement in the retained soil.

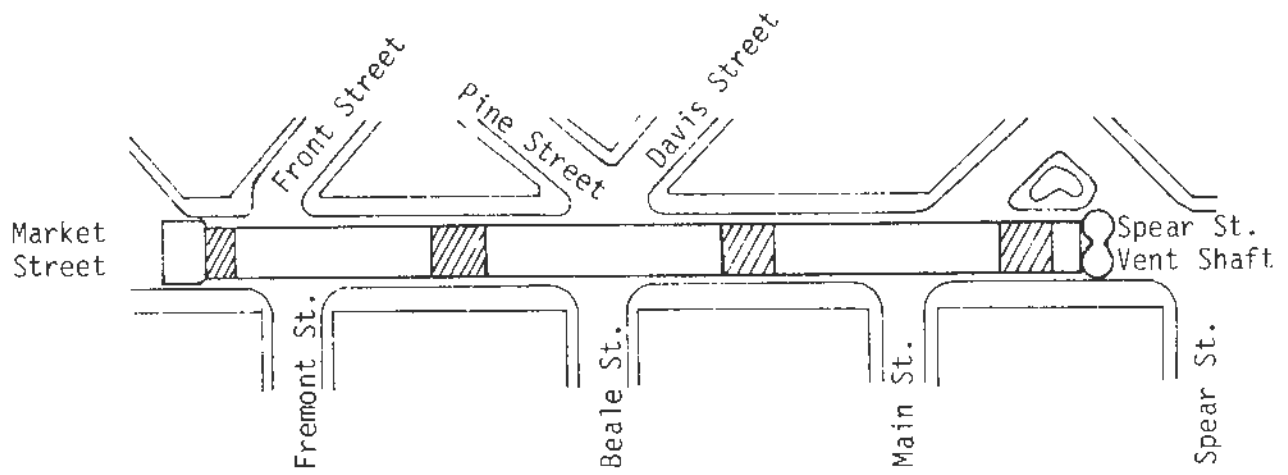


Figure 31. Site Plan

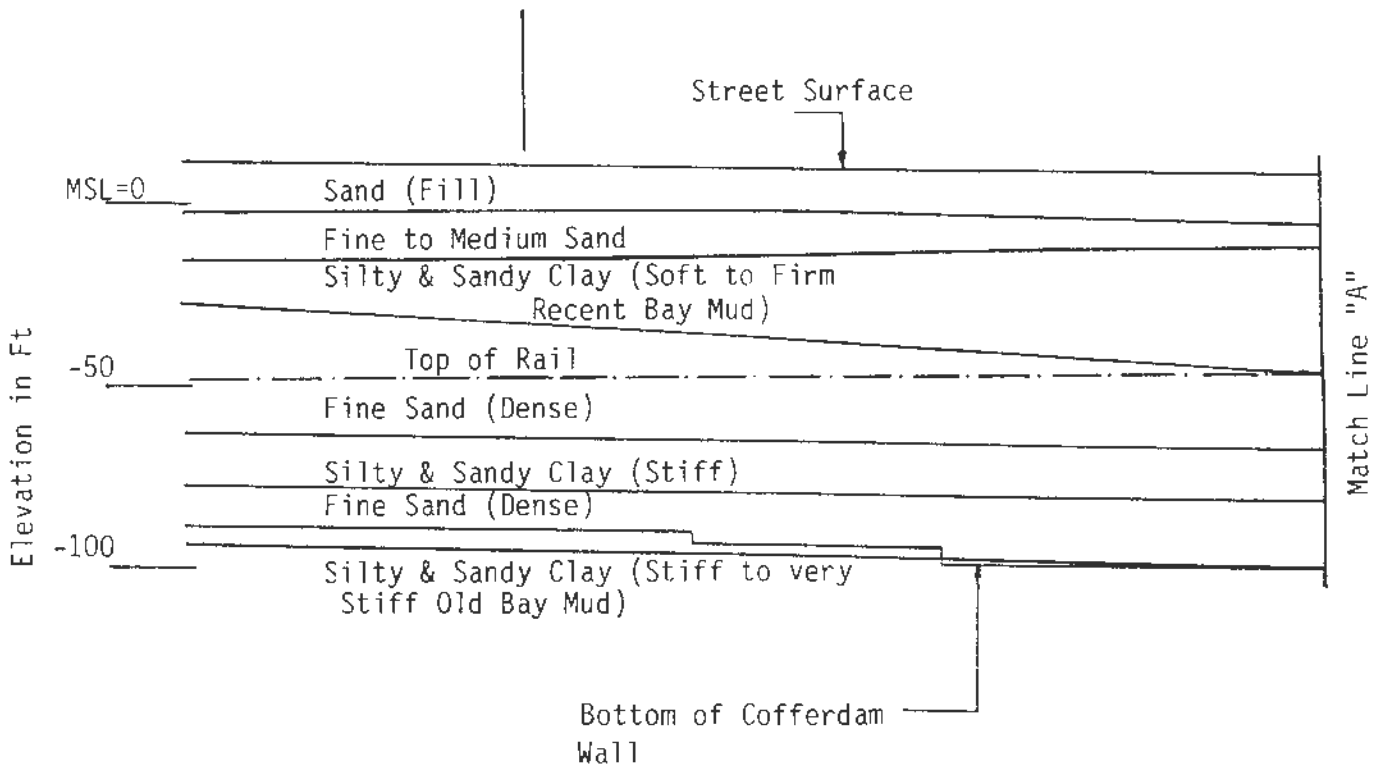


Figure 32. Longitudinal Soil Profile

IV. Design Information

- A. Design Criteria and Loading: To provide control of ground settlement and lateral movement called for a rigid cofferdam system with adequate bracing.

Active and passive lateral earth pressures to support excavation were conservative (see Figure 33). The consideration of rigidity and strength were key factors. Design calculations indicated that very high moments would develop. However, it was desirable to keep the wall within a reasonable thickness of 3 to 4 feet.

- B. Design Alternatives: Three design alternatives were considered:

1. Reinforced concrete walls.
2. Secant, cast-in-place reinforced concrete pile walls.
3. Structural steel shape with concrete panels.

- C. Final Choice: It was decided, that in order to minimize ground movement during wall construction, slurry walls of steel beams and concrete system offered the best solution.

V. Results

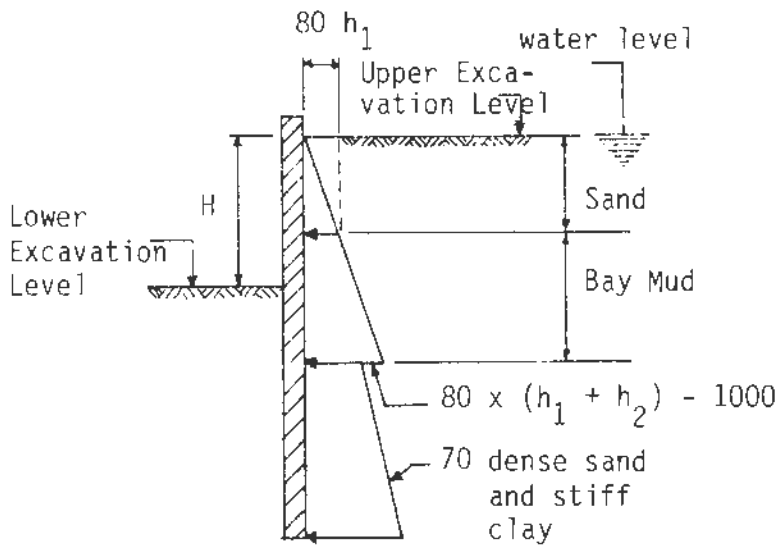
- A. The Structure: Figure 34 shows a cross section of the excavated site. The area enclosed by the cofferdam was 1160 ft (354 m) long, 55 ft (17 m) wide, and the excavation depth was 70 ft (21 m).

Cross-lot bracing was used during the excavation. The bracing beams were rigidly connected to the walls on the ends and had vertical supports at suitable locations. The bracings had appropriate horizontal spacings and were shimmed by walings according to the contractor's discretion. However, the vertical spacing of the braces and the removal schedule were tightly controlled by the design engineer since any deviation could affect the safety of the walls. Figure 35 shows how the slurry wall was incorporated into the final structure.

- B. Performance: Ground water level monitored through a series of observation wells installed both outside and inside the cofferdam.

Painted markers and settling measurement points were driven into the ground to monitor downward movements. Maximum settlement for the westerly half was 1.1 in (28 mm), while maximum settlement for the easterly half was 1.7 in (43 mm). The lateral inward movements were measured with an inclinometer. Generally, readings indicated that the walls moved toward the excavation. The final cumulative deflection of the wall sections varied in magnitude in direct proportion to the thickness of the soft clay soil.

Active Pressure



Passive Pressure

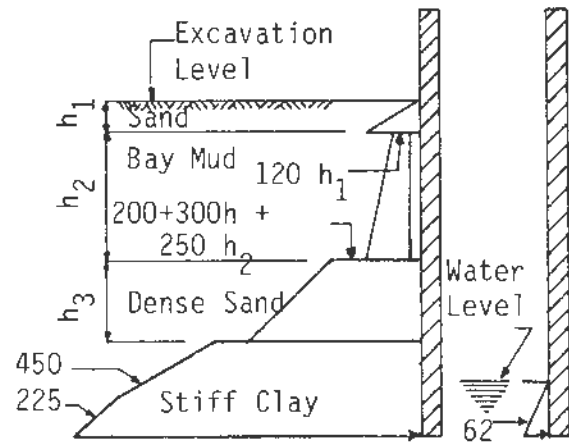


Figure 33. Design Lateral Pressure and Design Passive Resistance for Support of Excavation

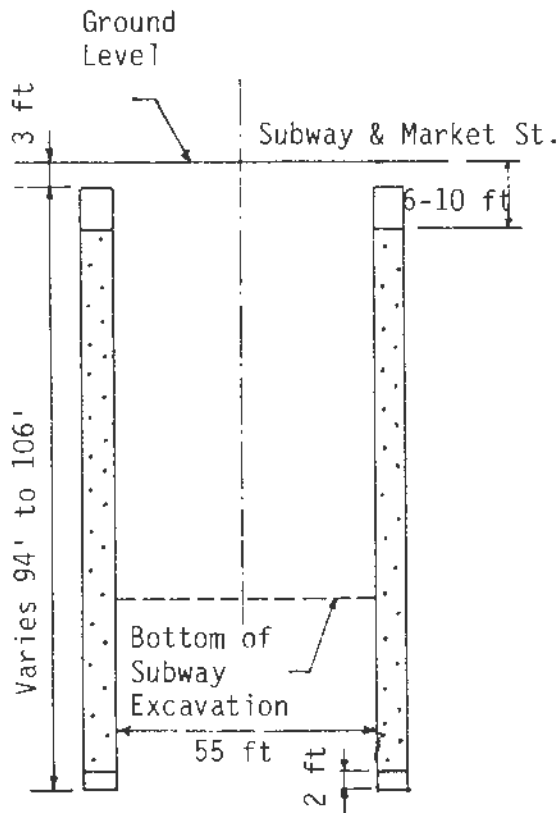


Figure 34. Cross Section of Cofferdam Wall

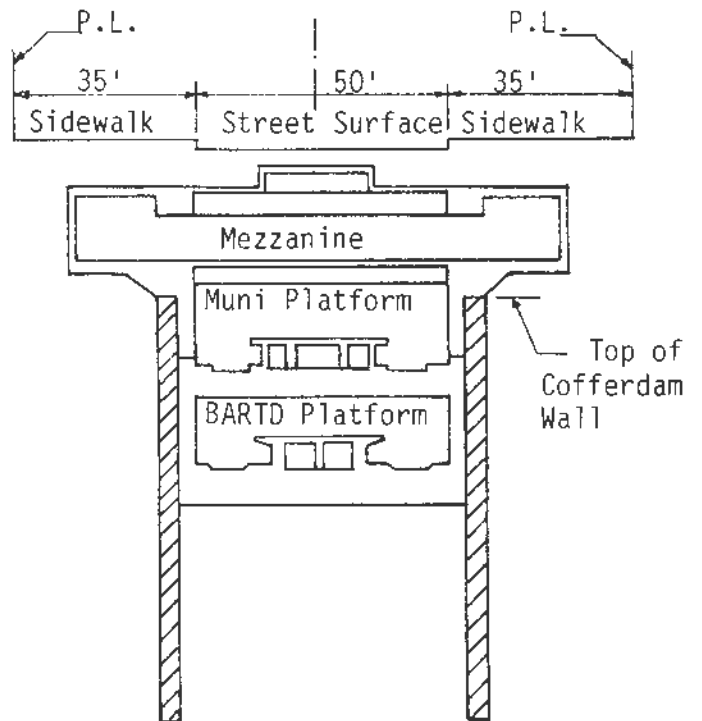


Figure 35. General Section (1 ft=0.3m)

I. Project Overview

- A. Location: Washington Metropolitan Transit Authority
(WMATA) Washington, D.C.

The station is located along D Street between 2nd and 4th Streets, S.W. Washington, D.C.

- B. Description: Station structure accommodates inbound and outbound tracks on a single bed with double crossover tracks at the east end.
- C. Project Status: Completed
- D. Site Plan (see Figure 36)
1. Right of Way: The distance between building lines across D Street varied between 80 and 90 ft (24.38 - 27.43 m)
 2. Buildings and Foundations: The D Street corridor is bound on each side by buildings 5 to 10 stories high. There is a Terminal Refrigeration Building of brick construction, while the Health Education and Welfare and Federal Bureau of Investigation buildings are constructed of reinforced concrete. The foundations of these buildings required support immediately along both sides of the excavation.
 3. Congestion: Traffic was heavy in the area, but final plans called for rerouting traffic around the construction area.

II. Geotechnical Information

- A. Soil Profile: Typical soil profile (taken from boring number IX-7, ground surface elevation +15.9 ft (4.85 m)) consisted of the following materials with thicknesses indicated:

- 10 ft (3.05 m) of silt, sand cinders (fill)
- 13 ft (3.96 m) of medium compact brown silt, fine sand
- 7 ft (2.13 m) of sand and gravel
- 26 ft (7.92 m) of stiff gray-brown clay, trace fine sand, occasional compact brown fine to medium sand, some gravel, trace silt
- 24 ft (7.32 m) of hard mottled brown-gray plastic clay, occasional lignite fragments

Figure 37 shows a detail of the soil profile along the station length.

III. Site Requirements

- A. Constraints or Conditions:

1. The multi-story buildings adjacent to station excavation required a support system to protect their foundations.

* Factual material was obtained from unpublished design documents made available by WMATA and project consultants.

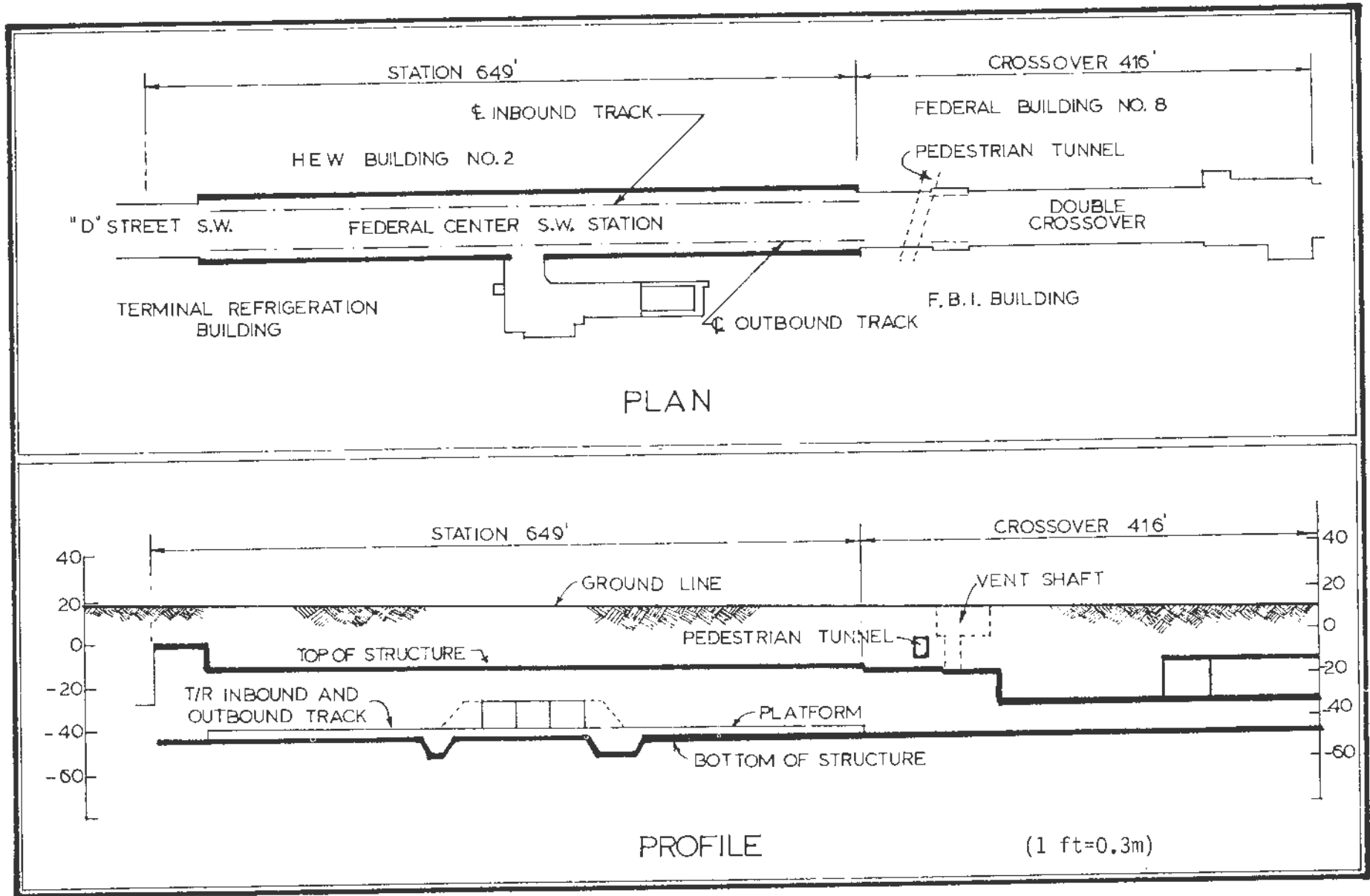


Figure 36. Site Plan And Profile For The WMATA Federal Center S.W. Station (D4b Project)

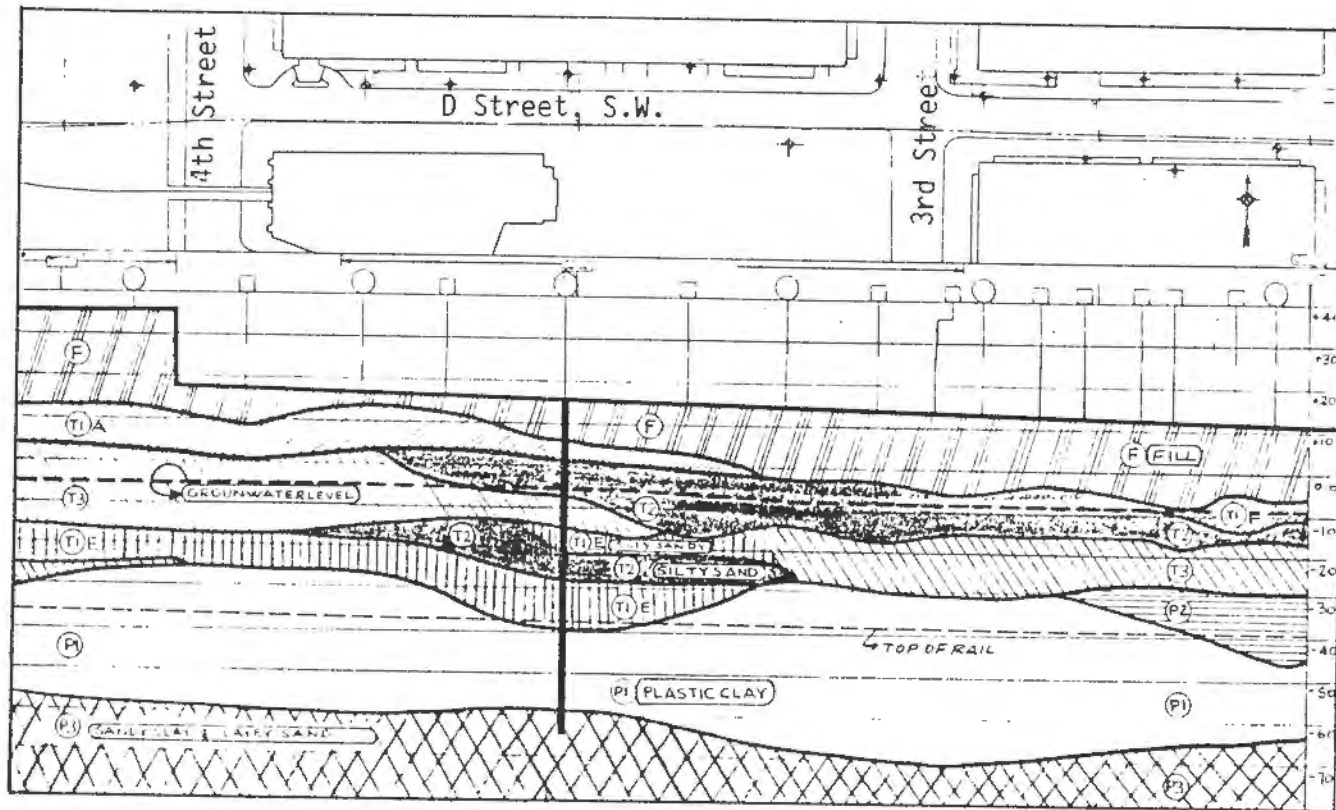


Figure 37. Longitudinal Soil Profile

LEGEND

- F: Fill
- T: Silty Sand
- P: Clay

2. There was limited working room between the building lines along the D Street corridor.
- B. Special Considerations: Station is 649 ft (197.82 m) long; crossover following the station part is 416 ft (126.8 m) long. Station is 70 ft 10 in. wide (21.59 m) and about 35 ft (10.67 m) high. Excavation depth was about 65 ft (19.82 m).

Distance between the excavation and the existing building limit is about 10 ft (3.05 m). Closest foundation footing is about 5 to 8 ft (1.52-2.44 m) from the surface of the wall.

IV. Design Information

- A. Design Criteria and Loading (see Figure 38): Ground water table is 18 to 20 ft (5.49-6.10 m) below the ground surface.

Symmetrical and asymmetrical loading conditions were considered. Symmetrical loading consisted of full hydrostatic pressure, surcharge loads, and soil pressure for which the at-rest pressure is applied on both sides of the frame. The same is true for asymmetric loading, except for the soil pressure for which at-rest pressure on one side and active pressure on the other side of the frame is applied.

- B. Design Alternatives: The section designers offered general contractors a choice of two schemes for supporting the multi-story buildings adjacent to the station excavation. The first alternative required the installation of a conventional jack-pile and steel sheeting system. The second alternative required the construction of a rigid slurry wall.

The first alternative of a conventional flexible wall support system required an extensive underpinning process for the foundations of adjacent structures. A cost study was performed, and results showed the slurry wall alternative to be more economical.

- C. Final Choice: Because the slurry wall alternative eliminated the extensive underpinning requirements, the winning bid indicated a \$2.3 million savings over the conventional alternative. Steel beams with lightweight concrete reinforced slurry walls were installed as a temporary earth support system during station excavation. The slurry wall was designed to carry horizontal loads in excess of 8 ksf (390.61 kg/m²).

V. Results

- A. The Structure: Two parallel slurry walls were constructed along the length of the site. The surface area for each wall was 170,000 sq. ft. (15,790 m²), a thickness of 30 in. (76 cm) and extended to a maximum depth of 80 ft (24 m). Figure 39 shows a typical section of the slurry wall and final station structure. A detailed panel of the slurry wall is shown in Figure 40. The slurry walls were not incorporated into the permanent station structure, though they were left in place and backfilled.
- B. Performance: Field Measurements: Lateral movements of the slurry wall were monitored by inclinometers and maximum deflection was found to be 0.12 in (3.05 mm).

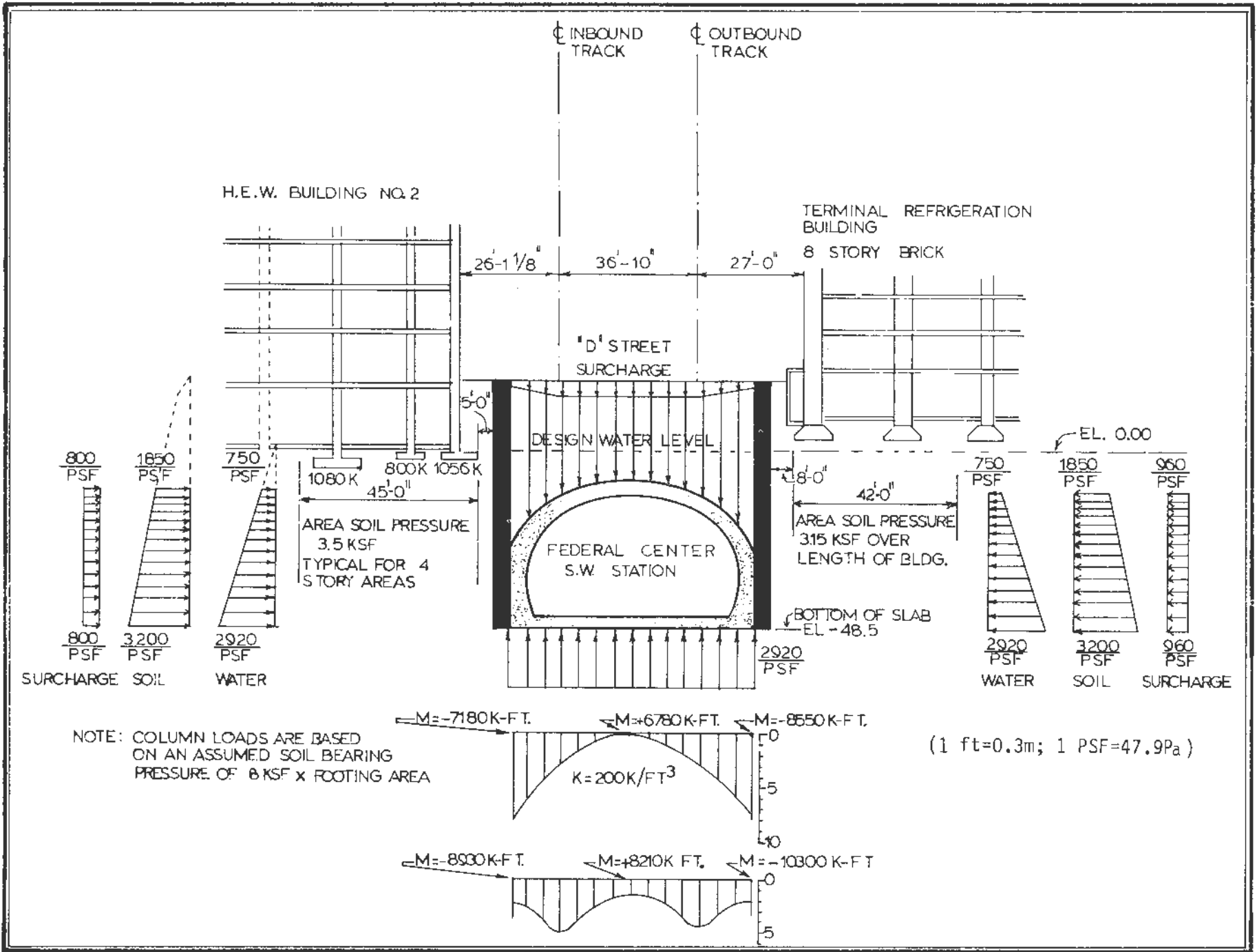


Figure 3f. Long-Term Loading On The D4b Final Structure

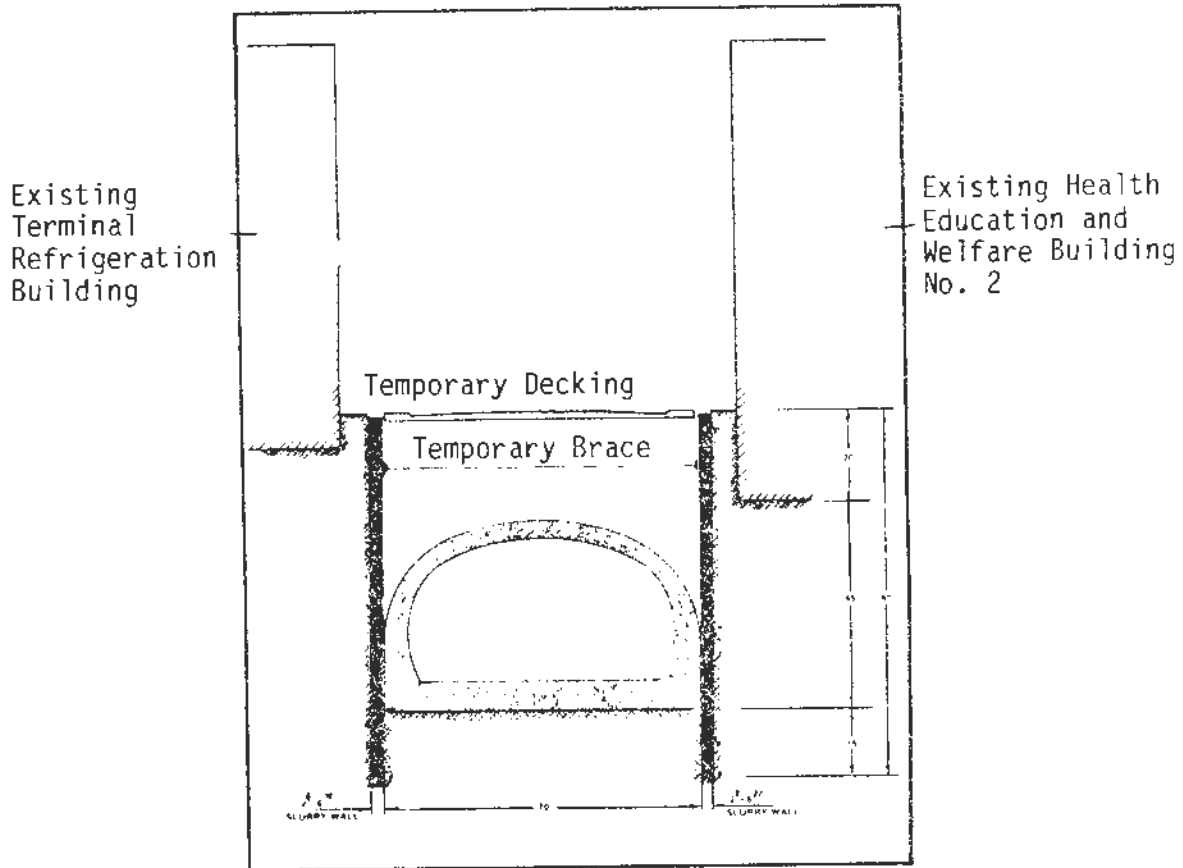


Figure 39. Typical Section

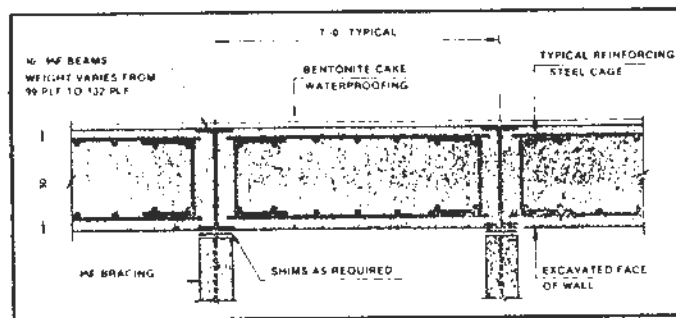


Figure 40. Typical Slurry Wall Panel Detail

I. Project Overview

- A. Location: Baltimore Region Rapid Transit Station
Mass Transit Authority (MTA), Baltimore, MD
- B. Description: Located in downtown Baltimore, the Charles Center Station runs along East Baltimore Street between Hanover and Calvert Streets.
- C. Project Status: Under construction at time of this report.
- D. Site Plan:
 - 1. Right of Way: Figure 41 provides a plan view of the slurry wall locations.
 - 2. Buildings and Foundations: As the Plan (Figure 41) shows, the station is surrounded by several large commercial buildings:
 - 1. Lord Baltimore Hotel
 - 2. B & O Railroad Building
 - 3. W.R. Grace Building
 - 4. Union Trust Co. Buildings
 - 5. Morris Mechanic Theatre
 - 6. Savings Bank of Baltimore
 - 7. Maryland National Bank

It was determined that underpinnings would be required to protect these structures in conventional construction.

- 3. Congestion: Traffic volume in the site area is heavy and is to be maintained. Pedestrian walkways must be provided along both sides of East Baltimore Street to allow access to shops.

II. Geotechnical Information

- A. Soil Profile: Typical soil conditions are summarized in Table 5 along the length of the station.
- B. Ground Water Level: Ground water table ranges from 4 ft (1.22 m) above to 4 ft (1.22 m) below elevation 0. characteristics and normal dewatering practices should be sufficient to lower the water table.

III. Site Requirements

- A. Constraints or Conditions:
 - 1. Protection and support of numerous multi-story buildings must be provided.

* Information and figure obtained through the courtesy of Baltimore MTA.

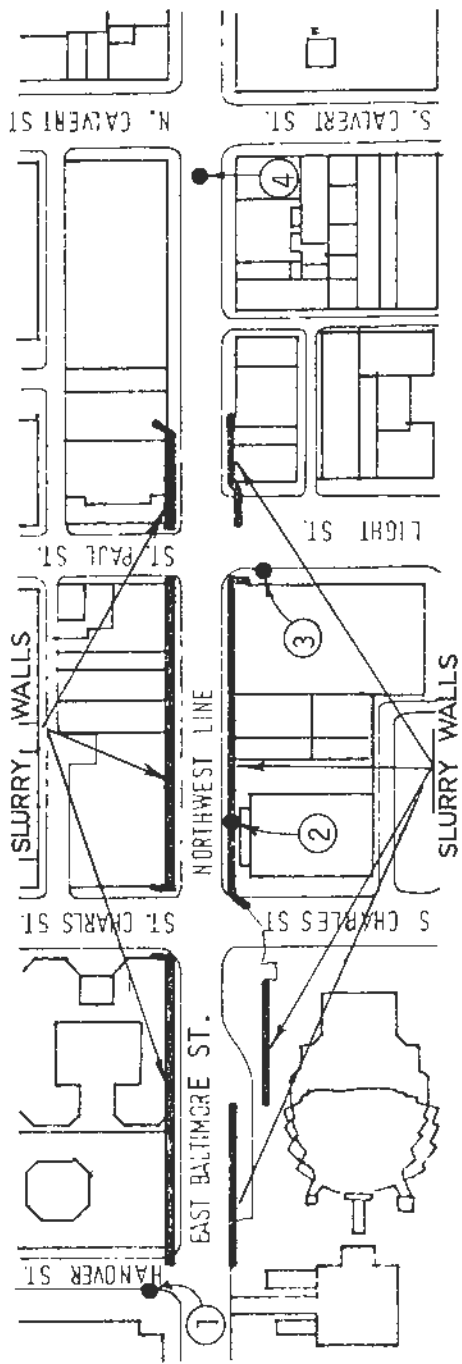


Figure 41. Site Plan

Table 5. Summary of Soil Profile, Charles Center Station

1 B2-7 (E1.42.6)		2 NW-1 (E1.38.2)		3 SOX-1 (EL.35.9)		4 NEX3A (E1.30.5)	
Depth (ft)	Soil Description	Depth (ft)	Soil Description	Depth (ft)	Soil Description	Depth (ft)	Soil Description
	Asphalt Paving		Asphalt Paving		Asphalt Paving		Asphalt Paving
10-	fine to coarse sand fine to medium gravel	10-	clayey silt some sand	10-	fine to medium sand fine gravel	10-	fine sand silt & clay fill
20-	fine to coarse sand silt	20-	fine to coarse sand some clay fine to coarse sand	20-	silty clay	20-	fine sand some gravel
30-	fine to medium gravel fine to coarse sand	30-	fine to medium gravel	30-	GWT. fine to coarse gravel & sand	30-	GWT fine sand fine to medium gravel
40-	GWT fine to coarse sand	40-	GWT	40-		40-	
50-	small gravel	50-	clay & silt some fine to medium sand	50-	fine to coarse sand some gravel	50-	some clay & silt
60-	fine to coarse sand clayey silt	60-		60-	medium gravel fine sand	60-	fine sand clayey silt
70-	silt	70-	fine to coarse sand rock fragments	70-	clay & silt some sand	70-	clayey silt & sand
80-	fine to medium sand	80-	rock	80-		80-	rock
90-	decomposed rock	90-		90-	rock fragments rock	90-	
100-		100-		100-		100-	

GWT - Ground Water Table
Elevations as shown; exact elevations may fluctuate seasonally by 4 ft.-.

2. Site must be open to vehicular traffic through construction corridor.
3. Existing shops and commercial businesses along the corridor must be accessible.
4. Pedestrian traffic through construction corridor be maintained.
5. In vicinity of Lord Baltimore Hotel the sound level must be limited to 70 db after 10:00 pm.
6. Ground water must be controlled to prevent damage to adjacent structures.

IV. Design Information

- A. Site Condition and Loading: Because of the limited space between the face of existing buildings and the exterior of the subway structure (a minimum of approximately 3 ft (0.91 m)) and because of the underpinning requirements for existing structures, it was felt that a rigid earth support system would provide greater protection for the buildings than a conventional flexible earth retaining systems. The integral wall construction was considered as a method to utilize slurry walls as part of the complete structure, so that the final structure would be capable of withstanding the excessive stress of deflection.
- B. Design Alternatives: Composite construction with a tremie concrete slurry wall and a conventional rigid frame structure was considered. Two concepts were investigated.
 1. The slurry wall will carry the lateral forces directly to the floor levels, while the internal structure carried unbalanced lateral loads and moments from roof and slabs. In this case, the slurry wall is not in direct contact with the rigid frame.
 2. The slurry wall is to share all long term loads. In this case, the slurry wall is in direct contact with the rigid frame.

V. Results

- A. The Structure: Slurry wall (steel beam and concrete) was used as temporary earth support to:
 1. Reduce the extent of the underpinning required.
 2. Reduce the settlement of adjacent buildings.
 3. Reduce the cost of dewatering.

I. Project Overview

- A. Location: Baltimore Region Rapid Transit System
Mass Transit Authority (MTA)
- B. Description: Subway Station and Vent Shafts in downtown Baltimore to consist of a concrete box section 450 ft (137.16 m) long.
- C. Project Status: Under construction at time of this report.
- D. Site Plan (see Figure 42)
 - 1. Right of Way: Along Pennsylvania Avenue at its intersection with Laurens Street, the work area has a mere 66 ft right of way.
 - 2. Buildings and Foundations: Existing structures consist primarily of two to three story brick row houses. There are two other structures, however, that merit special attention because of their size and function. These structures, Lafayette Market and the new Post Office Building, are shown in Figure 42.

II. Geotechnical Information

- A. Soil Profile: The site provides a variety of soil conditions. An overall soil profile is presented in Figure 43.
- B. Ground Water Level: Ground water table ranges between 15 to 25 ft (4.57-7.62 m) below ground surface.

III. Site Requirements

- A. Conditions or Constraints:
 - 1. Because of a narrow 66 ft (20.12 m) right-of-way, street construction clearance was minimal. Consequently, the behavior of ground and minimization of ground movement was of prime concern.
 - 2. Dewatering efforts were limited to the use of deep wells. The Amtrack Wilson Station is in the vicinity, therefore, dewatering efforts must not lower the ground water level in this area.
 - 3. Two important structures, the Lafayette Market and the U.S. Post Office, because of age, structural condition, and function, would require certain minimum protection.

IV. Design Information

- A. Design Criteria and Loading: Temporary earth support would be provided during excavation. A concrete box section 450 ft (137.16 m) long would be constructed in the excavated area to serve as station. Total force against temporary support system (flexible type) was taken as 1.4 times the total active force. In the case of a rigid wall system, design loads would be

* Information and figures obtained through the courtesy of Baltimore MTA.

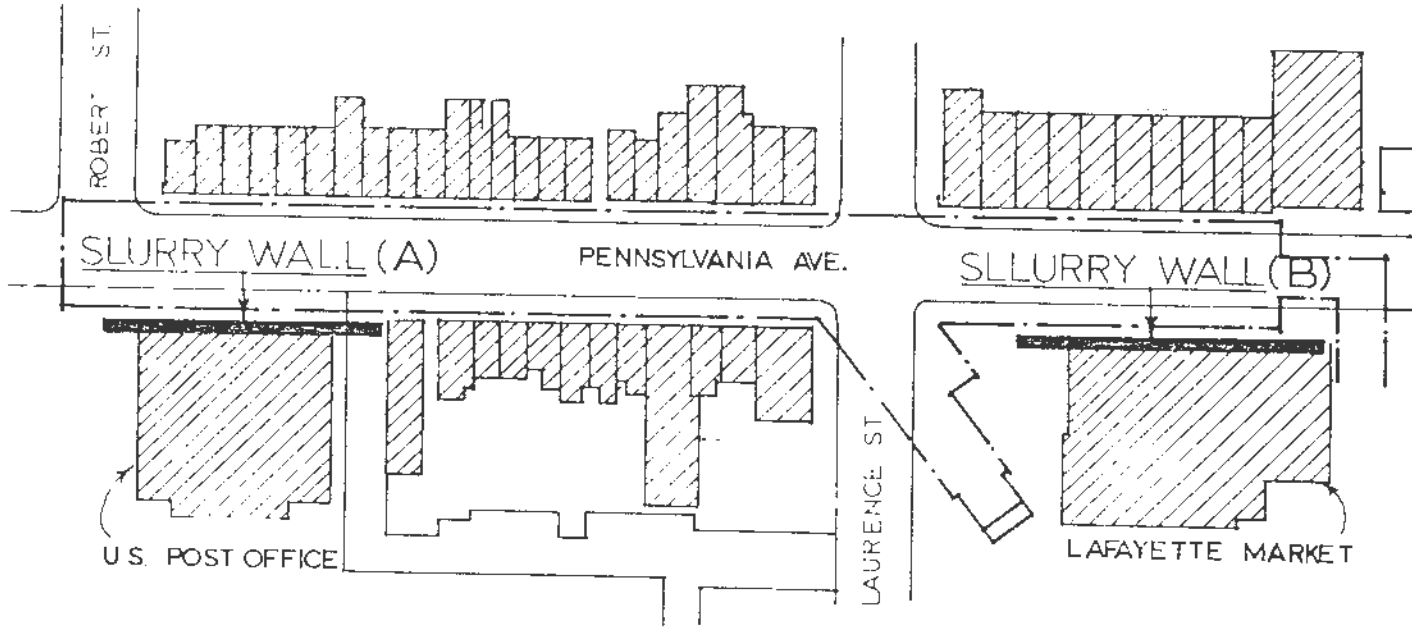
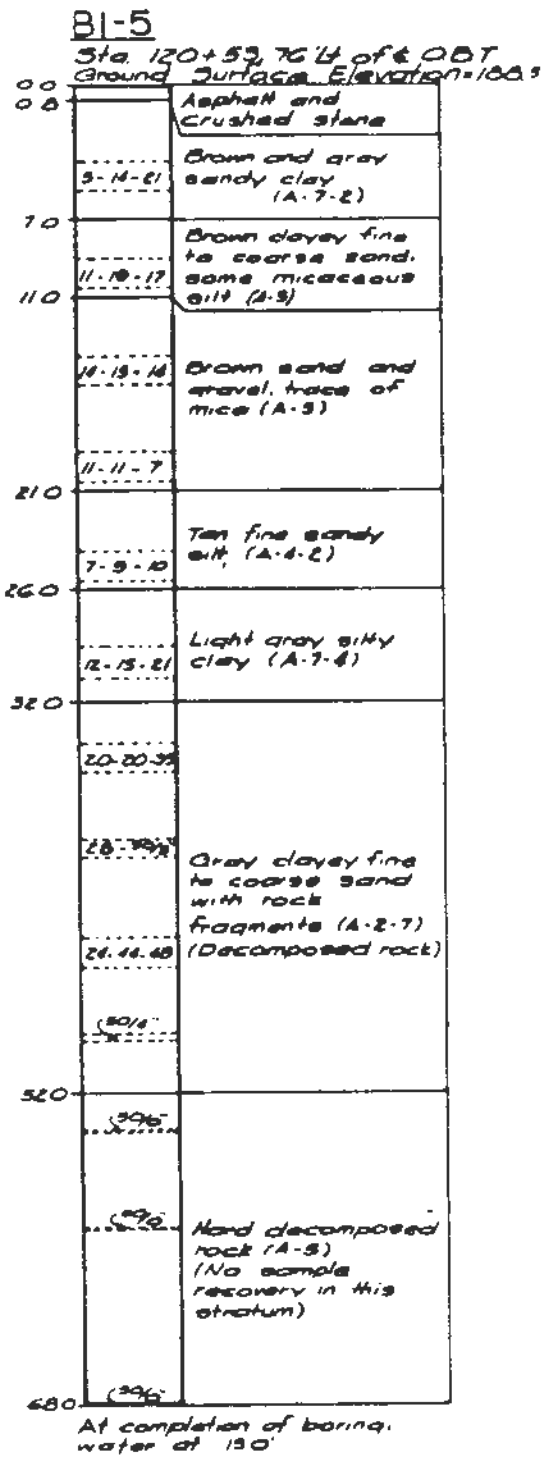
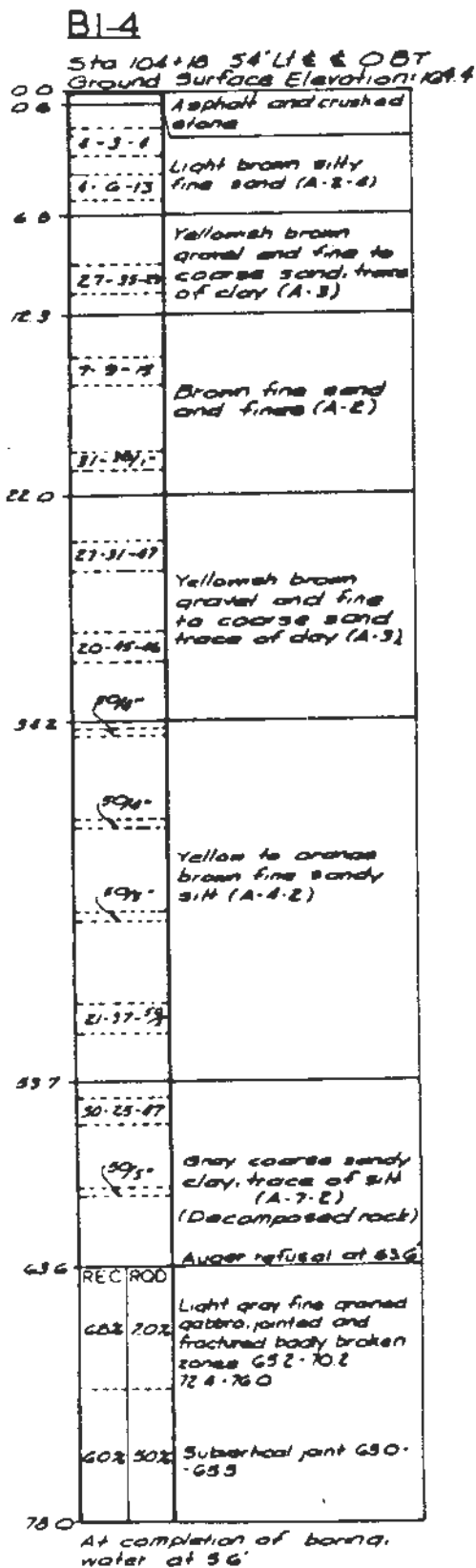


Figure 42. Site plan



(1 ft=0.3m)

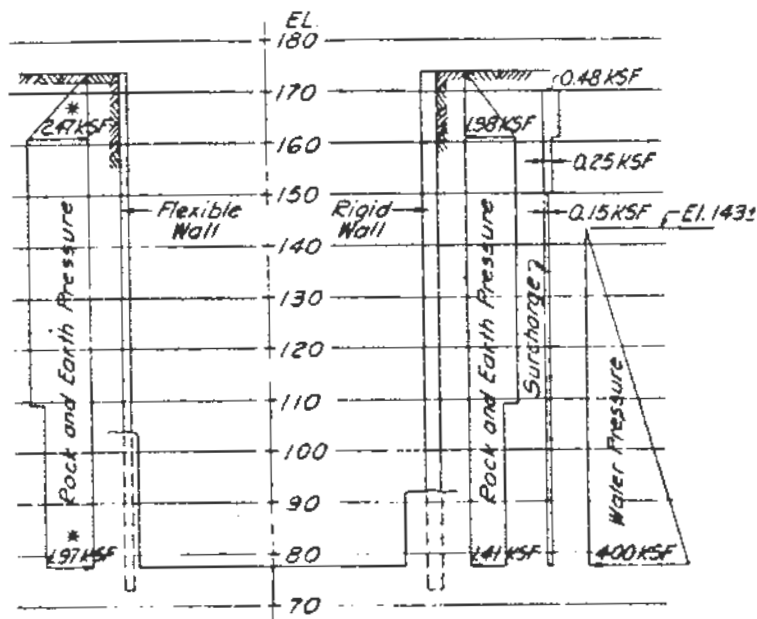
Figure 43. Typical Soil Boring Charts

equal to the total horizontal load due to earth, rock, and ground water.

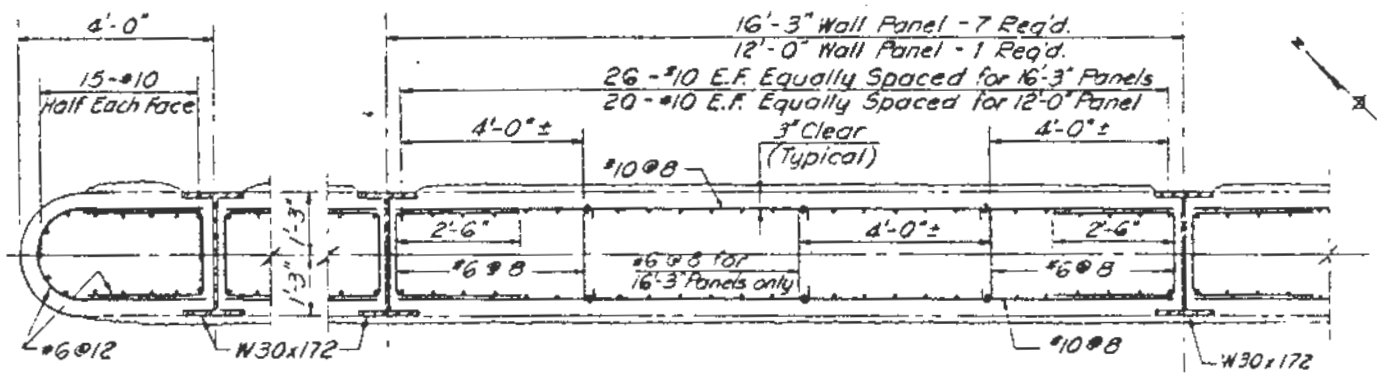
- B. Design Alternatives: Conventional soldier pile and lagging (a flexible wall system) along with controlled dewatering was considered for the majority of the station. However, a rigid wall system would provide support in the vicinity of Lafayette Market and the U.S. Post Office Building.
- C. Final Choice: The type of support system to be used remains the option of the contractor, except for that portion of the Durid Station system located in the vicinity of the existing Lafayette Market and U.S. Postal Services.

In front of the future U.S. Post Office Building, a Steel Beam and Concrete type slurry wall ("A" of Figure 42) 135 ft 9 in. (40.77 m) in length is to be installed. The wall is to extend to an elevation of 75 ft (22.86 m) having an average depth of 90 ft (27.43 m) which will be anchored in bedrock. Figure 44a and 44b show the lateral design pressures and details of the Slurry Wall Panel.

To provide support for the Lafayette Market Building a Steel Beam Tremie Concrete type slurry wall ("B" of Figure 42), 142 ft (43.28 m) in length will be installed as a temporary earth support. The wall will extend some 80 ft (24.38 m) into the ground where it will be anchored in bedrock. Figure 45a shows the lateral pressure used in the design, and Figure 45b shows typical wall details.

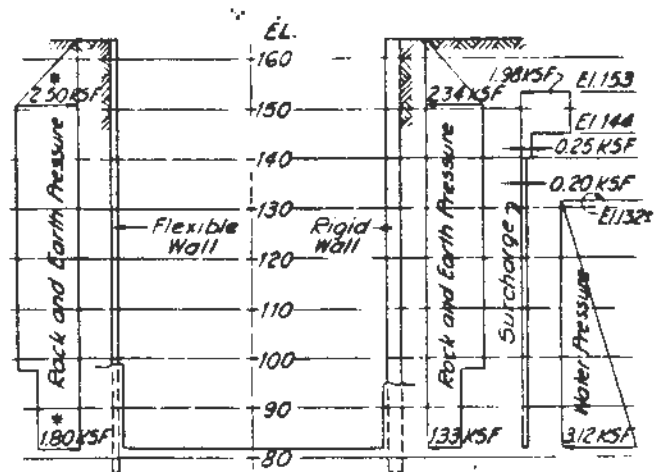


a. Lateral Pressures For Design Of Braced Excavation



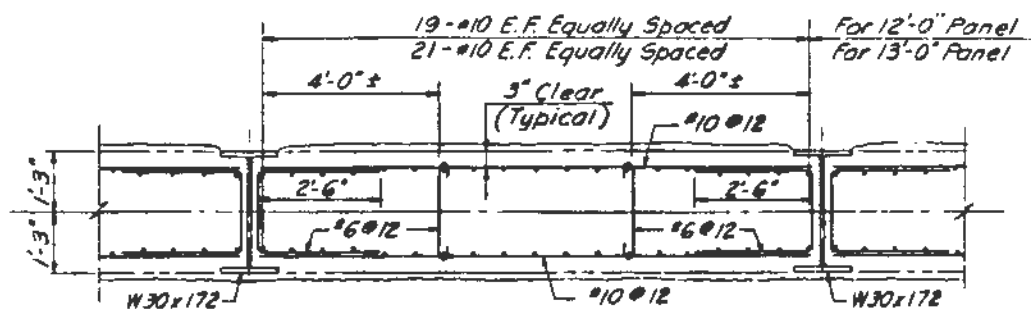
b. Typical Panel Details Of Slurry Wall (a) (1 ft=0.3m)
(Scale 1/2" = 1' - 0")

Figure 44. Lateral Design Pressures And Panel Details
(Support Of U.S. Post Office Building)



a. Lateral Pressures for Design of Braced Excavation Supporting Lafayette Market

(1 ft=0.3m)



SECTION 1-1 (12'-0" Panel shown)
SECTION 2-2 (13'-0" Panel similar)
Scale: 1/2" = 1'-0"
5-123

b. Typical Panel Details for Slurry Wall (B)

Figure 45. Lateral Design Pressures and Panel Details (Support of Lafayette Market)

I. Project Overview

- A. Location: Baltimore Region Rapid Transit System
Mass Transit Authority (MTA), Baltimore, MD
- B. Description: Subway Station in downtown Baltimore along Pennsylvania Avenue.
- C. Project Status: Under construction at time of this report.
- D. Site Plan: (See Figure 46.)
 - 1. Right of Way: Station runs along Pennsylvania Avenue between R Street and W. North Avenue. The distance between building faces along this corridor is about 60 to 70 ft.
 - 2. Buildings and Foundations: This section of the Baltimore MTA system is lined with old two and three story brick residential and commercial buildings. Some of these structures have shallow basements. Three other buildings have wood frame floors. The general condition of these structures is thought to range from poor to fair. Therefore, slurry walls should be used in front of these buildings.

II. Geotechnical Information

- A. Soil Profile: The soil profile can be summarized as follows using approximate depth values:

- 5 - 10 ft (1.52-3.05 m) fill (manmade)
- 10 - 20 ft (3.05-6.10 m) residual soil
- 20 - 40 ft (6.10-12.19 m) residual zone no. 1 (soil-like)
- 40 and below ft (12.19 m) residual zone no. 2 (rock-like)

Figure 47 provides a detailed profile of the station site and actual results of a typical soil boring made at the site.

- B. Ground Water Level: Ground water table is approximately 10 to 20 ft (3.05-6.10 m) below the surface (assumed to be 12 ft (3.66 m) overall). The depth of the rock surface to which water has to be lowered, is estimated to vary from 41 to 84 ft (12.50-25.60 m).

III. Site Requirements

- A. Constraints or Conditions:
 - 1. Site is located in downtown urban area.
 - 2. Traffic and utilities must be maintained.
 - 3. Ground water level must be controlled and lowered to rock level.

*Information and figures obtained through the courtesy of Baltimore MTA.

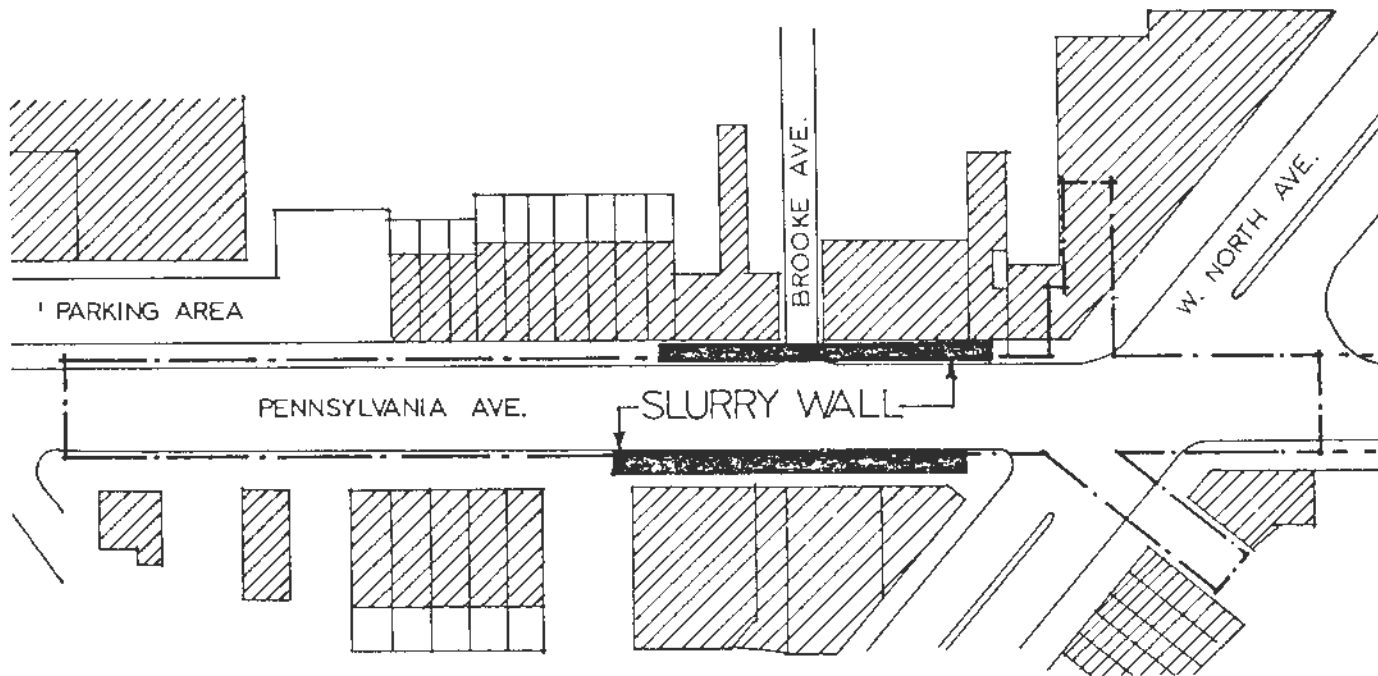
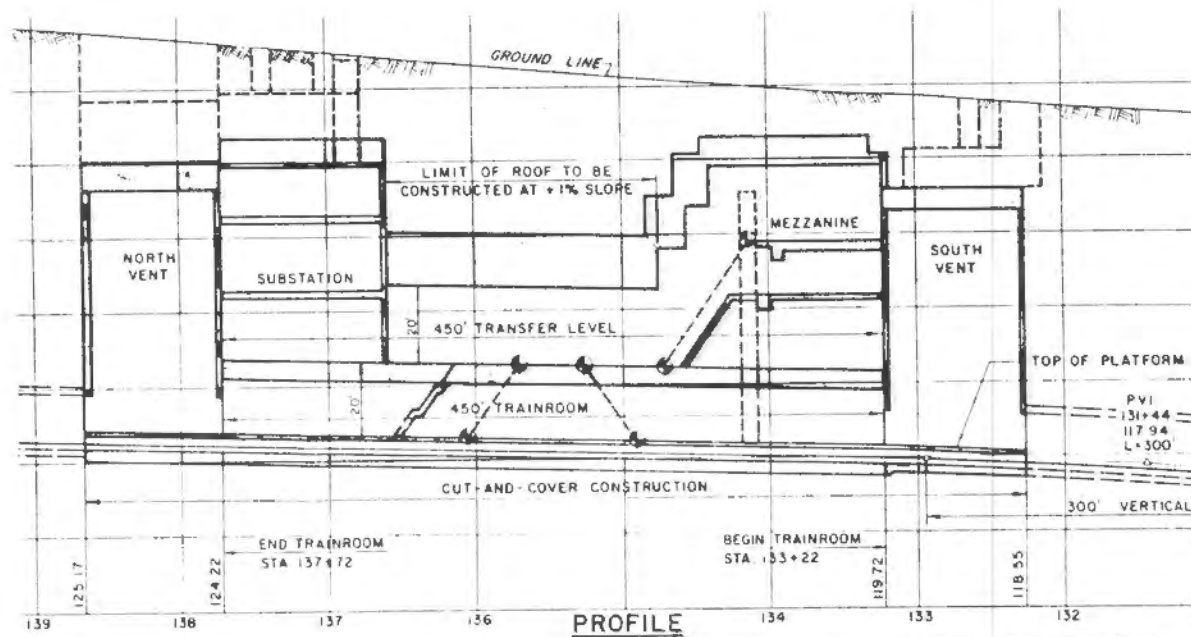


Figure 46. Site Plan

(courtesy of Baltimore MTA)



a. Typical Station Profile (1 ft=0.3m)

STRATA DEPTH DESCRIPTION (Color, Density, Moisture, Proportion, etc.)

8.0	Brown moist of SAND, some clayey silt, tr. mf gravel, tr. brick frgmts (Fill) Same (Fill) 214.6
13.0	Brown moist Silty CLAY, some of sand, little mf gravel (CL) 209.6
23.0	Brown & Lt. green CLAY, tr. fine sand (MH, RS) Greenish gray moist CLAY & SILT and of SAND, tr. fine gravel (MH, RS) 199.6
38.0	Gray & Brown moist of SAND and Clayey SILT, tr. mica, tr. mf rock frgmts (RZ-1, SM) Green & Brown moist of SAND, some silt & clay, tr. mica, tr. mf rock frgmts (RZ-1, SM) Same (RZ-1, SM) 184.6

a. Typical Boring Log

Figure 47. Station Profile and Boring Log

which may impair the stability of soil mass and accelerate the decomposition of rock.

4. The distribution of fissures in the rock, with respect to the face of excavation, will be an important factor in the excavation.

B. Special Consideration: Building age and structural sensitivity.

IV. Design Information

- A. Design Criteria and Loading: Design specified the use of a steel beam and concrete slurry wall as temporary ground support during the excavation for those commercial buildings indicated on the site plan. In the design calculations, an extra 5 ft (1.52 m) of head was provided to account for seasonal fluxuations in the height of ground water.
- B. Design Alternatives: Two alternatives offered for use were either a conventional soldier pile and lagging or a rigid type slurry wall which would extend into the rock surface. These walls would be held by internal bracing or tiebacks.
- C. Final Choice: Selection between a conventional soldier pile and lagging or a slurry wall was left to the contractor. If soldier piles and lagging with tiebacks is selected, control of ground water would be necessary to keep the excavation dry and to prevent the loss of soil particles through water seepage. The space between the lagging should be filled with hay or plastic fillings. This is critical in the areas where adjacent structures are protected.

If a slurry wall is to be used, the rigidity of such a wall minimizes the ground movements. Tiebacks for part of the slurry wall are located on only one side of the excavation. The slurry wall will be anchored at least 2 ft (.61 m) into the rock.

I. Project Overview

- A. Location: Metropolitan Boston Transit Authority (MBTA)
Boston, Massachusetts
- B. Description: Red Line Extension, Harvard Square Station
- C. Project Status: Under construction
- D. Site Plan: (See Figure 48.)

II. Geotechnical Information

Soil Profile: (See Figure 49.)

III. Site Requirements

Conditions or Constraints: The site is unique because of the presence of historical buildings with questionable structural integrity.

IV. Design Information

- A. Design Criteria and Loading: Rigid walls must be used to prevent ground movement. The underpinning should be eliminated, since it is noisy and harmful to historic buildings. Ground water drawdown must be minimized.
- B. Design Alternatives: In some areas, preliminary design investigations indicated that a flexible wall (soldier pile and lagging with piles drilled into the ground) would be adequate. For most areas, a flexible wall would not be suitable because of the expected excessive settlement which would be harmful to the adjacent historic sites of questionable structural integrity. For convenience, a rigid slurry wall is chosen for the entire area. Design criteria imposed symmetrical loading conditions on the final structure. This led to the need of a 48 in. (12.2 m) thick slurry wall. Problems with rebar placement, tolerances, joints, site space, equipment limitations, and utilities all presented problems which seemed insurmountable. Final decision called for a composite wall, with a combination of a 24 in. (0.61 m) slurry wall and a 42 in. (1.07 m) in-situ reinforced concrete wall. No shear connection was provided between the slurry wall and the concrete wall. A composite design was considered and rejected because of difficulties in providing an adequate shear connection between them.

V. Final Choice

Slurry walls were used as temporary support and later incorporated into an integral part of the permanent structure. Figure 50 shows details of the final design.

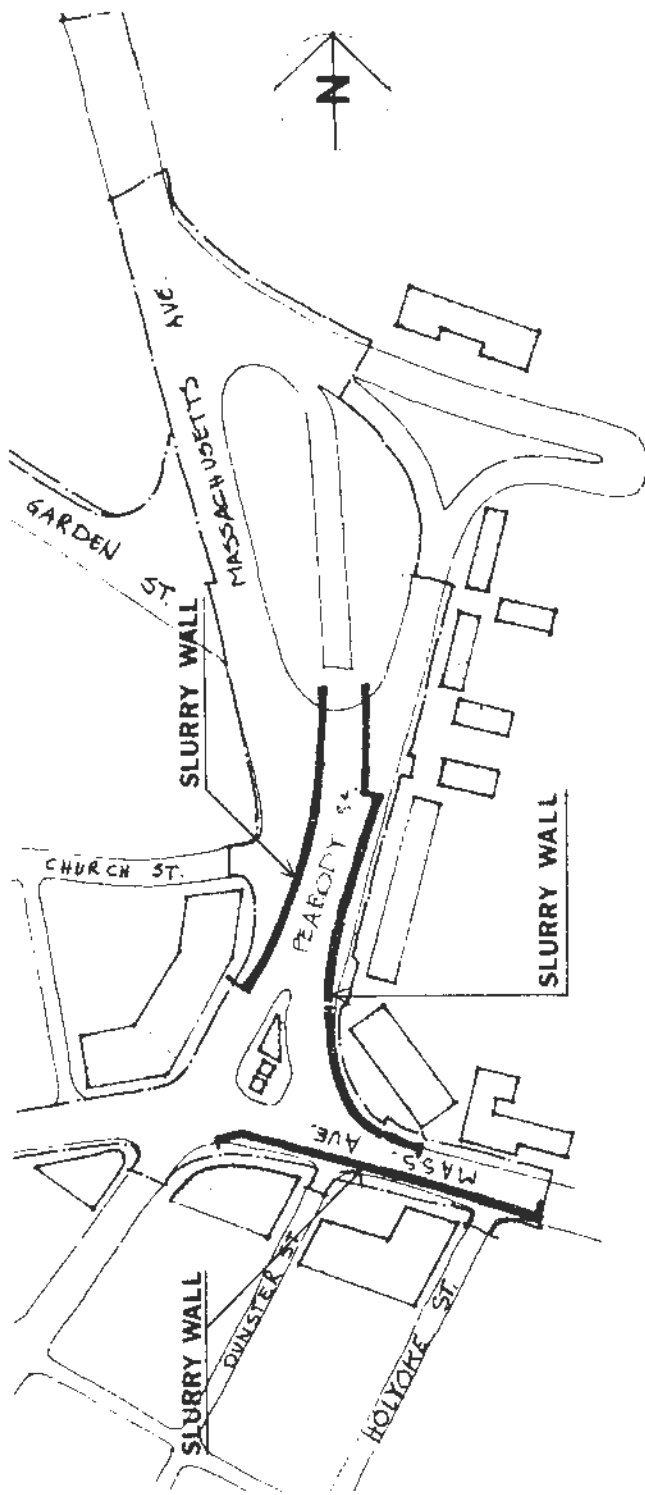


Figure 48. Site Plan

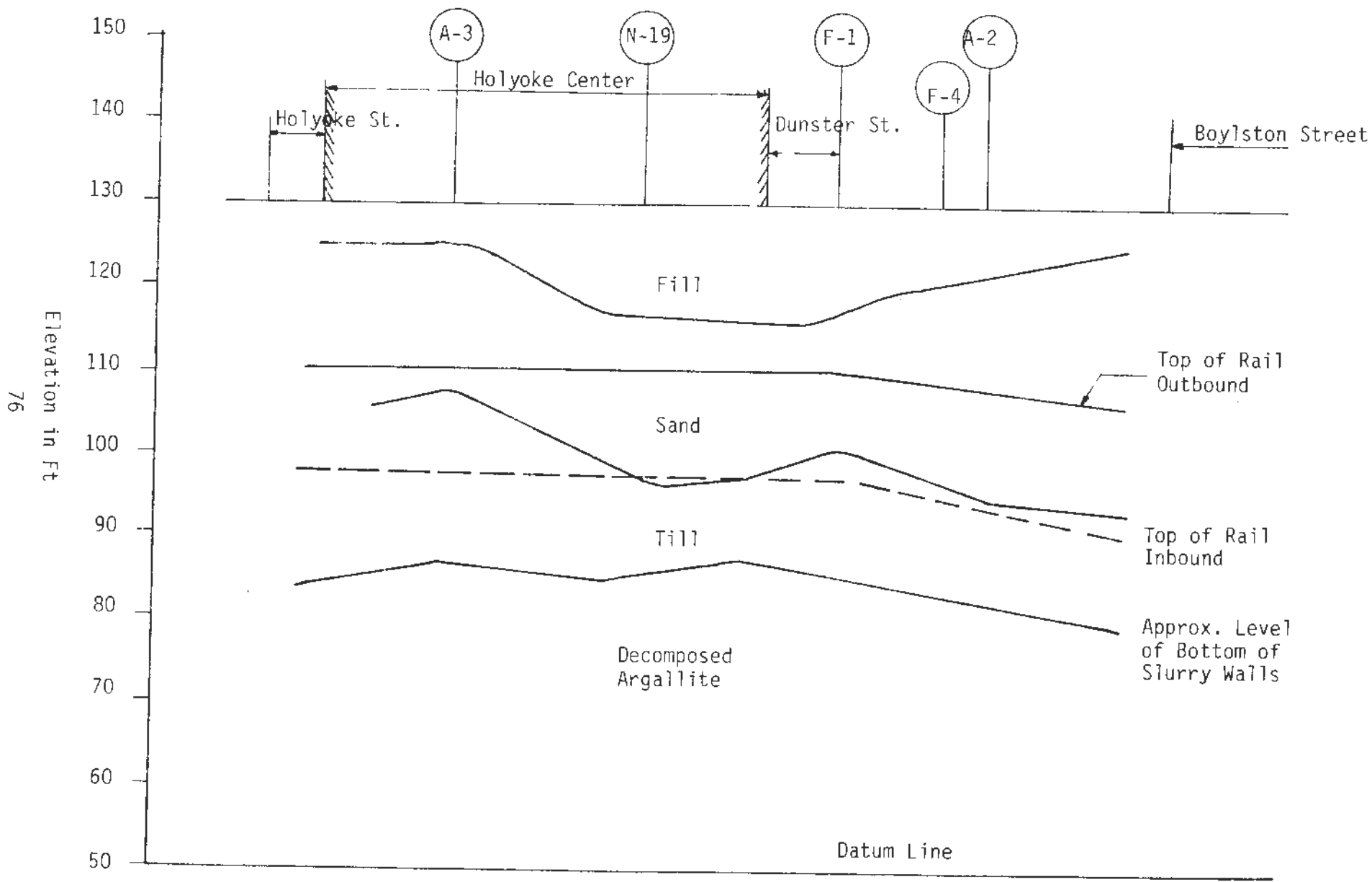
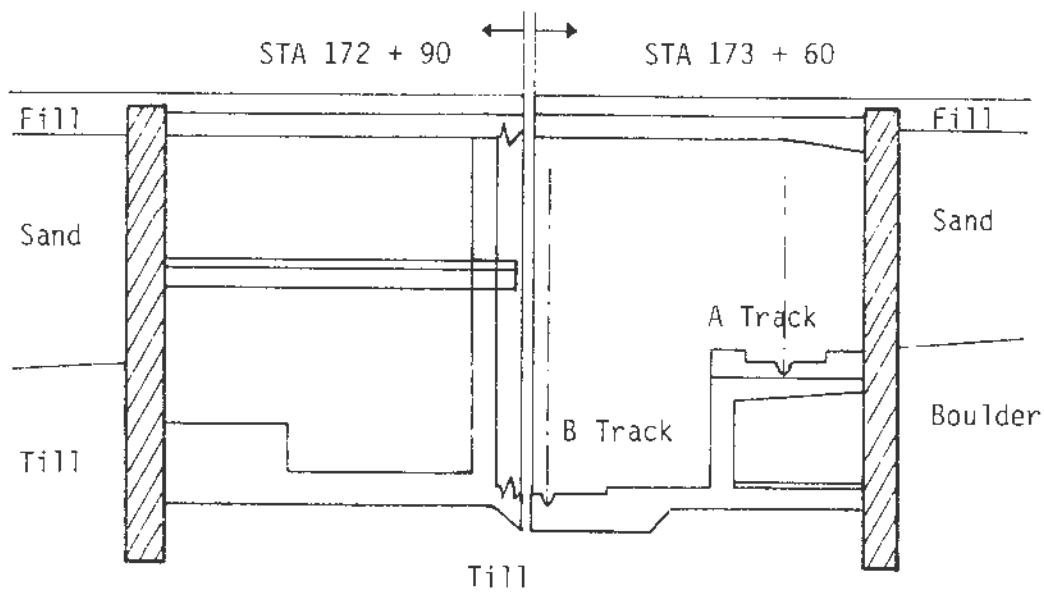
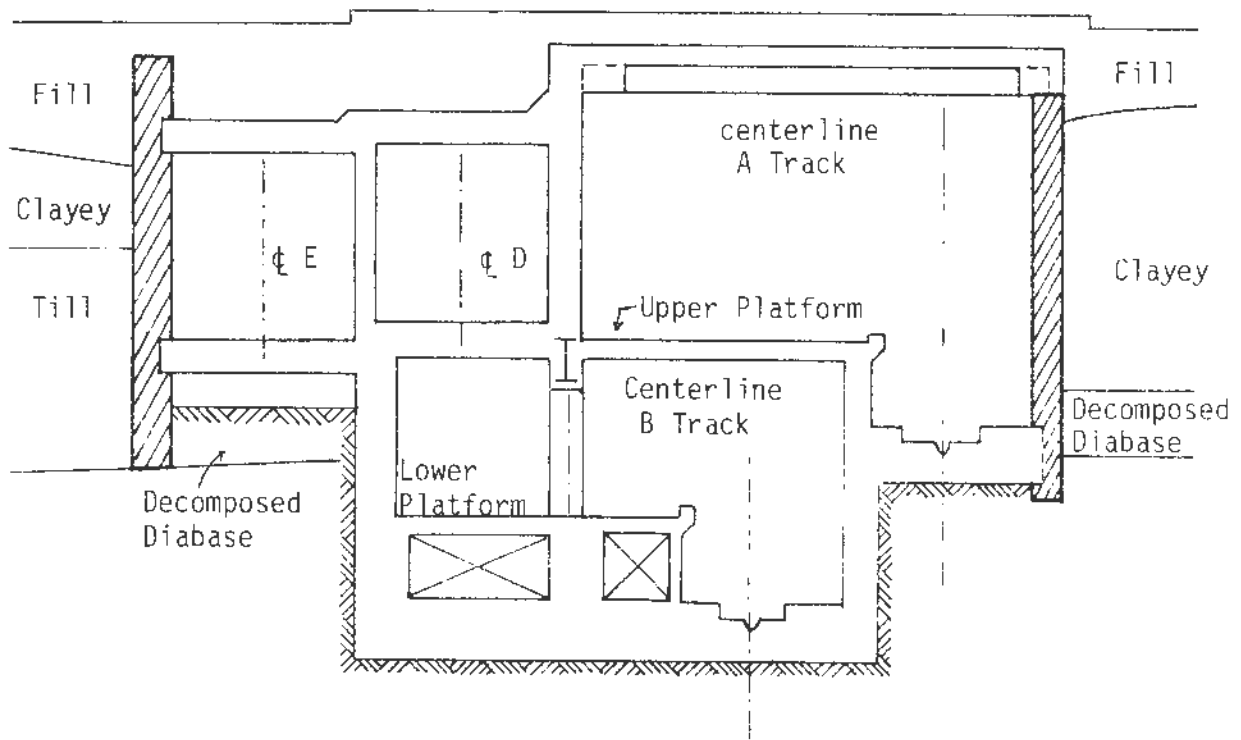


Figure 49. Typical Soil Profile



Decomposed Argillite

Figure 50. Typical Station Sections with Slurry Wall

I. Project Overview

- A. Location: Metropolitan Boston Transit Authority (MBTA)
Boston, Massachusetts
- B. Description: Red Line Extension, Davis Square Station
- C. Project Status: Under construction.
- D. Site Plan: (see Figure 51)

II. Geotechnical Information

- A. Soil Profile (see Figure 52)

III. Site Requirements

Conditions or Constraints: The site is on a saturated, fine grained soil stratum with ground water table 13 ft (3.96 m) below surface. The construction must accommodate an 84 in. (2.13 m) diameter storm drain.

IV. Design Information

- A. Design Criteria and Loading: The ground water would cause settlement in adjacent structures which could lead to a serious problem. Hence, an asymmetrical, lateral loading with unbalanced hydrostatic pressure should be considered.
- B. Design Alternatives: Conventional soldier pile and lagging or slurry wall.
- C. Final Choice: Narrow space, dewatering restrictions, need to avoid underpinning and desire to incorporate the lateral support system as part of the permanent structure led to the choice of a slurry wall.

*Information and figures obtained through the courtesy of the MBTA and engineering and architecture firms working on the station design and construction.

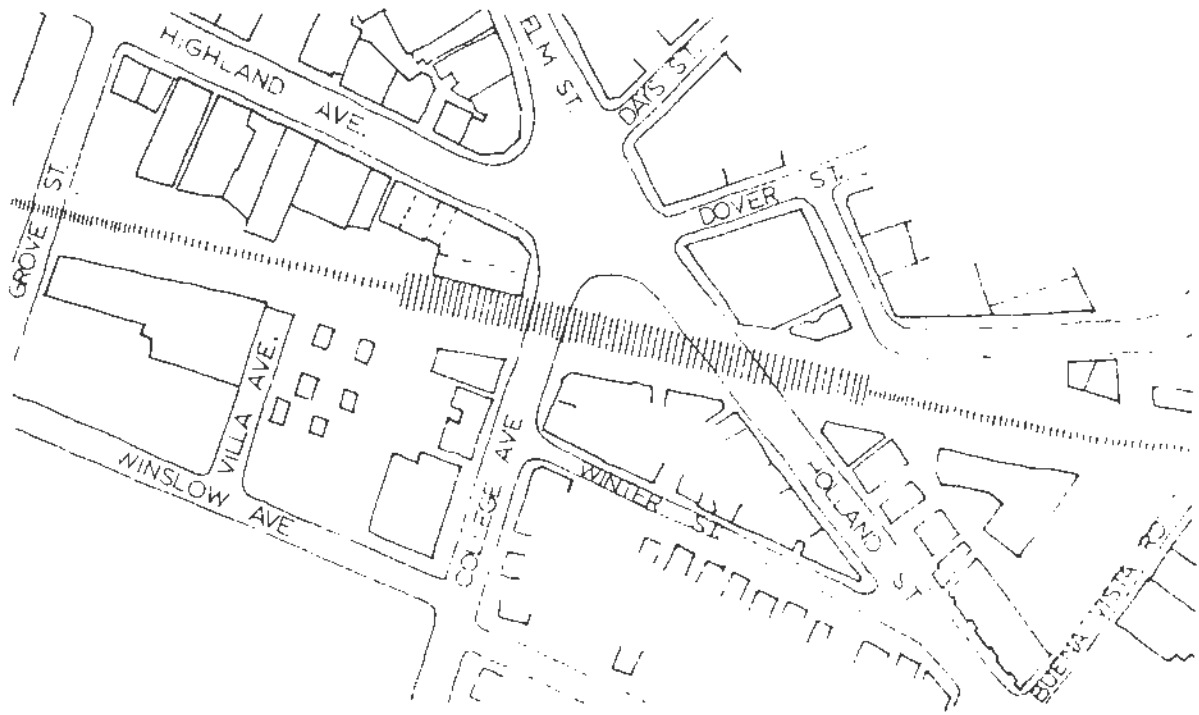


Figure 51. Site Plan

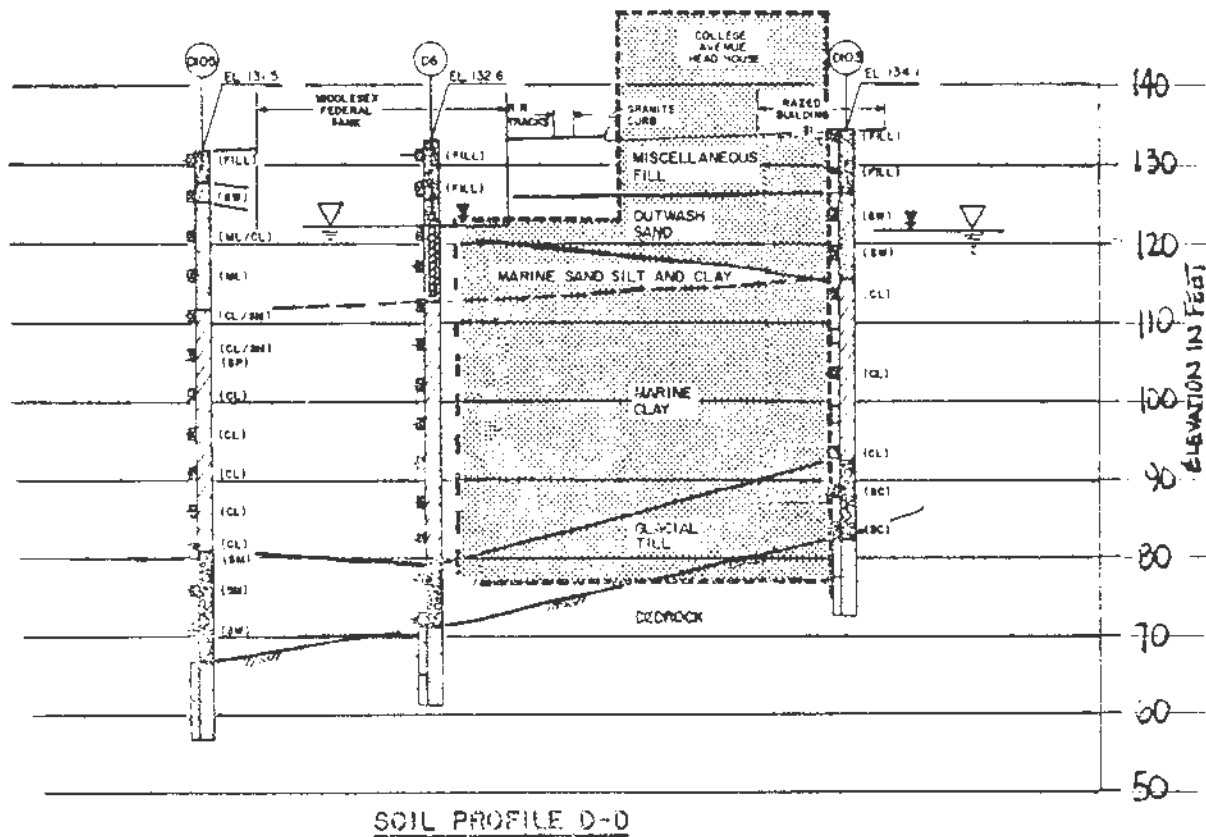


Figure 52. Soil Profile

i. Project Overview

- A. Location: Metropolitan Boston Transit Authority (MBTA)
Boston, Massachusetts
- B. Description: Red Line Extension, Davis Square to Alewife
- C. Project Status: Under construction
- D. Site Plan: (see Figure 53)
- E. Summary of site conditions: 1-1/2 mile (2.41 km) cut-and-cover tunnel project. Typical depth of excavation is 35 ft (10.67 m). The ground water table is 15 to 30 ft (4.57-9.14 m) below ground surface.

II. Geotechnical Information

- A. Soil Profile: The site consists of a variety of soil profiles, broken down into 7 sections.

- Section 1: 10 ft (3.05 m) fill
8 ft (2.44 m) fine to medium sand
4 ft (1.22 m) sand to stratified clay
25 ft (7.62 m) glacial till bedrock
2 ft (0.61 m) fill
- Section 2: 8 ft (2.44 m) fine to medium sand
15 ft (4.57 m) fine to medium sand
13 ft (3.96 m) stiff clay
3 ft (0.91 m) glacial till bedrock
- Section 3: 8 ft (2.44 m) fill
65 ft (19.81 m) sand (fine to medium, fine, and stratified with clay) glacial till
- Section 4: 4-10 ft (1.22-3.05 m) fill
32 ft (9.75 m) sand (fine to medium over fine)
20 ft (6.10 m) sand stratified with clay
20 ft (6.10 m) clay stratified with sand
very soft clay
- Section 5: 4 ft (1.22 m) fill
20 ft (6.10 m) sand (fine to medium over fine)
30 ft (9.14 m) sand stratified with clay
clay stratified with sand
very soft clay
- Section 6: 6-8 ft (1.83-2.44 m) fill
8 ft (2.44 m) fine sand
6 ft (1.83 m) stiff to medium clay and clay stratified with sand
- Section 7: 8 ft (2.44 m) fill
25 ft (7.62 m) sand stratified with clay
clay stratified with sand
8 ft (2.44 m) stiff to medium clay
very soft clay

III. Site Requirements

- A. Constraints or Conditions: The Red Line Extension between Davis Square and Alewife is a one-and-a-half mile (2.41 km) cut-and-cover tunnel project.

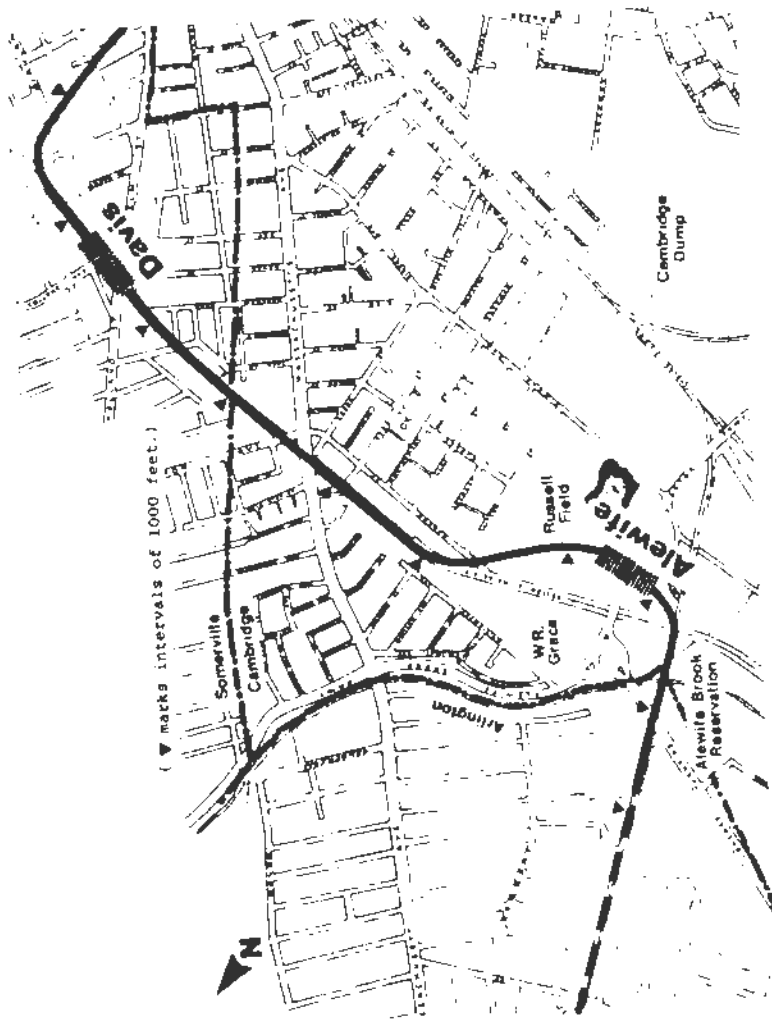


Figure 53. Site Plan

majority of tunnel will follow a narrow railroad right of way which must be maintained during construction.

- B. Special Consideration: Environmental restriction required a low noise level and minimum or no disruption to railway and street traffic.

IV. Design Information

- A. Design Criteria and Loading: Soil conditions required dewatering which was a major factor in the decision-making process.
- B. Design Alternatives: Soldier pile and lagging can be used in most of the sections if major dewatering efforts are included. Slurry wall is a viable alternative to soldier pile and lagging.
- C. Final Choice: Based upon site requirements and design criteria, slurry wall offered the best alternative because it could meet the following requirements:
 - 1. Minor dewatering requirements.
 - 2. Minimum noise.
 - 3. No disturbance to railroad traffic.
 - 4. Minimal disruption to vehicular traffic during construction.

A summary of the major slurry wall sites in the U.S. are summarized in Table 6.

It is shown in Table 6, that even in the limited examples available, a wide variety of soil conditions were encountered, as well as a wide variety of ground water levels and surcharge loads; attention is called to the unusually deep excavation in the Baltimore site (over 100 ft) (30.48 m) and the use of tiebacks instead of bracing. The last column lists reasons for the selection of slurry walls over conventional methods.

Table 6. Summary Of Facts For Major Slurry Wall Sites In The United States

SUMMARY OF FACTS FOR MAJOR SLURRY WALL SITES IN THE UNITED STATES		I BART		II WMATA	III BMTA			IV MBTA		
		A	B		A	B	C	A	B	C
SITE INFORMATION										
SOIL CONDITIONS	SAND & CLAYEY SILT	SOFT CLAY LAYERS BETWEEN SAND & SILT	SAND, GRAVEL W/SOFT & STIFF CLAY	SAND, CLAYEY SILT W/SOME GRAVEL	SAND & GRAVEL W/SOME CLAY	FILL OVER SAND & MOIST CLAY W/ROCK FRAGMTS	FILL OVER SAND, CLAY, & TILL	FILL OVER MARINE SAND & CLAY OVER TILL	FILL OVER SAND, SOFT CLAY AND TILL	
UTILITIES	REMOVED & RELOCATED	HEAVY/ RELOCATED	RELOCATED	RELOCATED	RELOCATE	NA	RELOCATE	NA	NA	
TRAFFIC	HEAVY/ MAINTAINED	HEAVY/ REROUTED	REROUTED	HEAVY/ MAINTAINED	MAINTAIN	NA	HEAVY/ MAINTAIN	HEAVY/ MAINTAIN	MAINTAIN OPER. RAIL ROAD	
GROUNDWATER TABLE	15 ft. BELOW GRADE	15 ft. BELOW GRADE	18 ft. BELOW GRADE	30 to 40 ft. BELOW GRADE	15 to 25 ft. BELOW GRADE	12 ft. BELOW GRADE	NA	13 ft. BELOW GRADE	3 to 15 ft. BELOW GRADE	
SURCHARGE	8 STORY COMMERCIAL BUILDINGS W/ OLD FOUNDATNS	5 to 32 STORY BUILDINGS	HEAVY, 5 to 10 STORY BLDGS.	MULTISTORY COMMERCIAL BUILDINGS	2 LARGE, OLD BLDGS. REQUIRE SPECIAL SUPPORT	3 LARGE, OLD BLDGS. REQUIRE SPECIAL SUPPORT	HISTORIC BLDGS. & LOW RISE COMMERCIAL BUILDINGS	LOW RISE COMMERCIAL BUILDINGS	OPERATING RAIL ROAD WITHIN RIGHT-OF-WAY	
SITE DIMENSIONS										
RIGHT-OF-WAY	120 ft.	98 ft.	80 to 90 ft.	DOWNTOWN CITY STREET	72 ft.	60 to 70 ft.	NA	83 to 85 ft.	80 ft.	
EXCAVATION	DEPTH	78 ft.	70 ft.	65 ft.	NA	80 to 90 ft.	106 ft.	50 to 60 ft.	55 ft.	35 ft.
	WIDTH	90 ft.	55 ft.	72 ft.	NA	66 ft.	55 ft.	NA	70 to 75 ft.	NA
SLURRY WALL SPECIFICS										
TYPE	STEEL BEAM & TREMIE CONC.	STEEL BEAM & TREMIE CONC.	STEEL BEAM & TREMIE CONC.	STEEL BEAM & TREMIE CONC.	STEEL BEAM & TREMIE CONC.	STEEL BEAM & TREMIE CONC.	REINFORCED CONCRETE	NA	NA	
ROLE	PERMANENT STRUCTURE	PERMANENT STRUCTURE	TEMPORARY EARTH SUPPORT STRUCTURE	TEMPORARY BLDG. SUPPORT STRUCTURES	TEMPORARY BLDG. SUPPORT STRUCTURES	TEMPORARY BLDG. SUPPORT STRUCTURES	PERMANENT STRUCTURE	PERMANENT COMPOSITE STRUCTURE	PERMANENT STRUCTURE	
DIMENSIONS	DEPTH	90 to 110 ft.	94 to 106 ft.	80 ft.	VARIES W/WALLS	80 to 90 ft.	108 ft.	70 to 80 ft.	NA	
	THICKNESS	24 to 36 in.	33 to 44 in.	30 in.	24 to 30 in.	30 in.	NA	24 in.	NA	
	LENGTH	740 ft.	1160 ft.	1062 ft.	VARIES W/WALLS	140 ft.-2walls	NA	530 ft.	7000 ft.	
BRACING	CROSS LOT	CROSS LOT	CROSS LOT	CROSS LOT	NA	TIEBACKS	NA	NA	NA	
PERFORMANCE	MAX. LATERAL MOVEMENT	1.25 in.	1.1 to 1.7 in.	0.12 in.	UNDER CONSTRUCTION	DESIGN PHASE	DESIGN PHASE	UNDER CONSTRUCTION	DESIGN PHASE	DESIGN PHASE
	MAX. HORIZ. SETTLEMENT	0.75 in.	VARIED W/THK. SOFT CLAY LAYER	NA						
SPECIAL COMMENTS										
(SUMMARY OF REASONS FOR CHOICE OF SLURRY WALL OVER CONVENTIONAL TECHNIQUES)	UNDESIRABLE TO LOWER GROUND-WATER TABLE	UNDESIRABLE TO LOWER GROUND-WATER TABLE UNCONSOLIDATED CLAY BUILDING SENSITIVITY	SLURRY WALLS SAVED \$2,300,000 IN UNDERPINNING COSTS	UNDERPINNING IMPOSSIBLE SLURRY WALLS PROVIDE BLDG. SUPPORT	SLURRY WALLS USED TO SUPPORT 2 LG. BLDGS. EARTH CONTROL CRITICAL	SLURRY WALLS USED TO SUPPORT 3 LG. BLDGS. EARTH CONTROL CRITICAL	HISTORIC BLDGS. CAN'T UNDERPIN GROUNDWATER CONTROL ENVIRONMENT CONCERNS	AVOID DEWATER MIMIMIZE EARTH SETTLEMENT	CONTINUITY OF CONSTRUCTION OVER 1.5 MILES GROUNDWATER CONTROL COST SAVING \$150 PER FOOT OVER OTHER METHODS	

PART II: SLURRY WALL: AN INTEGRAL

PART OF PERMANENT UNDERGROUND TRANSPORTATION STRUCTURES

CHAPTER 6

FEASIBILITY STUDIES

6.1 Advantages

The slurry wall technique lends itself extremely well to transportation-related structures, as evidenced by its utilization in highway overpasses, underground parking garages and subway tunnels. Its advantages over conventional construction are particularly evident when used in congested areas where land values are exorbitantly high, and functional space is at a premium. For such sites, rigid slurry walls often provide a viable construction alternative to flexible systems of steel sheet piling or soldier piling and lagging variety.

The slurry wall provides rigid support with minimum disturbance to adjacent structures, and allows construction to take place in very close proximity to the existing foundations of adjacent buildings. Slurry walls also minimize the horizontal ground movement and the settlement of adjacent structures during excavation work, since they form a waterproof barrier between the construction site and adjacent ground.

The slurry wall can be constructed in practically any soil conditions including sandy granular soil, clayey soil or soil comprised of boulders and rock. The construction process of slurry wall panels allows their width, depth, and length to be varied to meet specific site design parameters. Under favorable conditions, slurry wall can provide considerable savings in time and money by minimizing the need for dewatering and often eliminating the need for underpinning structures adjacent to the excavation site.

Other advantages of slurry walls as part of permanent structures, in both transportation and non-transportation related projects, are discussed in Chapter 2 of this report. Furthermore, the advantages and considerations involved in the use of slurry walls are summarized in Table 3 of the same chapter.

6.2 Limitations

Despite numerous advantages offered by the slurry wall, it is not a panacea. Initial difficulties have been encountered in both design and construction. These difficulties account for the fact that slurry wall use in the United States has, in the past, been limited to temporary structures.

One obvious limiting factor is construction tolerance. The building of a slurry wall entails the construction of a deep, below-grade structure through a slurry mud mixture. This construction is carried out from the ground surface level without having the benefits of visual inspection and direct control. Under these conditions, the specified tolerances, in both verticality and protrusions, normally exceed those tolerances found in conventional design.

If slurry walls are to become part of a permanent structure, there will be a need to provide a structural connection between the walls and other members

such as floors, slabs and roof. Experience shows that it is often difficult to execute these types of connections. For example, the location of joint blockouts in the slurry wall during construction is specified with considerable tolerance.

A second difficulty involves insuring the continuity of the reinforcing steel in the joints. For this, the bars must either be lapped or spliced using mechanical connectors. Usually the rebars are large (No. 11 or larger); consequently, the required overlap length into roof and floor slabs is quite long. It is impractical to allow a long bar to protrude from a slurry wall during its construction. A large size bar requires a large bending radius which cannot be easily accommodated by the wall thickness. This, of course, aggravates the problem.

The third difficulty arises from the need for a conservative assumption of the soil pressures on the frame structure which are formed by the slurry walls and horizontal slabs. It is common to assume asymmetrical lateral loads on the frame (as explained in the following chapter). This asymmetrical loading can cause excessive sidesway of the frame and can produce a large bending moment, which in turn, would require such a thick wall that it may become impractical to build.

There are also several difficulties in using tiebacks and ground anchors. Generally, easements must be obtained from adjacent property owners before the placement of tiebacks is allowed and owners are often reluctant to allow such installation. Also, existing foundations or utilities frequently obstruct the direct installation of these anchoring devices.

Innovative ideas in slurry wall design have emerged in efforts to overcome the limitations encountered when such walls are contemplated as part of a permanent structure. These ideas are presented in Chapter 7.

CHAPTER 7

INNOVATIVE DESIGN IDEAS

In an effort to alleviate the design problems associated with the use of slurry walls as part of a permanent station structure, the following design concepts were developed during the course of the present investigation. In addition, the writers focused attention on current construction practices and materials that make slurry walls feasible as permanent structures. Our Board of Consultants scrutinized and offered significant improvements in the further development of these design concepts.

7.1 Section Geometry

Through a literature survey and site visits, existing and planned subway structures were studied. It was concluded that a minor variation in the geometrical configuration can result in a pronounced difference in the behavior of such structures. The following cross-sections were investigated in efforts to study the effect of varying geometry on moments at the intersection of station slabs and slurry walls.

a. Arched Roof

The arch is a well-known structural element which transmits the load mainly through axial thrust rather than bending. With strong end supports, an arch can carry all dead loads by axial thrust. Under live loads, and in conjunction with the wall and floor, a certain amount of moment is unavoidable, especially near the ends of the arch. However, care should be exercised to provide enough axial thrust in the arch so that the steel requirement can be reduced sufficiently to facilitate the placing of rebars in the slurry wall. Because of the complex form work required, an arch structure is expensive to construct. Using precast sections to avoid the need for form work is not a solution since transportation and installation costs are high. If the curvature of the arch is large, the reinforcement may have to be bent to fit the surface contour resulting in an additional expense.

b. Folded Plate Roof

To circumvent the difficulties of an arch roof, the arch configuration was modified into a folded-plate roof structure (see Figure 54). Conceptually, a folded plate can have as many folds as desired. For simplicity and economy, however, a two-fold configuration was considered to be optimal. This configuration shows a close resemblance to an arch, but is much simpler to construct. Through judicious selection of frame dimensions, the bending moment and bearing pressure at the joints can be kept within manageable limits. The folded plate also develops a certain amount of axial thrust near the slab/wall joint, and so reduces the quantity of rebars that must be extended into the wall.

c. Intermediate Supports

Another alternative in geometrical configuration would be to provide intermediate supports to the roof (Figure 55). Intermediate supports within the station can greatly reduce the moments since the span between slurry walls is

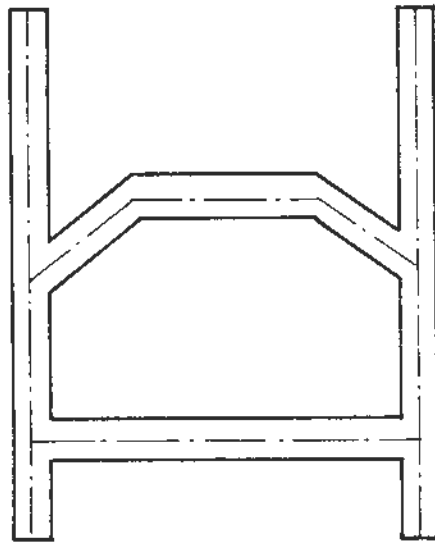


Figure 54. Folded Plate
Roof Section

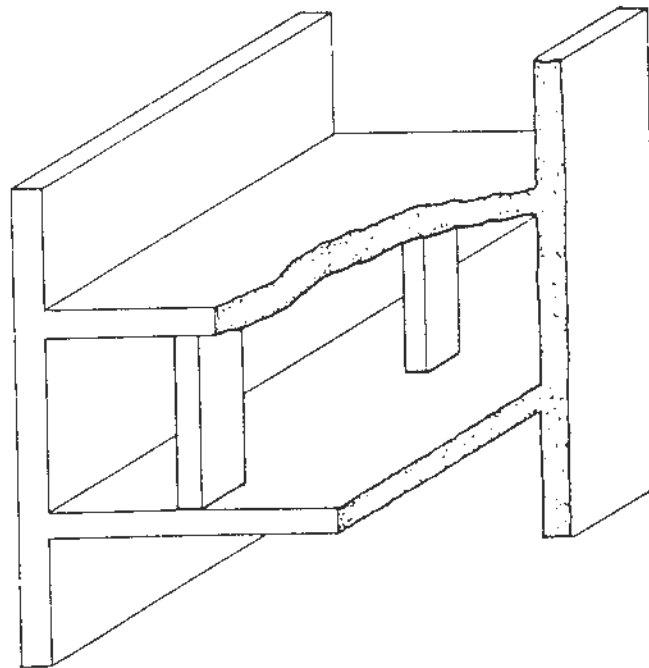


Figure 55. Perspective View Of A Box Section
with Intermediate Column Supports

subdivided. To reduce bending moments, columns spaced appropriately in the middle of the station would be almost as effective as a continuous wall. Such columns can also serve as supports for form work during roof slab construction.

7.2 Moment Releasing Joints

Still another means to reduce large moments between the walls and the slabs is to provide "hinged" joints at the points where the walls meet the horizontal members of the frame as shown in Figure 56.

A hinged joint which transmits only the shearing force into the slurry wall can be designed without difficulties. On the other hand, the bending moment at the midspan of the horizontal members with hinged ends could increase by as much as 50 percent. Accordingly, the thickness of the slabs as well as the reinforcement must also be increased.

Another serious drawback of sections having hinged joints is the considerable decrease in the rigidity of the frame thereby causing excessive sidesway. For this reason, it is not advisable to place hinges at all four joints. In our study, hinges were placed at the ends of the floorslab only while the roof remained rigidly connected. (Figure 56b).

One variation of the hinged joint concept entails a temporary rather than a permanent hinge. Temporary hinges enjoy some of the advantages of both permanent hinges and rigid joints in that the joints can be designed for flexibility during early-stage loadings, but can become rigid for long-term loadings. The advantage is clearly important for the multi-stage loading conditions that a permanent structure employing slurry walls must withstand. For example, the joints between the floor and the walls can be left as non-moment-transmitting hinges, while the dead load and uplift hydrostatic pressure are applied to the structure during the construction stage so that these loads do not cause any additional moment to the slurry wall. After two corresponding short-term loading deformations have taken place, and before the entire structure is exposed to the long-term (at rest) pressures from adjacent soils, the joints are made rigid by cement grouting or other suitable methods. The additional stiffness furnished by the "rigidified" joints is necessary to preserve the structural stability of the frame and to hold sidesway to a minimum in the event of unbalanced lateral earth pressures. Admittedly, these rigidified joints do transmit some moment to the wall. However, joints built to be rigid from the beginning transmit more moment. In other words, it is possible to "leak" as much moment out in its "hinged" state as in a controlled manner. The extent and timing of rigidifying the joints is within the control of the designer.

7.3 Frame Stiffness: Counterforts

One major problem in the section frame design is the need for high flexural rigidity to resist the asymmetrical loading conditions, especially if hinged joints are used. To obviate this difficulty, the writers developed the concept of putting counterfort ribs (see Figure 57) on the walls. Spacing between the counterforts can be varied so that full advantage is taken of the maximum efficiency of "T-beam" action. The rib dimensions are selected to furnish the needed amount

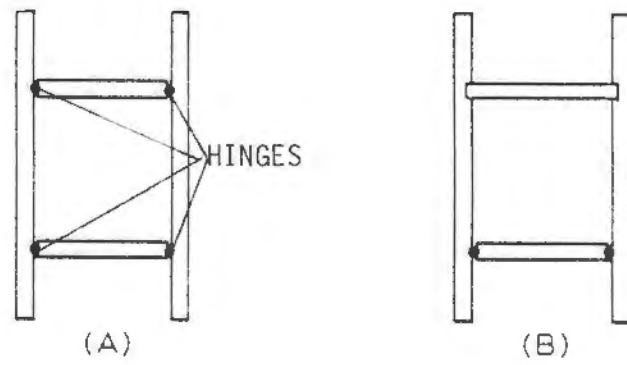


Figure 56. Basic Structures With Hinged Joints

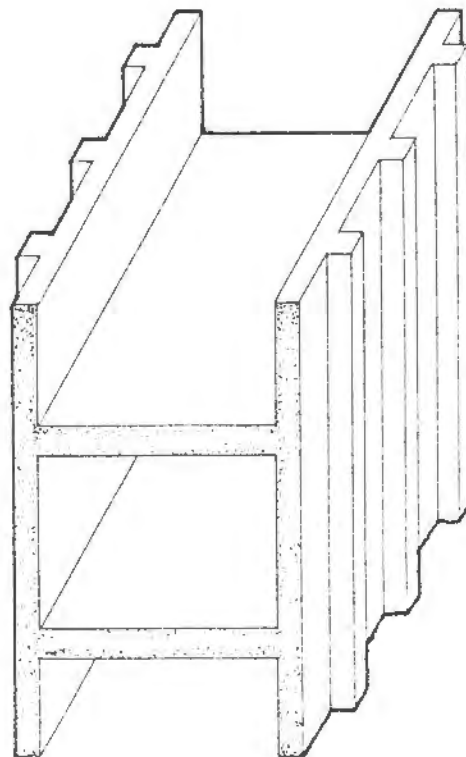


Figure 57. Isometric Projection Of Slurry Walls With Counterforts

of flexural rigidity as well as to keep the amount of reinforcing steel in the walls within allowable limits. There is little or no difficulty in excavating a T-shaped trench and placing rebar cages by means of currently available equipment and techniques (see Figures 7b and 58).

A counterfort should be placed on the tension side of the structure. Since moment reversals normally occur throughout the wall, counterforts should ideally be built on alternating sides of the structure. Practical considerations prohibit this arrangement, thus leaving the choice between either the "station side" (the interior side) or the soil side (exterior side) of the slurry wall. Generally, when moments near the wall/slab joints govern the design, the counterfort should be placed on the exterior side.

Counterforts can also be used in roof and floor slabs in much the same manner as used in the walls. If hinged joints are used in the design, the counterforts should be placed on the underside of the slab to help support any large moment at midspan (see Figure 59). The counterforts can easily be adapted to any section configuration including the folded plate roof design as shown in Figure 60.

7.4 Prestressed Slurry Wall

Prestressed slurry wall is yet another novel concept, tested and employed successfully in Europe. The behavior of such a wall is governed by induced prestressing. The basic concept of post tensioning is illustrated in Figure 61 and a typical example is shown in Figure 62.

The use of prestressing in slurry walls greatly increases the maximum allowable unbraced height, and therefore, has important economic advantages. In order to maximize the flexural strength of the wall by this method, tendons should be placed as eccentrically as possible within the wall. This arrangement will produce a moment resisting field that almost counterbalances the one produced by the external moments. Because of these advantages, a prestressed slurry wall will normally require less bracing than an ordinary slurry wall.

Construction Practices

The use of slurry walls in other countries has offered designers and contractors the opportunity to directly address many of the difficult problems associated with the construction of the walls as permanent structures. During the course of this contract, the writers investigated the various measures used by contractors to provide solutions to problems such as the continuation of reinforcing steel between walls and slabs, as well as measures for wall support. At this point it is advantageous to review some concepts and current practices being employed in slurry wall construction.

7.5 Joint Fabrication

A. Tolerance Limits

The tolerance specification for a slurry wall is an important consideration. It is inevitable that unreasonably close tolerance requirements for keys and blockouts, which are costly to meet, would be reflected in higher bid prices.



Figure 58. T Shaped Reinforcing Cage

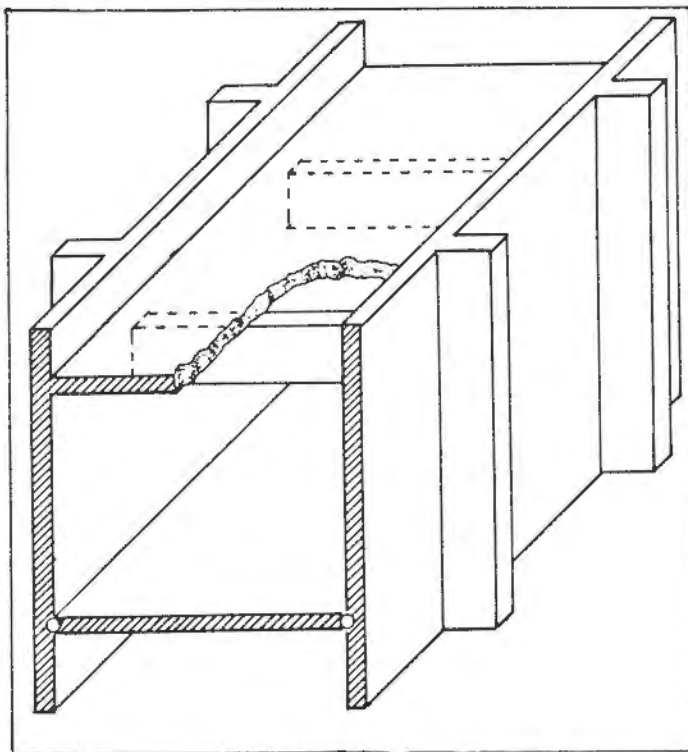


Figure 59. Counterforts Used On The Slurry Walls And As Beams On Ceiling

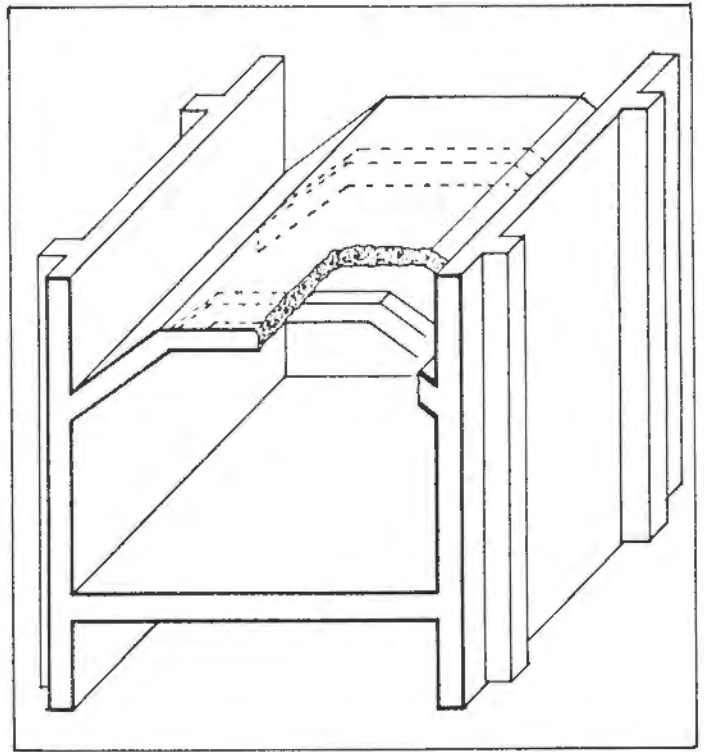
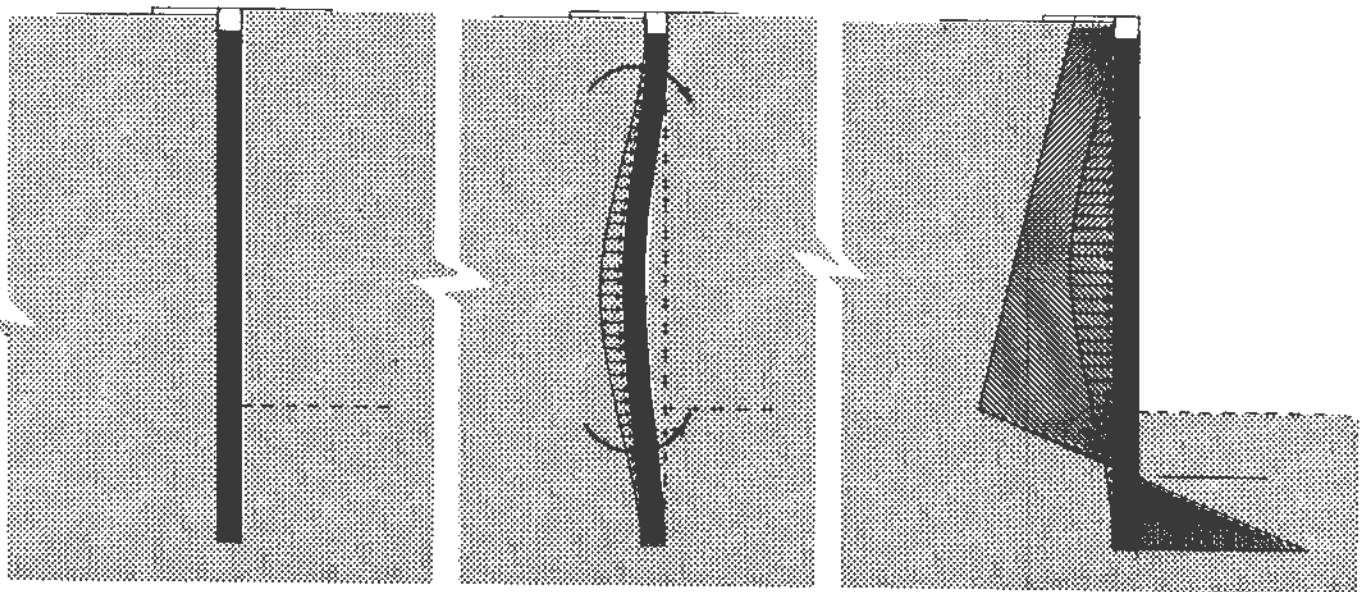


Figure 60. Counterforts As Applied To The Folded Plate Roof

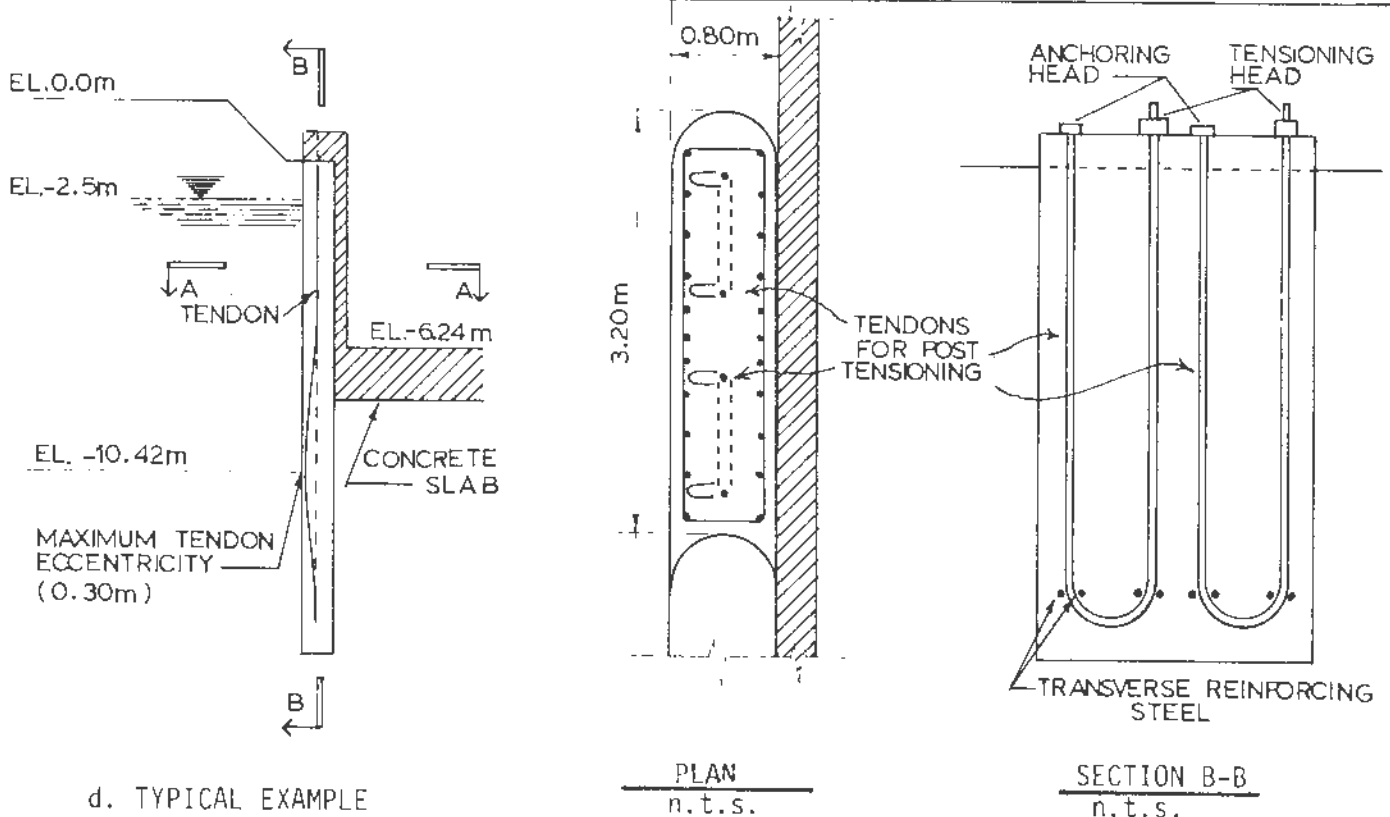


a. BEFORE TENSIONING

b. AFTER TENSIONING BEFORE EXCAVATION

c. AFTER EXCAVATION

Figure 61 Basic Concept Of Prestressed Walls



d. TYPICAL EXAMPLE

PLAN
n.t.s.

SECTION B-B
n.t.s.

NOTE: SECTIONS SHOW ONE TYPICAL PANEL OF THE SLURRY WALL

Figure 62. Typical Example Of Post Tensioned Slurry Walls

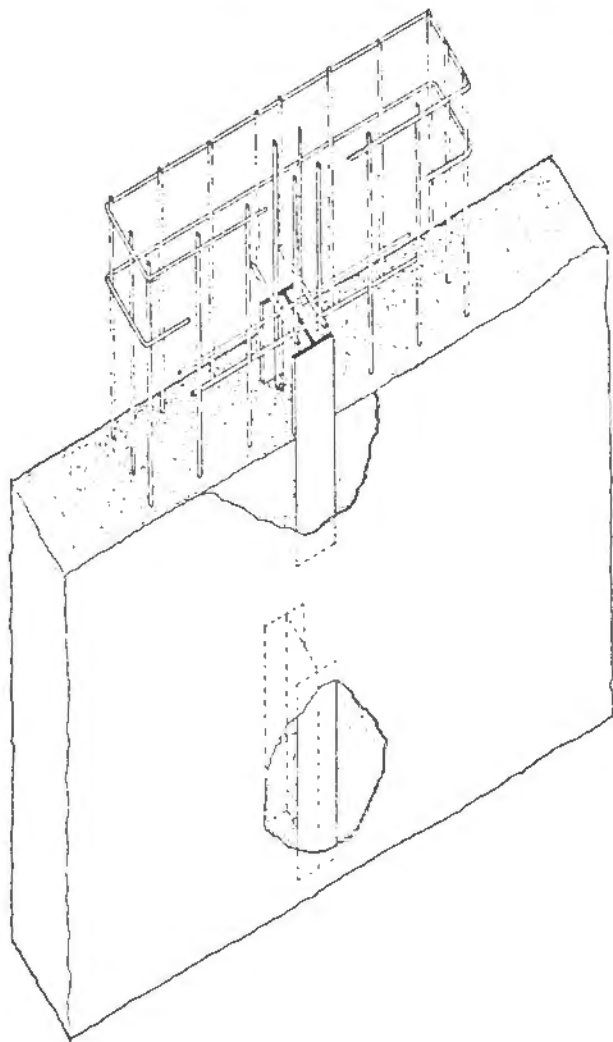
The space for a slab/wall joint can be provided by an expanded polystyrene blockout in the wall. After excavating the wall, the blockout is removed to expose the wall's reinforcing steel, leaving a shear key on which the slab rests. Using this method, a misalignment of the joint area of as much as ± 6 in. (15.2 cm) vertically and ± 3 in. (7.6 cm) horizontally may be expected and must be allowed for in the design.

A closer tolerance can sometimes be achieved by chipping the concrete away near the location where a joint is intended. However, this method is not suitable if continuity of reinforcing steel between the slab and wall is required. A wall, reinforced by rolled structural steel shapes, may be better suited for this approach. Through the use of good quality welding, the rolled shapes can be easily connected to the rebars in the slabs. This is a noted advantage of the steel beam and concrete type slurry wall.

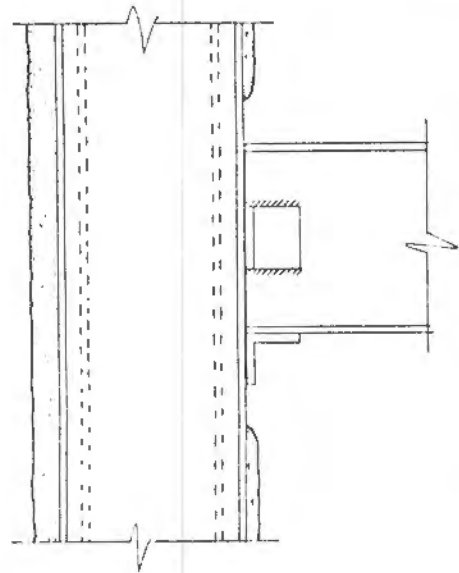
As a variation of the steel beam and concrete wall, a method is proposed in this report that is more practical for slurry walls as a part of permanent structure. This method entails the embedment of short segments of structural steel sections in the slab and the slurry wall to form slab/wall joints. These segments should be embedded at levels where joints are desired, as shown in Figure 63. The steel section is rigidly clamped into the reinforcing steel cage so that the concrete can be tremied around it and the structural shape embedded in the roof or floor slab can be welded directly to it. In this way, the continuity of reinforcement is assured and difficulties associated with strict tolerance requirements for locating slab/wall joints are no longer a problem.

B. Slab to Wall Joint Fabrication

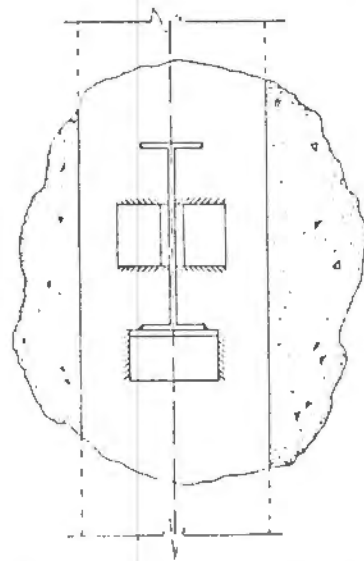
Several specially designed mechanical devices are currently employed to provide the continuity of rebars in a slurry wall. One method uses a mechanical coupler with lock nuts to splice the rebars, as shown in Figure 64. Another device used in splicing the rebars is called "Rebar Lapp Splicer" (Figure 65). As the name implies, it employs bolted clamps torqued to 50 ft-lb (6.93 m-kg) and is guaranteed to develop a working load up to 27,000 lbs (12,300 kg). Another ingenious process entails a field welding procedure. This employs a specially prepared metal alloy known as "Cadwell", heated in a crucible, and applied through a graphite pouring basin to the butted rebars being spliced. The Cadwell alloy is essential to transmit the tensile stress. Figure 66 shows sketches and supporting technical information pertaining to this device. The process has found wide use in the construction of nuclear power facilities, as well as slurry wall construction. One of the major advantages of this process is that it requires only a minimum exposed length of reinforcing steel coming out from the slurry wall to effect the splice with slab steel (see Figure 65). For example, to splice a No. 11 bar, it must extrude 6-1/4 in from the wall surface. A blockout in the slurry wall should provide enough space to allow for working room and construction-inherent variance in the location of the reinforcing steel.



a. ISOMETRIC VIEW OF SLURRY WALL PANEL SHOWING HOW STRUCTURAL STEEL SHAPES ARE EMBEDDED IN THE WALL



b. SIDE VIEW OF A JOINT

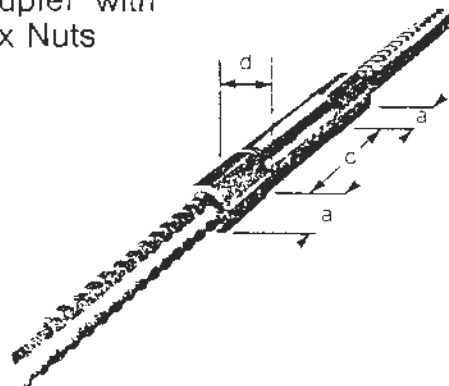


c. FRONT VIEW OF A JOINT SHOWING CONCRETE CHIPPED AWAY TO LOCATE BEAM IN THE WALL

Figure 63. Alternate Method To Steel Beam And Concrete Type Of Slurry Wall

DYWIDAG
Threadbar Reinforcing Systems *

Coupler with
Hex Nuts



Coupler with
Lock Nuts

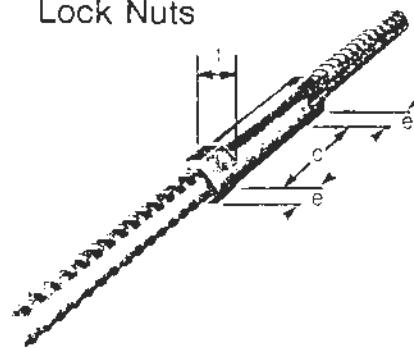


Figure 64. Mechanical Coupler With Lock Nuts

(Information Supplied By Dyckerhoff & Widmann, Inc.)*

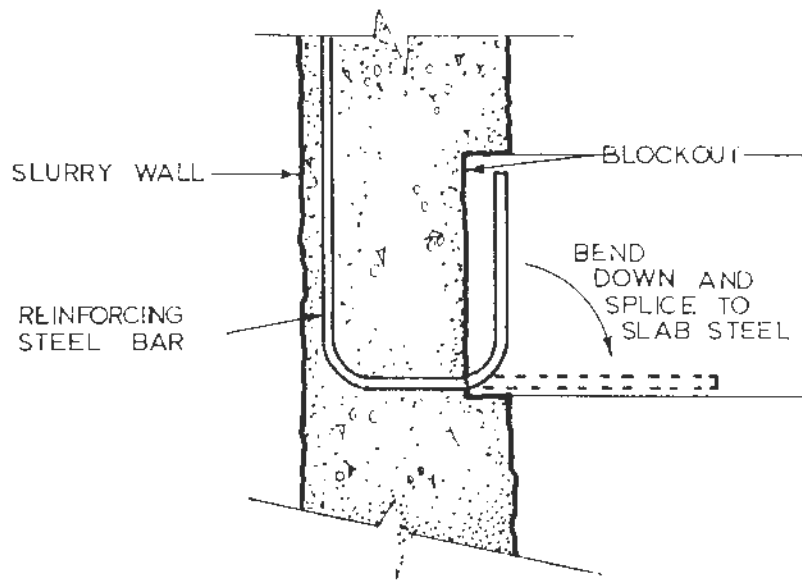


Figure 65. Rebar Position During Wall Fabrication And After Excavation

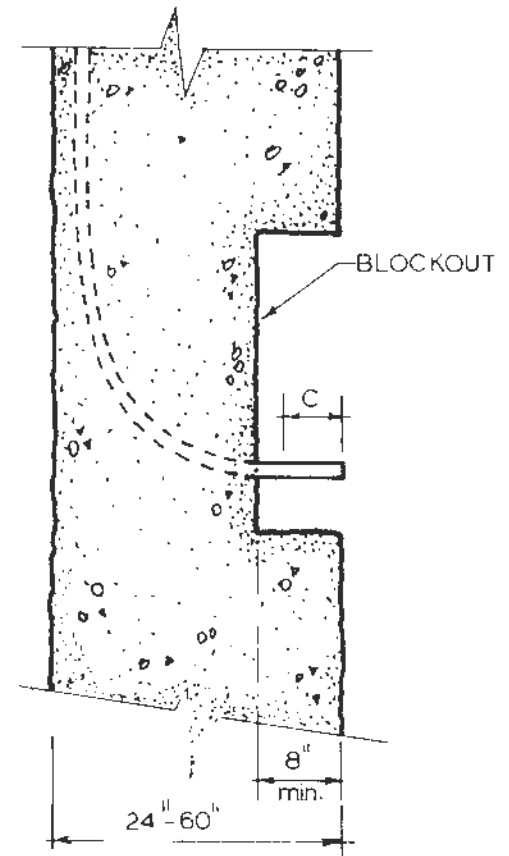


Figure 66. Rebar Position In Wall Fabrication (Alloy Welding System)

The use of sections of structural steel embedded in the slurry wall and slabs to fabricate a rigid joint, as shown in Figure 63, can also provide the means for fabricating the temporary hinge. Figure 67 depicts graphically the sequence of installing the temporary hinge. A joint weld is applied only to one flange of the structural shape which is embedded in the slab during construction. An access hole is left in the concrete slab around the joint so that further welding can be done after the structure is loaded. Minor deformation of the structure is allowed to occur as bracing is removed from the walls and the slab starts to carry the load and the access hole in the slab can then be grouted. This process provides a permanent rigid joint for long-term loading conditions on the final structure.

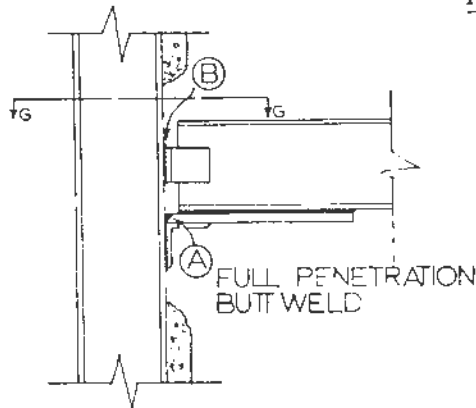
7.6 Bracing and/or Tiebacks

To resist the soil pressure between the start of the excavation and final backfilling, it is standard procedure to use bracing and/or tiebacks which serve as temporary, slurry wall supports. The spacing of braces and/or tiebacks, as well as the sequence of placing and removing them and often their prestress loads, are determined by engineering design and analysis.

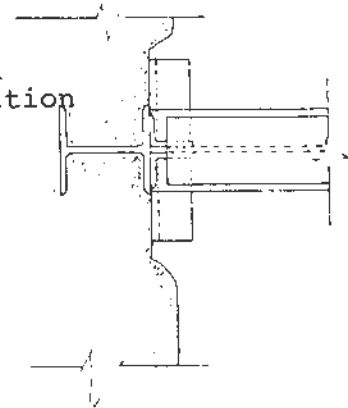
At approximately 4 to 6 ft below ground level, a top brace is usually installed to minimize wall movement. This should be done as soon as possible after the excavation process begins. Additional levels of bracing, at predetermined spacing, are installed as excavation progresses. After construction of the roof and floor, appropriate bracing levels will be removed according to a specific schedule. Premature removal of bracing can cause unduly high stresses in the wall.

Tiebacks serve a function similar to internal bracing, except that tiebacks can be either temporary or permanent, while internal bracing is usually temporary, only occasionally being left in place during backfilling operations. Tiebacks, which anchor the wall by embedment into the soil mass or rock behind the wall, have the obvious advantage of leaving a large, unobstructed working space between the slurry walls on opposite sides of the trench. In contrast, it is difficult to work around bracing during excavation and backfilling operations.

A striking example of the extensive use of tiebacks at a difficult construction site is shown in Figure 68. The use of tiebacks in slurry wall construction in the U.S. has been very limited. This limited use is attributed to the difficulty of obtaining easement for the use of permanent tiebacks from the owner(s) of adjacent properties.

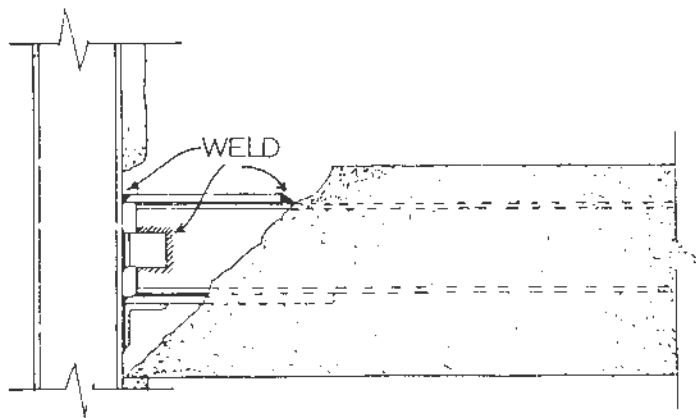
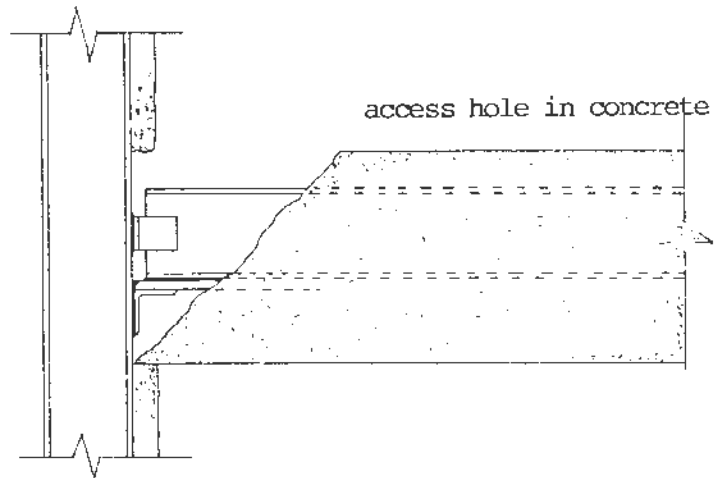


- A. Create Temporary Hinge by welding slab steel with wall steel only at position (A) to the wall beam. Angles on both sides of the slab steel (B) restrain lateral movement.



SECTION-GG

- B. Concrete is poured for the bottom slab but a space is left open for access to the hinge. The structure is allowed to deform under temporary loading conditions as bracing is removed.

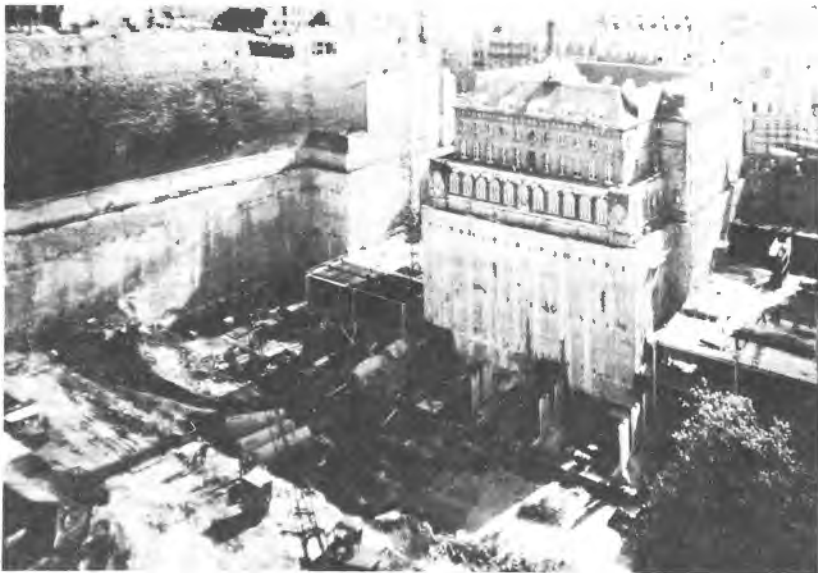


- C. After minor deformations have occurred butt welds are completed on slab web and flange to connect them to the slurry wall embedded steel. Hinge is thus locked and concrete is backfilled with grout.

Figure 67. Possible Design For A Temporary Hinge



a. TYPICAL TIEBACK INSTALLATION
(PHOTO COURTESY OF ICOS)



b. DEMONSTRATION OF A BUILDING SUPPORTED BY TIEBACK ANCHORED
SLURRY WALLS (PHOTO COURTESY OF SOLETANCHE)
(Paris, France)

Figure 68. Examples of the Extensive Use of Tiebacks

TABLE 7

Results Of Parametric Study No. 1: Varying Roof Geometry And Thickness

Results: **

		Simple Box Section		Folded Π Roof Section	
		Moment at A ft-kip	Moment at B ft-kip	Moment at A ft-kip	Moment at B ft-kip
Symmetric Loading Full (at rest) earth pressure on both sides of structure	6' Thick Roof $I=5.333\text{ft}^4$ $I_1=18\text{ft}^4$	-847	-654	-499	-612
	7' Thick Roof $I=5.333\text{ft}^4$ $I_1=28.583\text{ft}^4$	-721	-671	-424	-629
	8' Thick Roof $I=5.333\text{ft}^4$ $I_1=42.667\text{ft}^4$	-626	-685	-351	-643
Asymmetric Loading* Full (at rest) earth pressure one side - $\frac{1}{2}$ (at rest) earth pressure other side		Moment at B	Moment at C ft-kip	Moment at B	Moment at C ft-kip
	6' Thick Roof $I=5.333\text{ft}^4$ $I_1=18\text{ft}^4$	-835	-921	-750	-667

Note: Only maximum moments at wall-slab intersections shown
(1 ft-lbs=1.36 Nm)

*For this analysis $\frac{1}{2}$ (at rest) earth pressure was applied to the right side of the structure replacing the full(at rest) earth pressure shown above on both structures for the asymmetrical loading condition.

** Results obtained by using finite element analysis method

8.2 Parametric Study No. 2: Arched Roof vs. Folded Plate Roof

PURPOSE: To compare the above mentioned two basic cross-sections subjected to the asymmetrical loading.

CASES

STUDIED: For this study a similar loading condition was assumed which included the soil-structure interaction. See the soil pressure beneath the structure in Figure 70 which also shows the cross sections studied and the loading condition assumed.

RESULTS: Summarized in Table 8.

CONCLUSION: In terms of the state of stress, for the loading condition shown, the arched roof section did not indicate any significant advantage over the folded plate roof. In addition, the arched roof section was considered more difficult to construct than the folded plate roof.

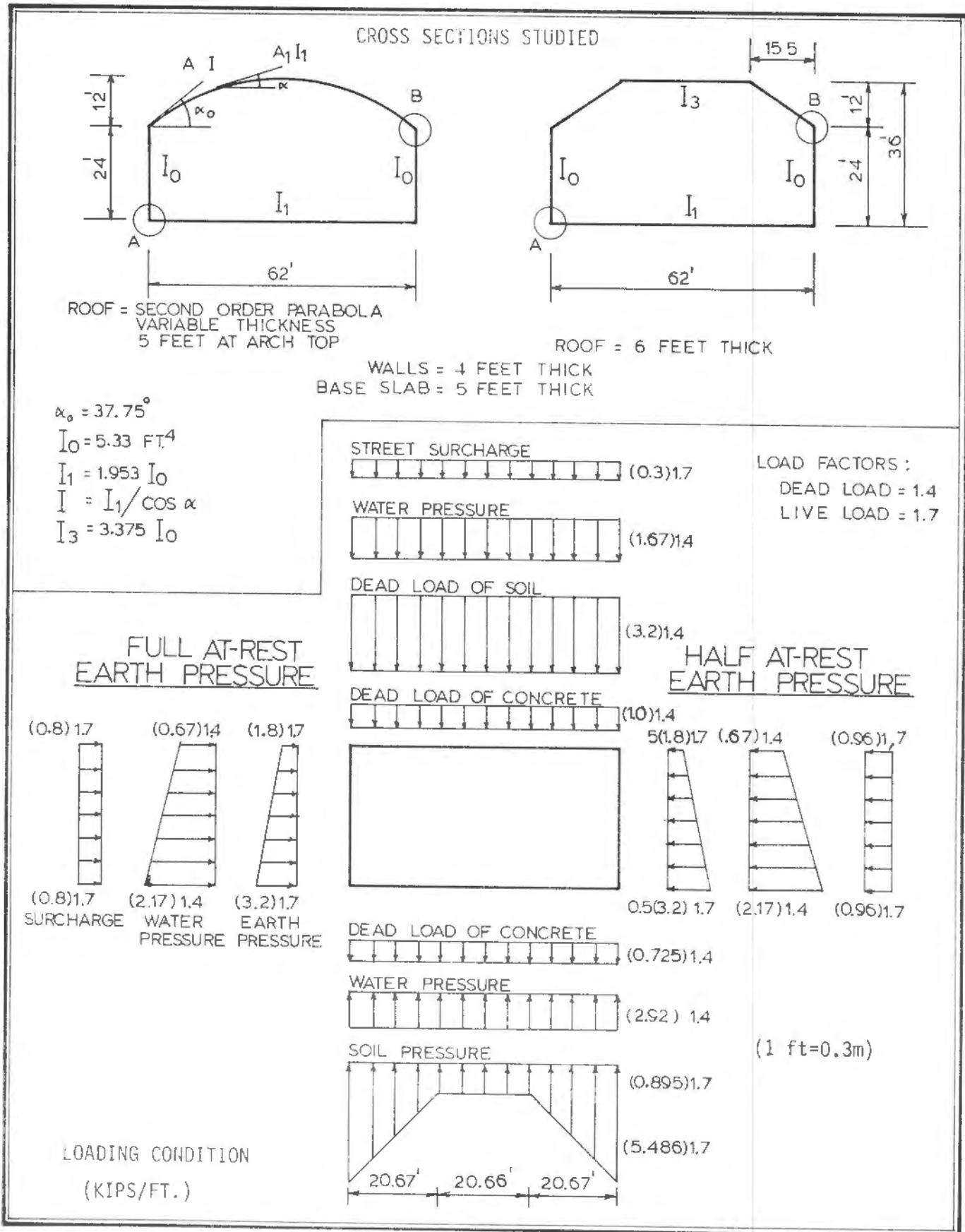


Figure 70. Cases Studied And Loading Condition For Parametric Study No. 2

Table 8. Results Of Parametric Study No. 2: Arched Roof Vs. Folded Plate Roof

		Arched Roof Section		Folded Plate Roof Section	
		A	B	A	B
Moment (ft kip)		-1471.34	-1652.22	-1468.7	-1386.89
Shear (kip)	Wall	114.59	100.67	126.8	91.95
	Slab	-277.06	-172.52	-256.08	-187.92
Axial Force (kip)	Wall	-255.22	-285.30	-262.07	-279.07
	Slab	—	-240.89	—	-242.52

(1 ft-lbs=1.36 Nm)

8.3 Parametric Study No. 3: Effect of Axial Thrust on Rebar Quantity

PURPOSE: To study various design alternatives to reduce the amount of steel required at the wall-base slab intersection.

CASES STUDIED: Figure 71 presents the loading conditions and the section geometries.

Note: The loading condition assumed for this study simulated a water table below the excavation level, at rest soil pressure, and a saturated condition in the soil

RESULTS: Summarized in Table 9.

CONCLUSION: Changing the geometry of the base slab reduces the moment at the wall-slab joint and increases the axial force in the slab.

Greater slurry wall thickness increases the moment at the wall-slab joint.

Variable base slab thickness (section C) results in a reduction in the amount of reinforcing steel required at the wall-slab joint.

An axial thrust introduced into the base slab creates a combined compression and bending situation. A more favorable reinforcement distribution is possible because the axial compression neutralizes some of the tensile stresses on the concrete section due to flexure.

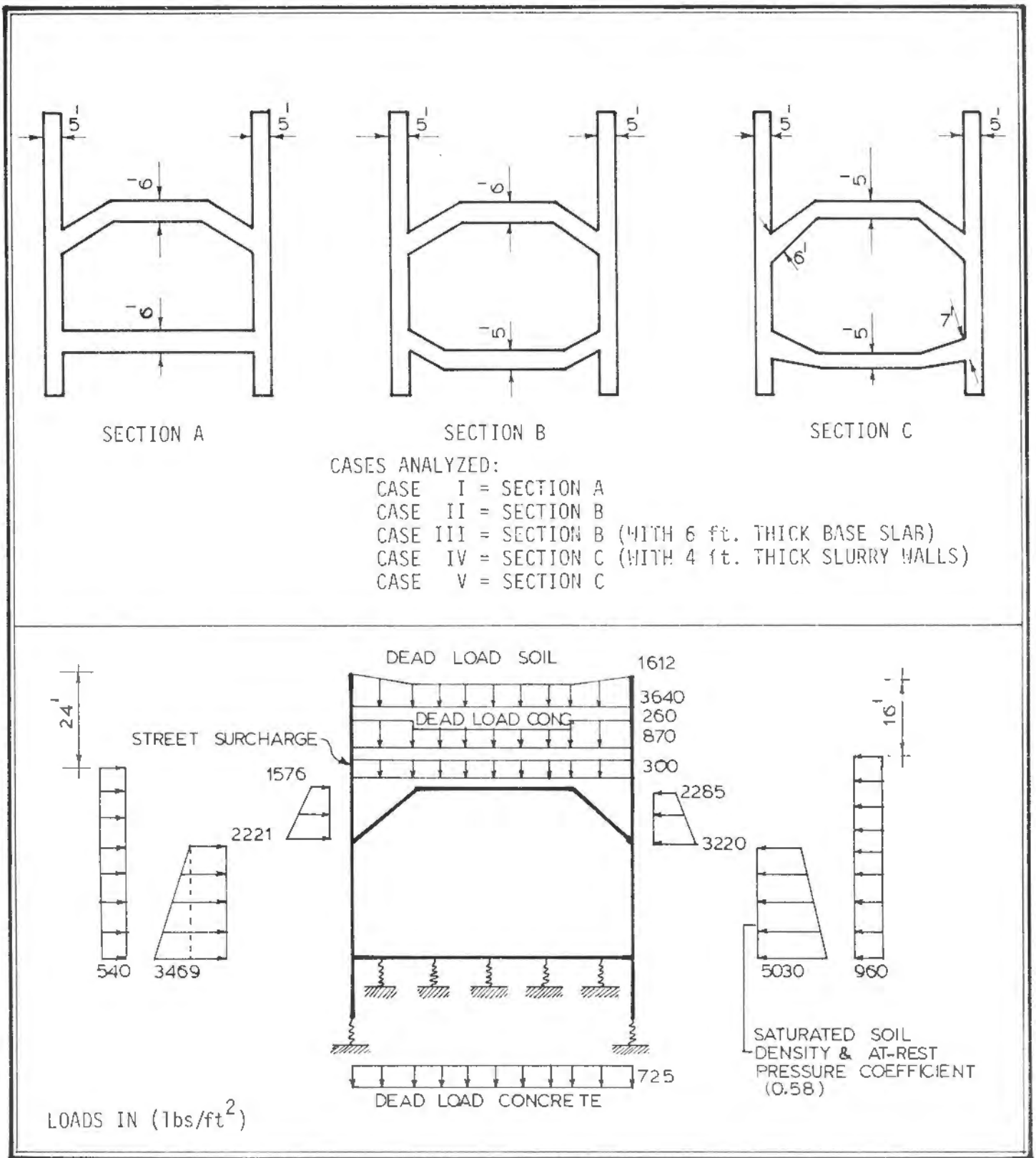


Figure 71. Cases Analyzed And Loading Conditions For Parametric Study No. 3

TABLE 9

Results Of Parametric Study No. 3: Effect Of Axial Thrust On Rebar Quantity

RESULT:

	CASE I	CASE II	CASE III	CASE IV	CASE V
Moment A (ft-kip)	-1584.96	-1207.41	-1250.92	-1120.22	-1302.51
Shear A (kip)	-174.21	-166.96	-171.29	-164.51	-166.80
Axial Force A (kip)	32.25	141.67	132.05	143.03	133.21

a. Statistics on base slab under asymmetrical loading conditions.

	CASE I		CASE III		CASE IV	
	A	B	A	B	A	B
Tension steel (in ²)	10.04	2.53	4.68	2.53	3.18	2.53
Compression steel (in ²)	1.56	2.53	2.56	2.53	1.00	2.53

b. Area of reinforcing steel required for continuation from slab to wall to properly handle moments.

8.4 Parametric Study No. 4: Effect of Counterforts on Reducing Sidesway

PURPOSE: To investigate the effectiveness of counterforts in reducing sidesway deflection on structure cross sections having hinged joints at the wall-base slab intersection.

CASES STUDIED: Figure 72 shows the section geometries studied.

Figure 73 shows the loading conditions assumed.

Figure 74 shows the details of the counterforts used on the roof slab and the slurry walls.

RESULTS: Summarized in Table 10.

CONCLUSION: Under asymmetrical loading conditions, counterforts reduce the deflection of the structure.

Equally important is the fact that the use of counterforts, in conjunction with hinge connections between the wall and base slab, allows a reduction of the sidesway deflection caused by asymmetrical loading. This sidesway is less than the corresponding deflection found in a structure having rigid joints and no counterforts.

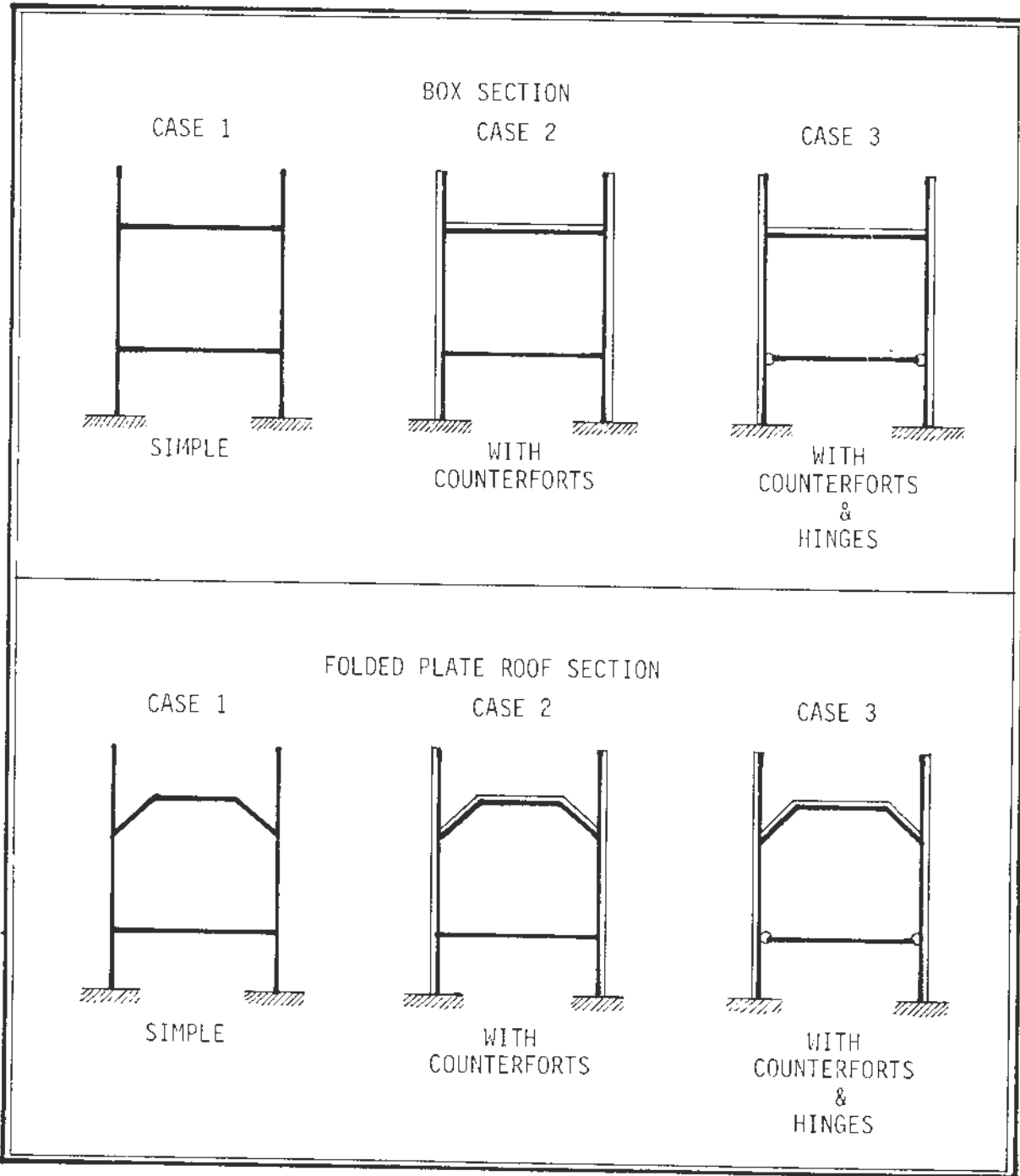
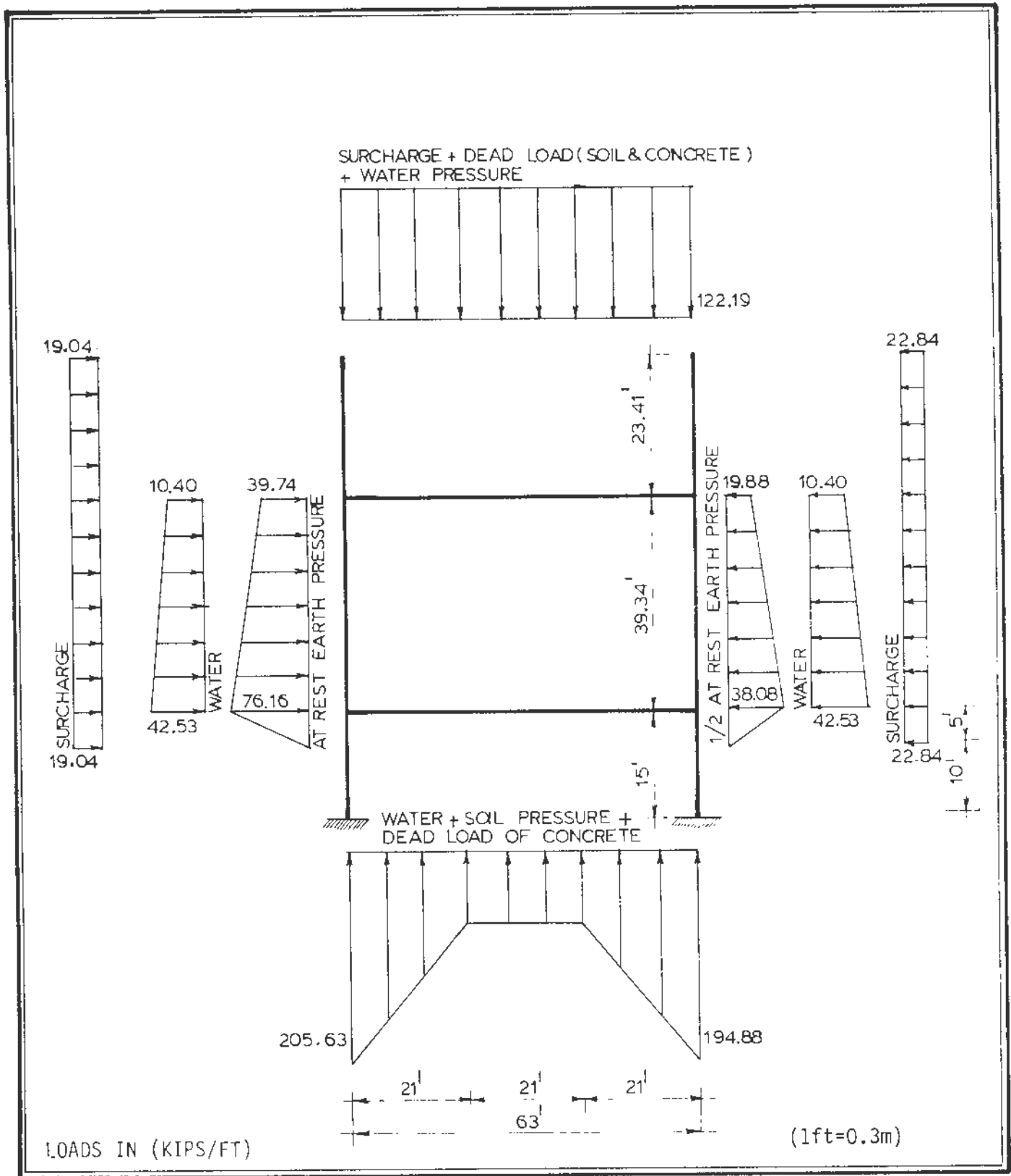
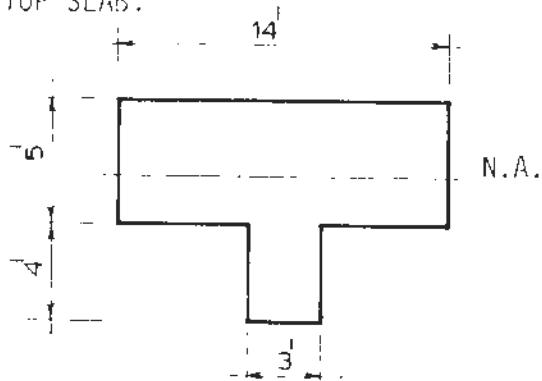


Figure 72. Station Cross Sections Studied For Parametric Study No. 4



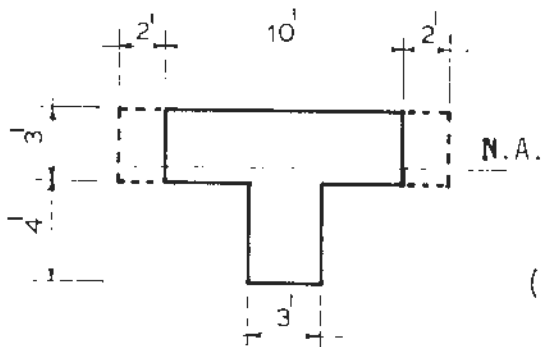
73. Loading Condition For Parametric Study No. 4

TOP SLAB:



$$\begin{aligned} \text{AREA} &= 82 \text{ ft}^2 \\ I &= 369.226 \text{ ft}^4 \\ b_{\text{eff}} &= 14 \text{ ft} \end{aligned}$$

SLURRY WALL:



$$\begin{aligned} \text{AREA} &= 42 \text{ ft}^2 \\ I &= 143.5 \text{ ft}^4 \\ b_{\text{eff}} &= 10 \text{ ft} \end{aligned}$$

(1 ft=0.3m)

Figure 74. Counterfort Information

TABLE 10

Results Of Parametric Study No. 4: Effect Of Counterforts On
Reducing Sidesway Of Structure

	Box Section		Folded Plate Roof Section	
	Moment* (ft kip)	Maximum Lateral Deflection (ft)	Moment* (ft kip)	Maximum Lateral Deflection (ft)
CASE 1	-26,656	0.066	-25,046	0.06
CASE 2	-28,116	0.047	-26,996	0.04
CASE 3	0	0.066	0	0.049

(1 ft=0.3m)

8.5 Summary

The authors have demonstrated several options available for overcoming difficulties associated with the use of slurry walls as permanent structures. Modification in the cross-section can reduce critical moments where the construction process and field requirements preclude more convenient box frame arrangements. In particular, a folded plate roof can reduce moments at the wall/slab intersection by 20 to 40 percent for asymmetrical and symmetrical loading cases respectively (see results of Parametric Study No. 1). In addition, the amount of tension steel reinforcement can be reduced by as much as 68 percent (see results of Parametric Study No. 3).

Various design options, such as intermediate supports or hinged joints and counterforts, can also alleviate design problems. As stated before, hinged joints can eliminate troublesome moments at the slab/wall intersection, while counterforts insure structural stability against sidesway to an extent similar to, or greater than, that found in a plain rigid box frame section (see results of Parametric Study No. 4).

CHAPTER 9

COST ANALYSIS AND COMPARISON

9.1 Introduction

In Chapter 8, the design parameters were limited to the thickness variations and centerline profiles. This approach was effective in the structural design for the practical use of slurry walls as a part of permanent structure. In this chapter, the study is broadened to include architectural and cost parameters so that a cost effectiveness analysis can be made. Care was exercised to compare the different design configurations under reasonable and fair physical and cost conditions.

In what follows, a set of eight different tunnel configurations, as shown in Figure 75, have been selected for design and cost analysis purposes. Four of these configurations have center posts. We shall include, in a separate section, a brief discussion on the effect of center posts with respect to cost analysis and comparison. Furthermore, sections without center posts have been designed and analyzed for two different types of slabs, i.e., slabs having uniform as well as variable thicknesses. This makes a total of 12 cases studied in this chapter.

9.2 Criteria for Comparison

In accordance with the remark made in the previous section, a set of criteria have been established by the writers for the design of various tunnel configurations, along with the corresponding cost analysis and comparison. These criteria are provided below.

- a. Same construction site.
- b. Same internal dimensions of station structures.
- c. Same design criteria.
- d. Same design concepts covered in parametric study.
- e. Same cost basis.

Each of these criteria is explained in detail in the next few pages.

- a. Construction site

Previous slurry wall design and construction projects have demonstrated that different construction sites and soil conditions may result in a significant difference in total construction cost. For a realistic cost comparison, it is, therefore, necessary to select the same construction site for all eight configurations mentioned earlier. With this in mind, a site similar to the WMATA D4b project was selected for the cost analysis and comparison.

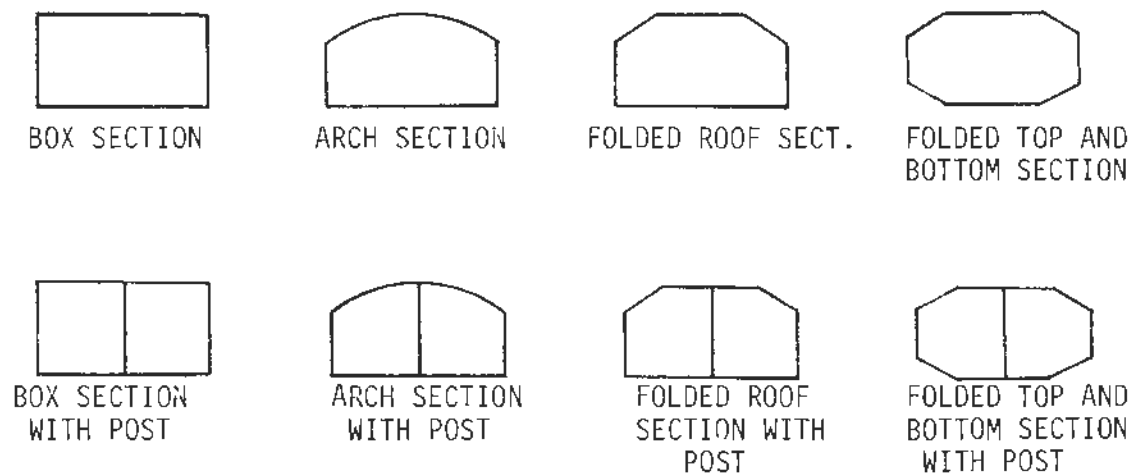


Figure 75. Selected Tunnel Configurations For The Cost Analysis

b. Internal Dimensions

Typical dimensions of a section, (Section 78 + 80,) of WMATA D4b project are illustrated in Figure 76. The figure indicates that the station structure is a box section with a center post. The internal dimensions of the box sections are specified below:

Width = 62 ft (18.90 m)
Height = 36 ft (10.97 m)

In addition, the length of the station is assumed to be 600 ft (182.88 m). For folded plate top and bottom sections, both with and without the center post, the height of the section is increased to 43 ft (13.10 m). This would allow the passenger platform to be constructed in the middle of the section.

c. Design Criteria

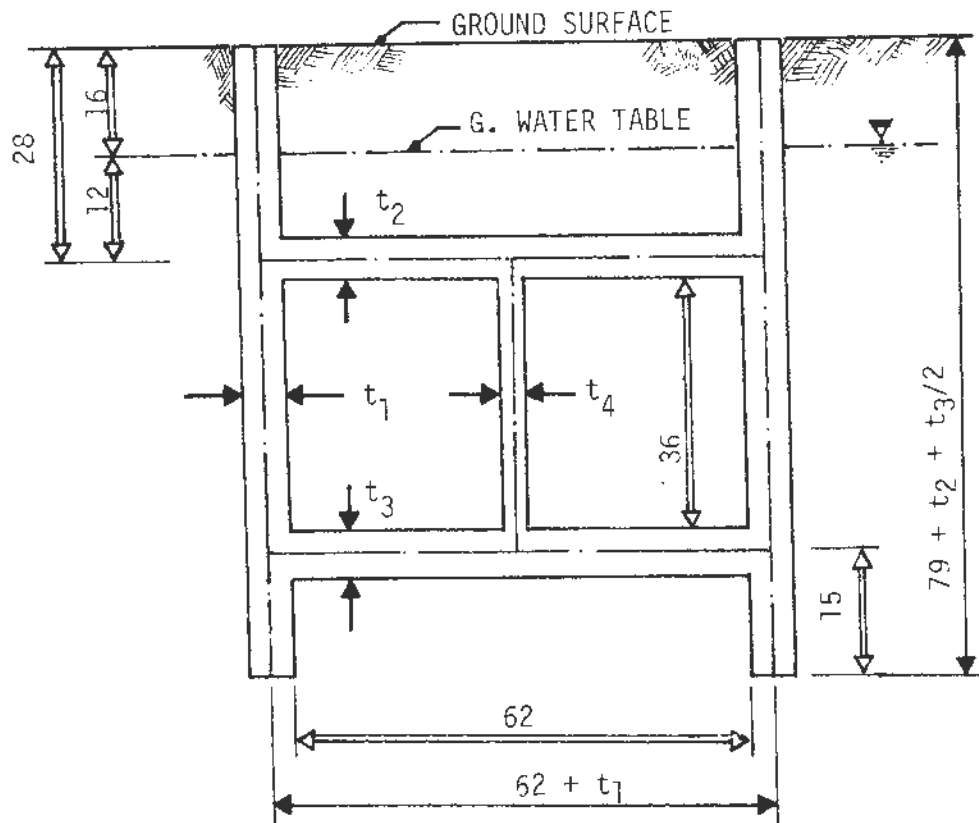
The design criteria for the present study correspond to the loading Case III that is used for the WMATA D4b project. This case assumes full vertical load, long-term horizontal load on one side of the structure, and short-term horizontal load on the other side. The structural analysis considers the top slab as unrestrained against horizontal translation, and horizontal springs are provided to account for elastic support of soil.

In addition, the following information on soil pressure, surcharge load, subgrade modulus, etc. has been considered uniformly for all tunnel configurations, and has been utilized in conjunction with the "Alternative Design Method" of ACI code (working stress method) to perform the parametric study.

Saturated Weight of Soil:	130 p.c.f	(20.42 kN/m ³)
Soil Weight Below G.W.T.:	68 p.c.f	(10.68 kN/m ³)
Unit Weight of Water	: 62.5 p.c.f	(9.82 kN/m ³)
D.L. of Concrete	: 145 p.c.f	(22.78 kN/m ³)
Street Surcharge	: 300 p.s.f.	(14.36 kN/m ²)
Surcharge from Buildings		
(one side)	: 960 p.s.f.	(45.95 kN/m ²)
(other side)	: 800 p.s.f.	(38.29 kN/m ²)
Subgrade Modulus	: 200 k.c.f.	(31,415.93 kN/m ³)
Soil Pressure Coefficients:	$K_a = 0.40$	(active)
	$K_o = 0.58$	(at-rest)

d. Parametric Study

The parametric study has been performed in accordance with the guidelines described in detail in the previous chapter. The results of the study are summarized in Table 11. As can be seen from the table, in the case of a tunnel with center post, the most desirable moment, shear, and axial load distribution



↔ : Fixed Dimension for all Configurations in ft.

↔ : Variable Dimension Depending Upon Design Options in ft.

Figure 76. Dimensions Of A Typical Tunnel Section

correspond to the dimensions of center post being 1.5 ft (0.46 m) by 3.0 ft (0.91 m), and to the center-to-center distance between the posts being 20 ft (6.1 m). For tunnel configurations without center post (cases 1 through 4 in Table 11), the numbers corresponding to t_1 , t_2 , . . . , t_5 have been found to be the most favorable dimensions as a result of the parametric study. The summary results of Table 11 are also illustrated in Figures 77 through 79 which show dimensions of various configurations and of various members in a particular configuration.

d. Cost Basis

The total cost of a subway station (or tunnel) is subdivided into the following three categories:

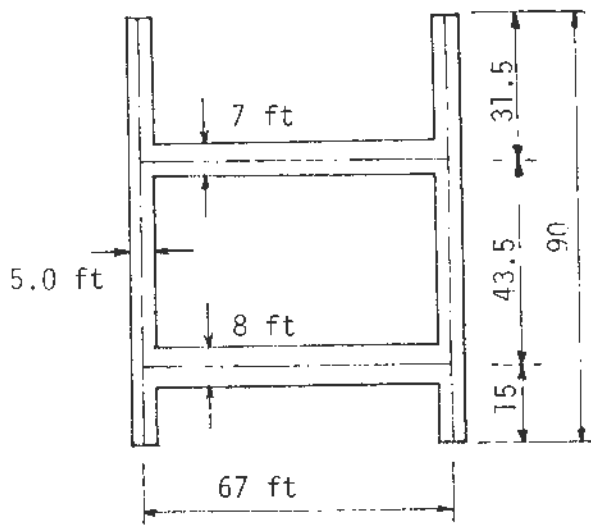
1. Cut and cover cost
 - (a) Excavation
 - (b) Backfill
 - (c) Bracing
2. Top and bottom slabs cost
 - (a) Top slab concrete
 - (b) Top slab reinforcing steel
 - (c) Bottom slab concrete
 - (d) Bottom slab reinforcing steel
 - (e) Center posts (if applicable)
3. Slurry wall cost

In the present study the unit cost of 5 ft (1.52 m) thick slurry wall is taken to be 10 percent higher than the unit cost of 4 ft (1.22 m) thick wall, reflecting the associated higher cost of equipment with the required larger capacities.

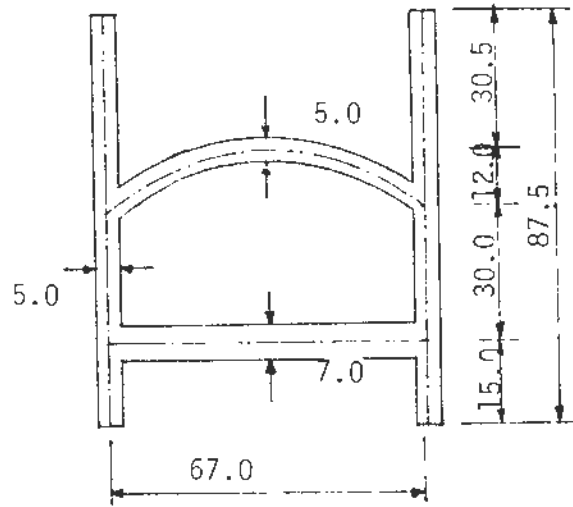
9.3 The Effect of Center Posts

In order to determine the dimensions and the spacing of the center posts, the effect of the posts on the behavior of the structure has been studied from both the architectural as well as the effectiveness viewpoints. On one hand, smaller posts and a larger distance between posts (large spacing) are architecturally more attractive. On the other hand, closely spaced large posts are structurally more effective. Therefore, various parametric studies have been performed combining these two options. In the analysis, the Finite Element Method, with both 3-D plate element and plane frame options, was used. The significant results are presented in Tables 12 and 13.

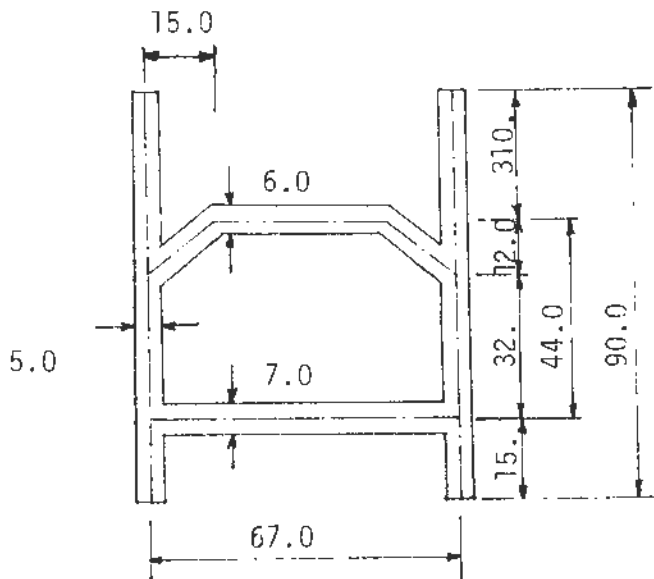
As a result of this study, we have decided to use 1.5 ft (0.46 m) x 3 ft (0.91m) posts. The distance between the posts was determined after having analyzed the section for 20 ft (6.10 m), 25 ft (7.62 m), and 50 ft (15.24 m) spacings. It can be noted in Figure 80 that, as spacing between posts is increased, the moment at the joints is increased, almost to the value corresponding to the section without posts. The axial compression on the post is



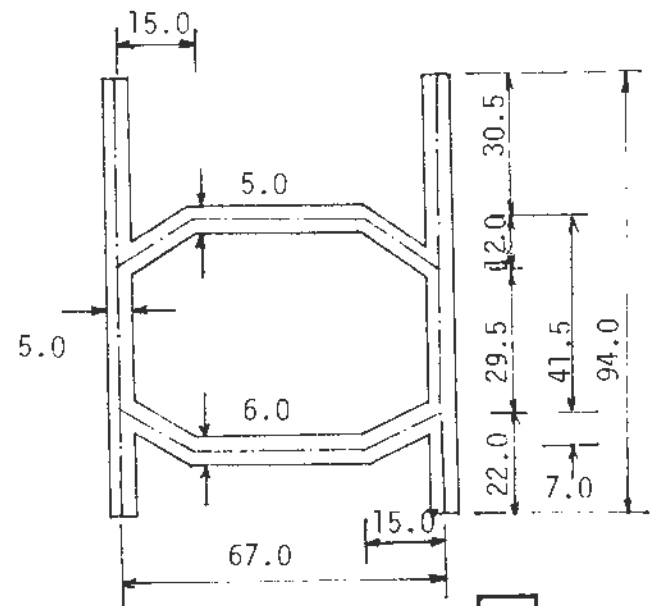
1U



2U



3U

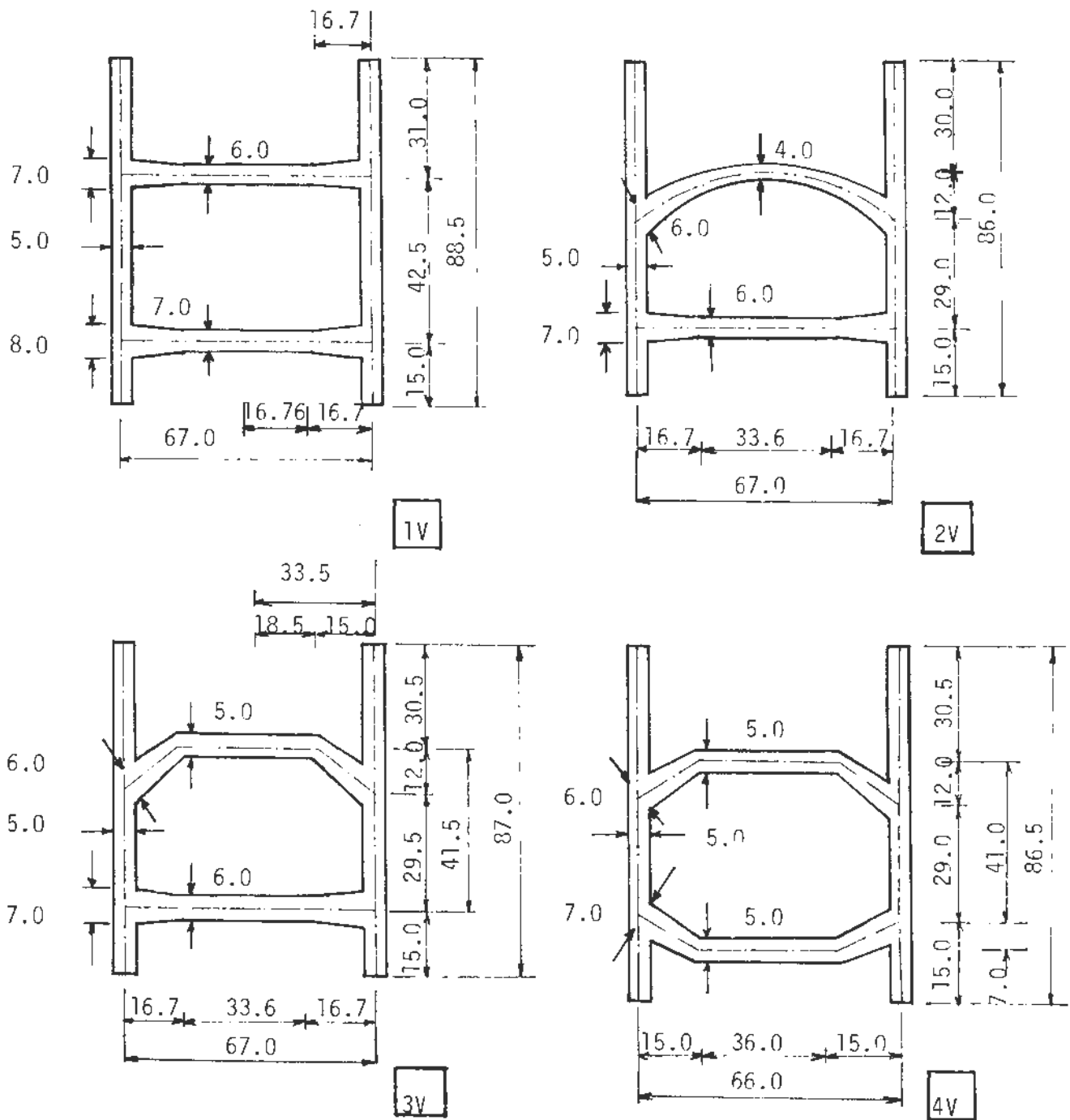


4U

NOTE: All units in ft

(1 ft=0.3m)

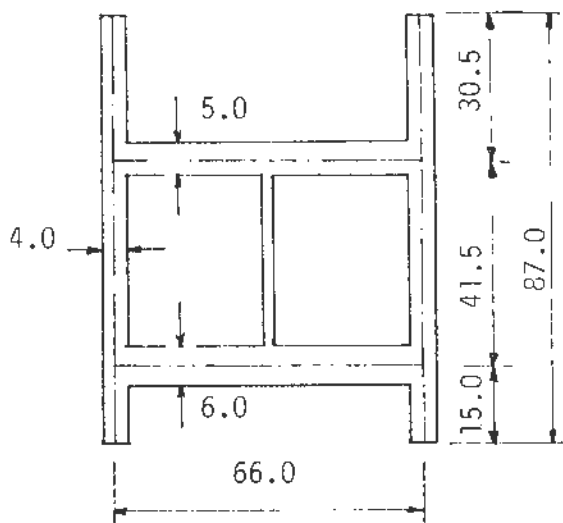
Figure 77. Sections Without Posts - Uniform Thickness



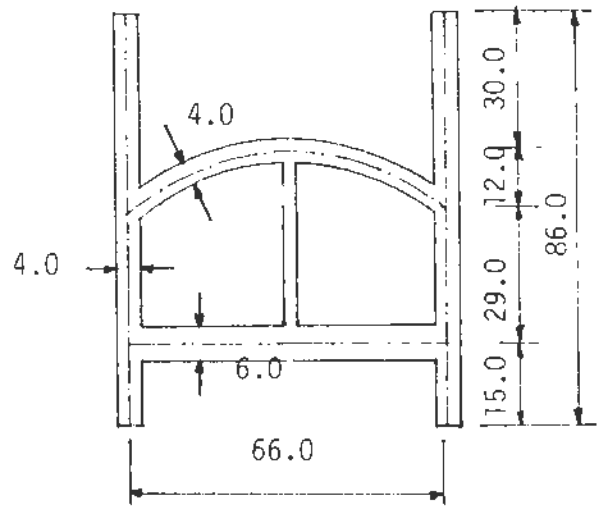
NOTE: All units in ft

Figure 78. Sections Without Post - Variable Thickness

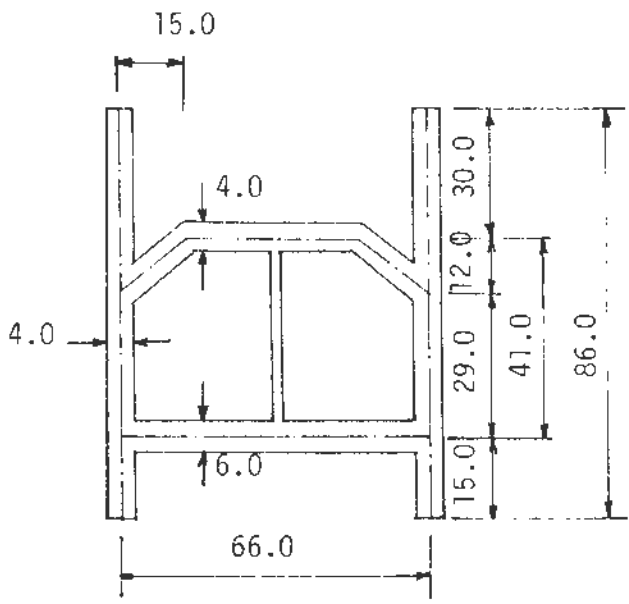
(1 ft=0.3m)



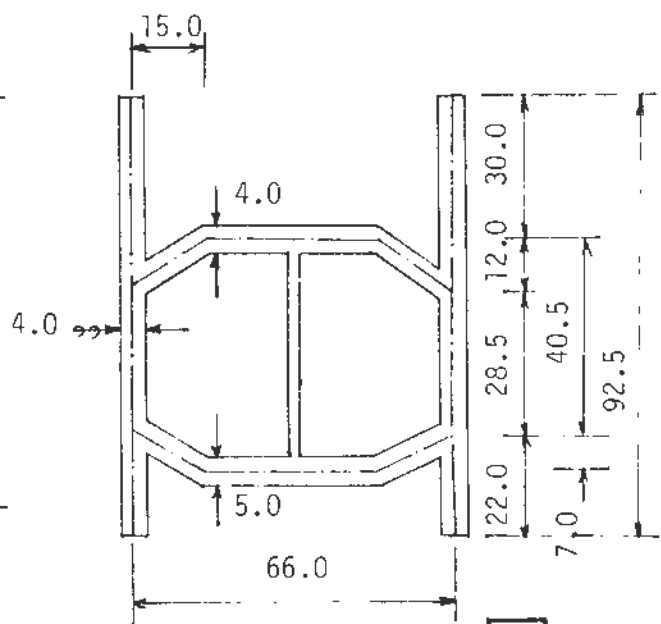
5UP



6UP



7UP



8UP

NOTE: All units in ft

Figure 79. Sections With Post - Uniform Thickness
(1 ft=0.3m)

TABLE 11

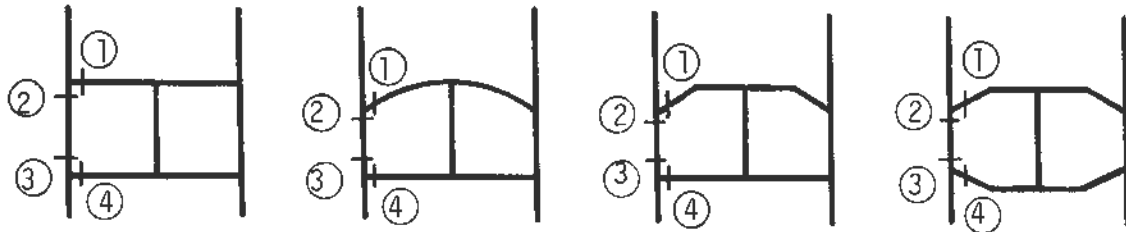
Summary Of Parametric Study Of Tunnel Dimensions

CASE	CONFIGURATION	TYPE	S.I. UNITS (METER)							U.S. CUSTOMARY UNITS (FT)						
			t ₁	t ₂	t ₃	t ₄	t ₅	t ₆	c	t ₁	t ₂	t ₃	t ₄	t ₅	t ₆	c
1		U	1.52	2.13	2.44					5	7	8				
		V	1.52	1.83	2.13	2.13	2.44			5	6	7	7	8		
2		U	1.52	1.52	2.13					5	5	7				
		V	1.52	1.22	1.83	1.83	2.13			5	4	6	6	7		
3		U	1.52	1.83	2.13					5	6	7				
		V	1.52	1.52	1.83	1.83	2.13			5	5	6	6	7		
4		U	1.52	1.52	1.83					5	5	6				
		V	1.22	1.52	1.52	1.83	2.13			4	5	5	6	7		
5		UP	1.22	1.52	1.83			.46 x .91	6.1	4	5	6			1.5 x 3.0	20
6		UP	1.22	1.22	1.83			.46 x .91	6.1	4	4	6			1.5 x 3.0	20
7		UP	1.22	1.22	1.83			.46 x .91	6.1	4	4	6			1.5 x 3.0	20
8		UP	1.22	1.22	1.52			.46 x .91	6.1	4	4	5			1.5 x 3.0	20

c: Center to Center Distance Between the Posts
 U: Uniform Thickness without Post
 V: Variable Thickness Without Post
 UP: Uniform Thickness with Post

TABLE 12

The Effect Of Post On The Bending Moment At The Joints



POSTS: 1.5 ft x 3.0 ft at 20.0 ft spacing

LOADING: UNSYMMETRICAL

#	BOX SECTION			ARCH SECTION			FOLDED ROOF SECT.			FOLDED TOP & BOTTOM SECTION		
	A	B	C	A	B	C	A	B	C	A	B	C
①	-1241.2	-559.3	55	-730.7	-220.1	70	-452.6	-63.8	86	-498.6	-96.2	81
②	-828.5	-535.3	35	-652.2	-222.6	66	-439.1	-105.9	76	-480.4	-152.4	68
③	-1061.4	-766.0	28	-873.4	-668.8	23	-935.9	-700.5	25	-739.9	-472.3	36
④	-1317.7	-870.0	34	-1244.0	-900.6	25	-1321.3	-897.5	32	-1047.2	-556.9	47

A; Bending Moment in Kip-ft/ft. for Sections Without Post

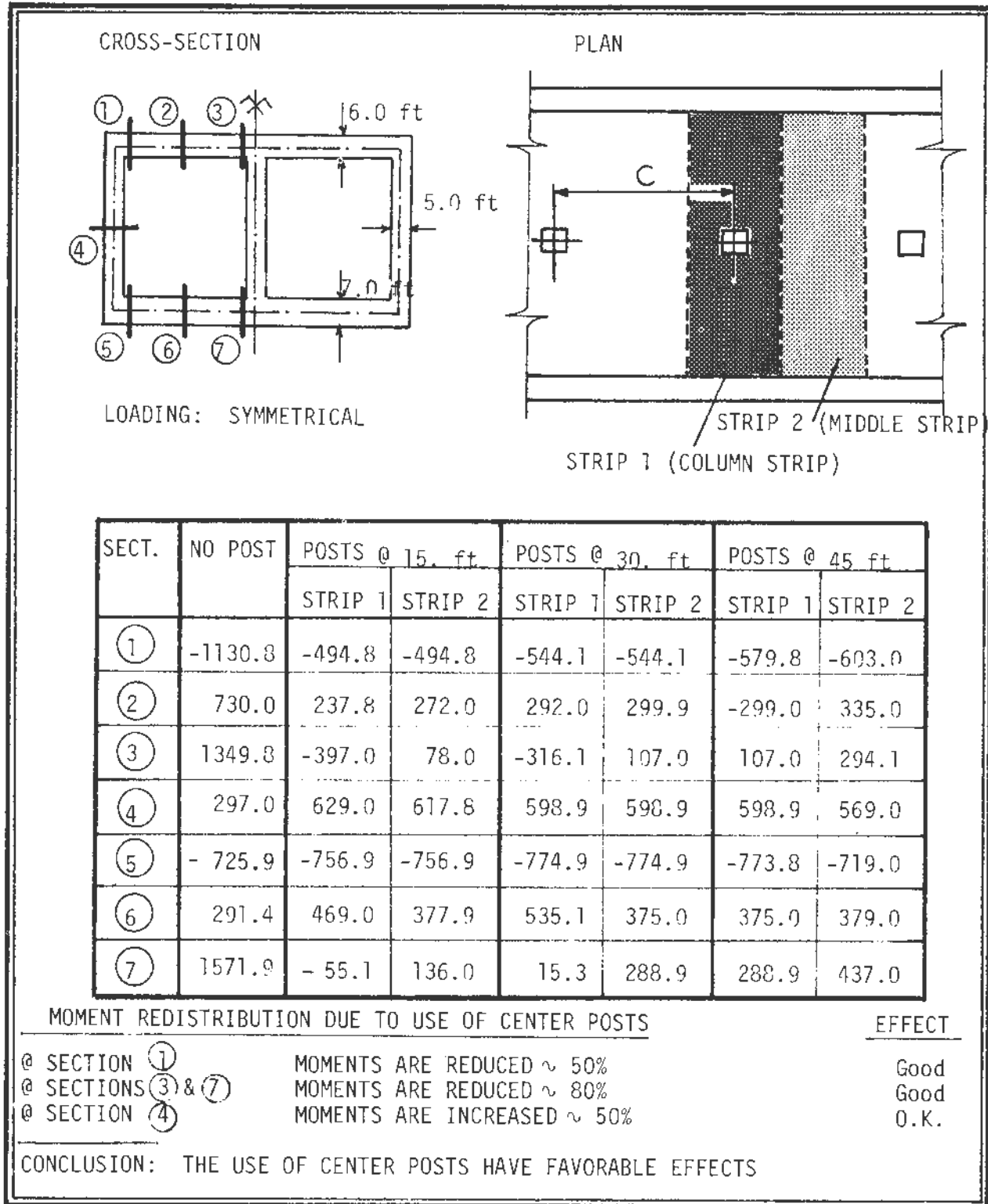
B; Bending Moment in Kip-ft/ft. for Sections With Post

C; Percentage of Reduction in Bending Moment

(1 ft=0.3m)

TABLE 13

The Effect Of The Center Posts Spacing On The Section



also increased to the value at which the design of the posts becomes almost impossible. As a result of these findings, we have decided to use 20 ft (6.10 m) center-to-center distance between the post.

9.4 Cost Analysis and Comparison

The cost analysis for four different design approaches with or without center posts was carried out in detail. These results are compared in graphic form (Figure 81) to indicate the relationship between the cost data and the tunnel configurations. It can be seen from the cost data that, in general, sections with the posts offer significant savings. However, when the center posts are used, architectural considerations should be kept in mind. More importantly, it should be noted that a well designed tunnel without post (i.e., folded plate top and bottom section of variable thicknesses) is as economical as the sections with posts. Moreover, sections without posts have an added advantage since they can be designed and constructed without sacrificing architectural esthetics. In the following chapter on the application of slurry wall as an integral part of permanent structures, we shall discuss the folded plate top and bottom design in connection with the hypothetical redesign of Federal Center, S.W. Station of WMATA.

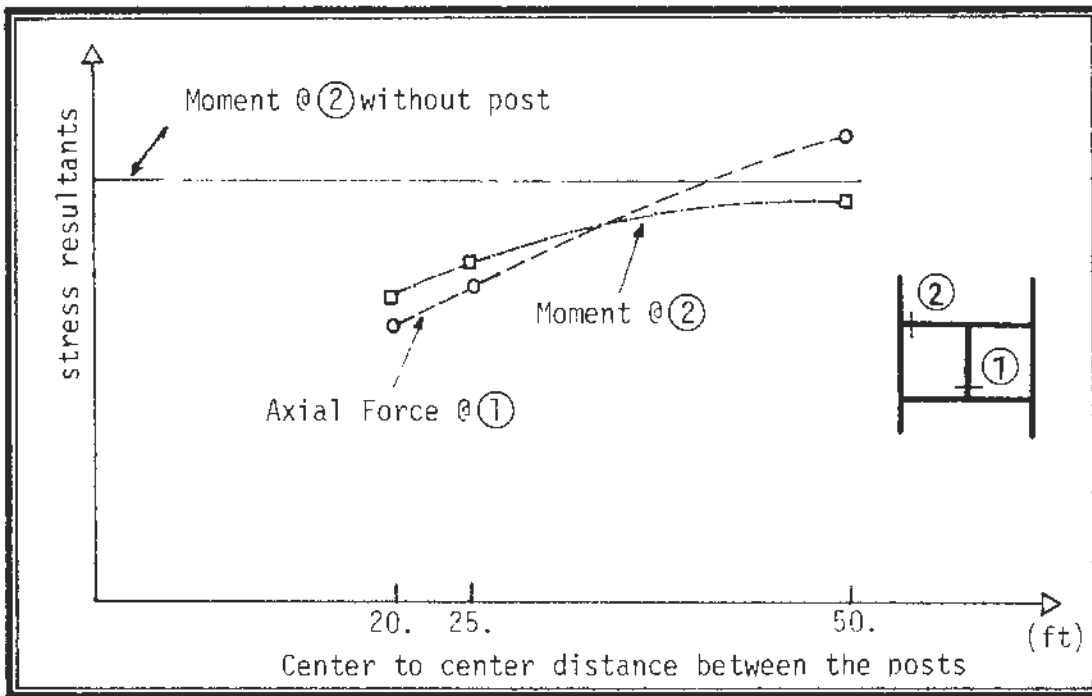


Figure 80. The Effect Of Spacing Of The Posts

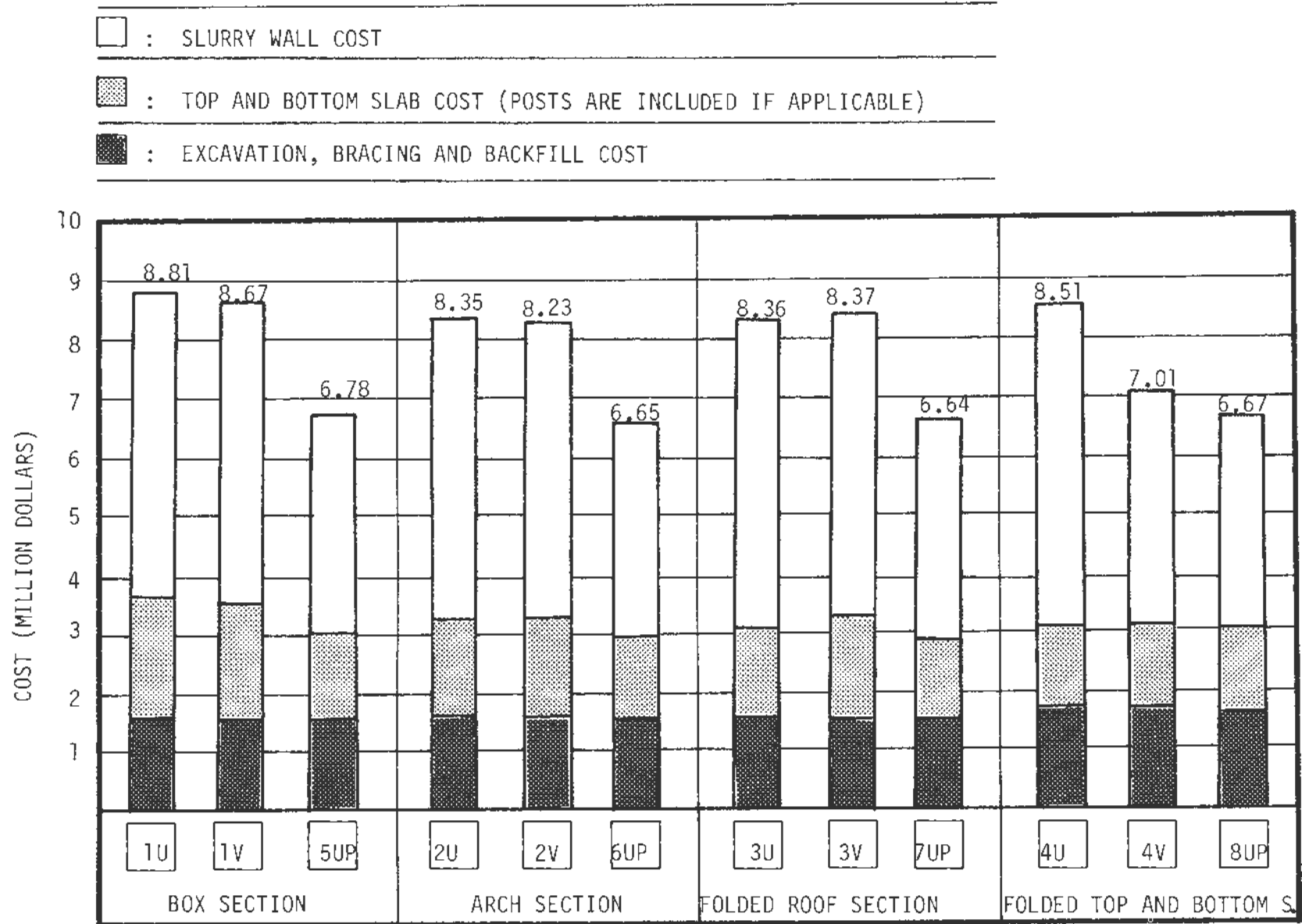


Figure 91. Relationships Between The Tunnel Configuration And The Cost Data

CHAPTER 10

APPLICATION: REDESIGN OF FEDERAL CENTER, S.W. STATION

10.1 Site Selection and Description

One of the goals of this project was to develop a preliminary design using slurry walls as part of the permanent structure for a site where temporary slurry walls were used.

It should be emphasized here that the purpose of the present study is to investigate the applicability of slurry walls to a permanent structure while utilizing a realistic design. This effort includes the design of a slurry wall that will withstand both temporary and long-term loadings. Further, it demonstrates the feasibility of incorporating that wall into a final structure by showing sufficient structural detail to make a realistic cost comparison.

A test case for the application of such a design effort was discussed at length with the Board of Consultants. Several possible cases were reviewed. Of several sites considered, the Federal Center, S.W. Subway Station of the Washington Metropolitan Area Transit Authority (WMATA) was finally selected. (see also Domestic Case Studies, Chapter 5). WMATA consultants designated the Federal Center S.W. Subway Station as Section D4b.

The D4b project is in downtown Washington, D.C., and has an 80 ft (24.4 m) right-of-way. It is bordered on both sides by short sized, heavy commercial buildings. The average ground water level is 20 ft (6.1 m) below grade. A site plan and profile of the station are shown in Figure 82.

The D4b project site was chosen as an example in this study for several reasons. The project had been well documented with an adequate amount of available information on the performance of the slurry wall since a monitoring program had been conducted both during and after construction. The documentation was easily accessible in the WMATA downtown Washington, D.C. office. In addition, two members of the Board of Consultants who had worked on this contract, J. Iffland and G. Tamaro, had direct working knowledge of the D4b project. Finally, the use of Option B (a temporary slurry wall) in the D4b project had already demonstrated a substantial cost savings. Those cost figures were available for comparison.

10.2 Design Criteria

As mentioned before, the site of the D4b project has a narrow right-of-way and a high ground water level. There are also large and heavy buildings adjacent to the site, which, at the time of construction, required protection against settlement or tilting. During the excavation phase, traffic was rerouted and an open-cut method was utilized.

The architectural design requirements called for underground stations on WMATA's "Metro" to have arched roofs with no internal supports. Figure 82 shows a plan and profile of the Federal Center S.W. station and Figure 83 shows a typical section.

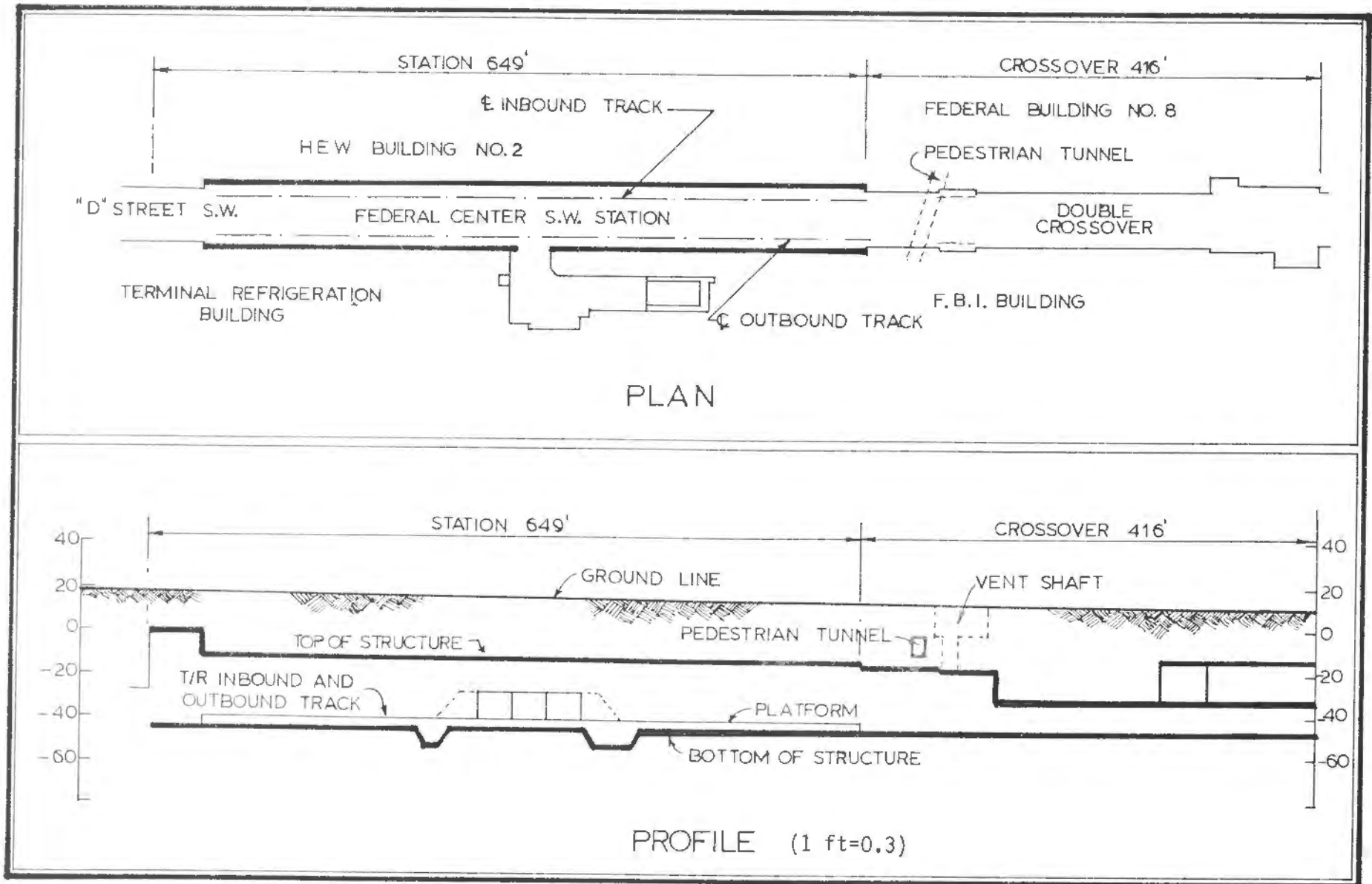


Figure 82. Site Plan And Profile For The WMATA Federal Center S.W. Station (D4h Project)

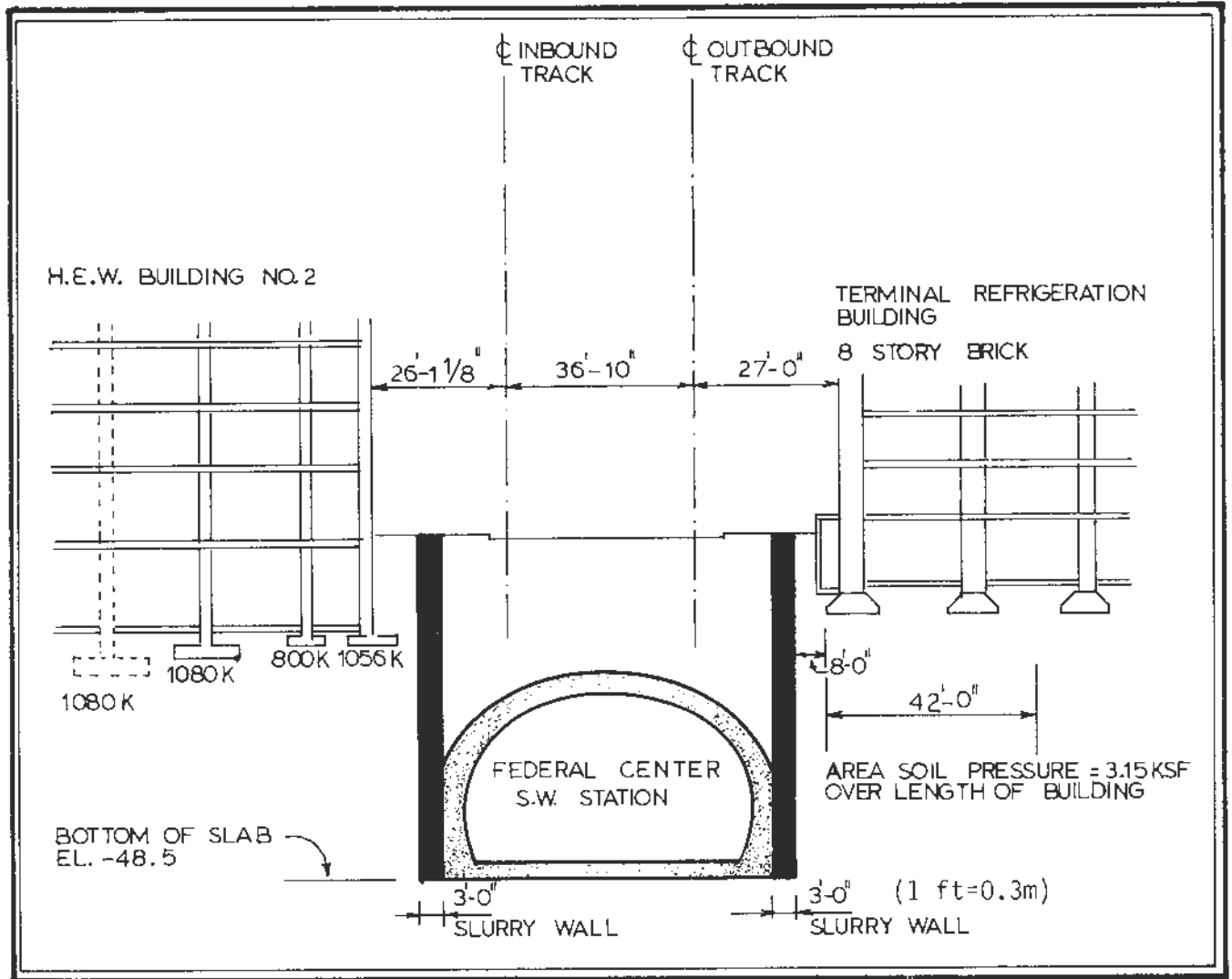


Figure 83. Section 78+80 Of The WMATA Fedral Center S.W. Station (D4b Project)

The criteria design of section D4b were:

- a. Use active soil pressure for temporary wall analysis.
- b. Use symmetrical at-rest soil pressure for permanent structure analysis.
- c. Use asymmetrical soil pressure, i.e. full at-rest soil pressure on one side of the structure and half at-rest soil pressure on the other side for permanent structure analysis.

The short term lateral pressure on the slurry wall which met the above criteria is shown in Figure 84. The loading included lateral soil and hydrostatic pressures as well as the lateral component of surcharge on the wall and main bracing members. In Figure 84 note the differences in the loading conditions on the slurry wall with and without ground dewatering.

Figure 38 shows the long-term loading, both lateral and vertical, on station section 78+80. It can be seen by comparing Figures 84 and 38 that the lateral soil pressure is greater during long-term loading than that encountered during short-term loading.

10.3 Analysis and Design

In Section 5 of this report, an analysis of different section geometries was discussed in detail. From this analysis it was concluded that the folded plate roof section offered the following advantages.

1. Maximum moment occurring at the midspan is reduced by 35 percent.*
2. Maximum deflection also occurring at the midspan is reduced by 40 percent.*
3. At the wall-roof slab joint, the moment is reduced by 40 percent for symmetrical loading and by 20 percent for asymmetrical loading.

Consequently, the folded plate roof structure was chosen for the study. Additionally, the inverted folded plate structure was chosen for the bottom slab. It was found in the preliminary analysis that this choice of the bottom slab reduced the moment at the wall/slab joint substantially.

The architectural view of a station section, using the final design suggested by the writers, is shown in Figure 85. In accordance with the WMATA design criteria specified previously for the D4b project, the proposed folded plate section was analyzed for four loading conditions:

- Case I Station structure (frame) under short-term loading conditions during backfill operations. Two sub-cases were studied to determine when the top brace between the slurry walls should be removed.
- a. Area above roof: not backfilled; top brace between slurry walls in place; high ground water conditions.

* When compared to the plain box section under the same loading conditions.

LATERAL PRESSURE ON SLURRY WALL					
LOCATION	D	CAUSED BY EARTH AND WATER		DUE TO SURCHARGE FROM ADJACENT BUILDINGS	
		GROUND OUTSIDE OF COFFERDAM NOT DEWATERED	GROUND OUTSIDE OF COFFERDAM DEWATERED		
		<p>SHAPE OF PRESSURE DIAGRAM</p>	<p>SHAPE OF PRESSURE DIAGRAM</p>		
		LATERAL EARTH & HYDROSTATIC PRESSURE P_D & P_w (P.S.F.)	LATERAL EARTH PRESSURE, P_D (P.S.F.)		
		MAIN BRACING MEMBERS & WALL	MAIN BRACING MEMBERS & WALL		
1. Sta. 77+03 to Sta. 79+35	Adjacent to Term. Refrig. Bldg.	34	$P_D=1500$ $P_w=3000$	$P_D=2270$	$W=1900\text{psf}$
	Adjacent to HEW Building #2	34	$P_D=1500$ $P_w=3000$	$P_D=2270$	$W=2250\text{psf}$
2. Sta. 79+35 to Sta. 83+00	Adjacent to HEW Building #2	46	$P_D=1470$ $P_w=3000$	$P_D=1830$	$W=2250\text{psf}$
	Elsewhere	65	$P_D=1680$ $P_w=3000$	$P_D=2550$	—
3. Sta. 83+00 to Sta. 86+80	Adjacent to FOB #1	44	$P_D=860$ $P_w=3000$	$P_D=1820$	$W=2600\text{psf}$
	Adjacent to FOB #8	39	$P_D=620$ $P_w=3000$	$P_D=1300$	$W=1900\text{psf}$
4. East of Sta. 86 + 80		65	$P_D=1680$ $P_w=3000$	$P_D=2550$	—

Figure 84. Short -Term Loading On The D4b Final Structure

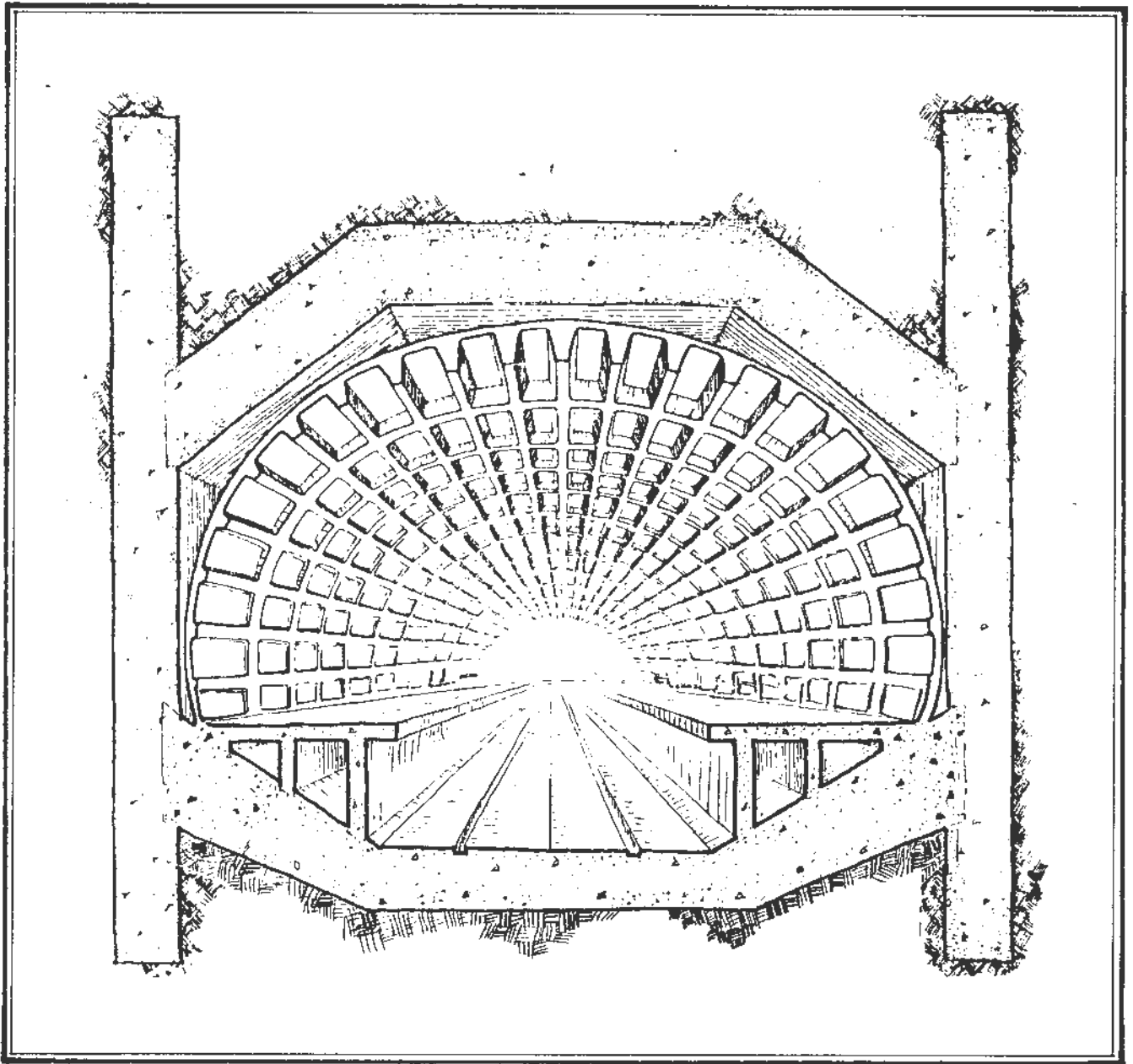


Figure 85. Sectional Perspective Of The
Alternate Station Design To D4b

- b. Before backfill of area above roof begins: top brace removed but high ground water conditions still exist.
- Case II Station structure (frame) subjected to full vertical load and long-term horizontal loads assuming symmetrical soil pressure.
- Case III Station structure (frame) subjected to full vertical load with long-term horizontal load on one side and short-term horizontal load on the other side (asymmetrical soil pressure). The following subcases were studied:
- a. The frame loaded asymmetrically, unrestrained against sidesway.
 - b. The top panel of the roof restrained against horizontal movement, but the upper portion of the slurry walls and the wall-roof intersection points free to sway laterally.
 - c. The frame restrained against horizontal movement at the wall-roof intersection points, but the top panel of the roof and the upper portion of the slurry walls free to sway laterally.
 - d. The entire roof restrained against horizontal movement, but the upper portion of the slurry walls free to sway laterally.
- Case IV Station structure (frame) subjected to full vertical load but short-term horizontal load, with ground water below structure depth and saturated soil conditions.

We note in passing that the above loading conditions were derived from what was used in the original design. In this manner a direct comparison between the original and the alternative designs can be made. For the same reason, the design approach employed in this report was the same, namely, the "working stress" method. It was incidental that the actual calculation was done in a microcomputer using a specially in-house written software named CHITRAN.

The loading conditions and moments, shears and axial thrusts at critical sections are summarized in Figure 86 and Table , respectively. After the sections are analyzed, the reinforcing steel requirement for selected cross-sections was calculated, as shown in Figures 87 and 88.

With the wall thickness and required reinforcement known, the slurry walls could be checked through the various loading conditions to which they would be subjected. Figures 89 and 90 show two analysis sheets summarizing the results. The figures also show the magnitude of stress change that the wall undergoes during bracing removal operations. As seen in Figure 89, the maximum displacement of the wall, with all levels of temporary bracing installed (i.e., at completion of site excavation), is 0.0385 in (0.0978 cm). Figure 90 shows that repositioning one brace and removing another to allow fabrication of the roof slab to take place, can cause large moment increase at midspan of the wall with a maximum elastic displacement of 0.1907 in (0.459 cm).

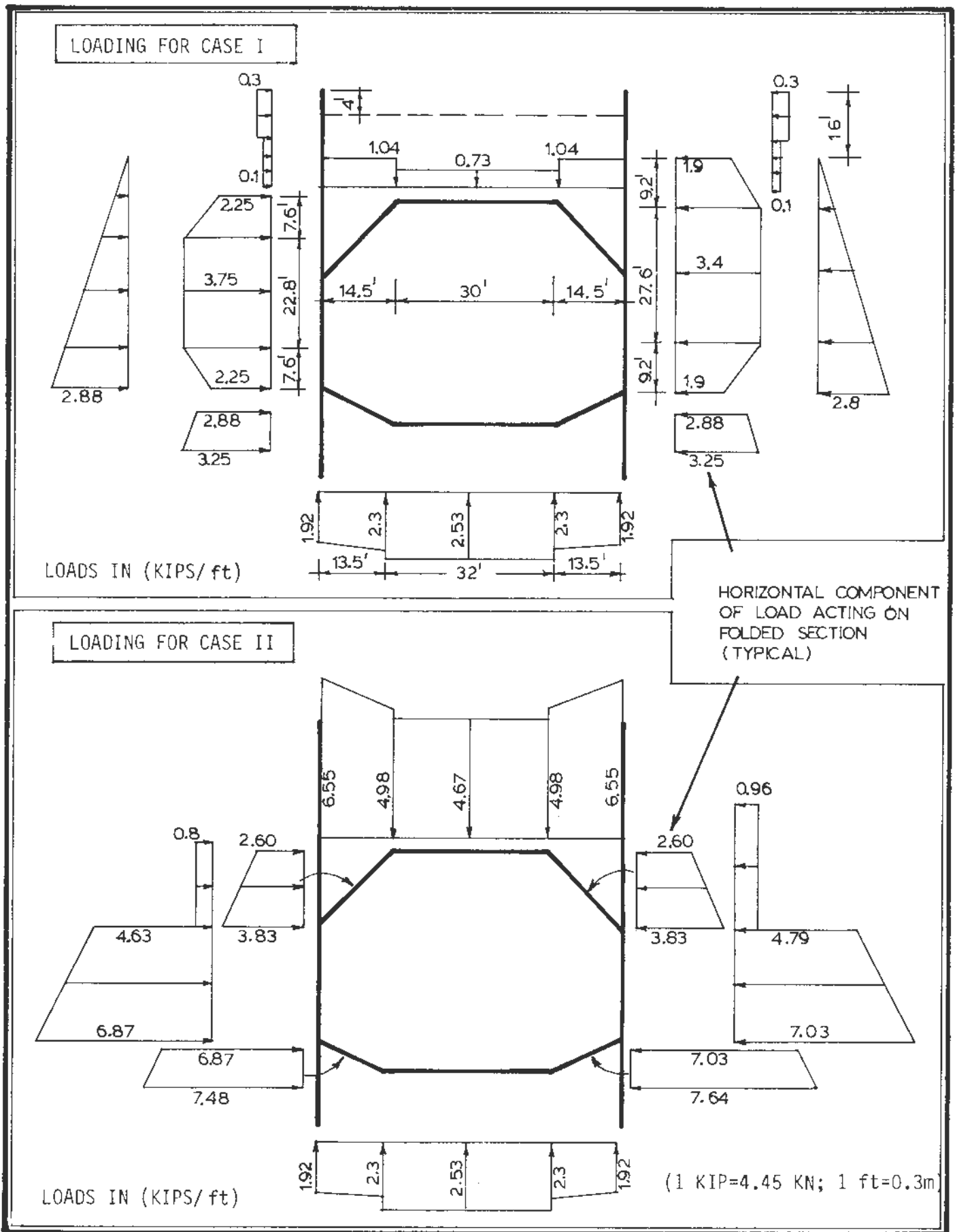


Figure 86. Loading Conditions Corresponding To The WMATA Design Criteria For The D4b Project

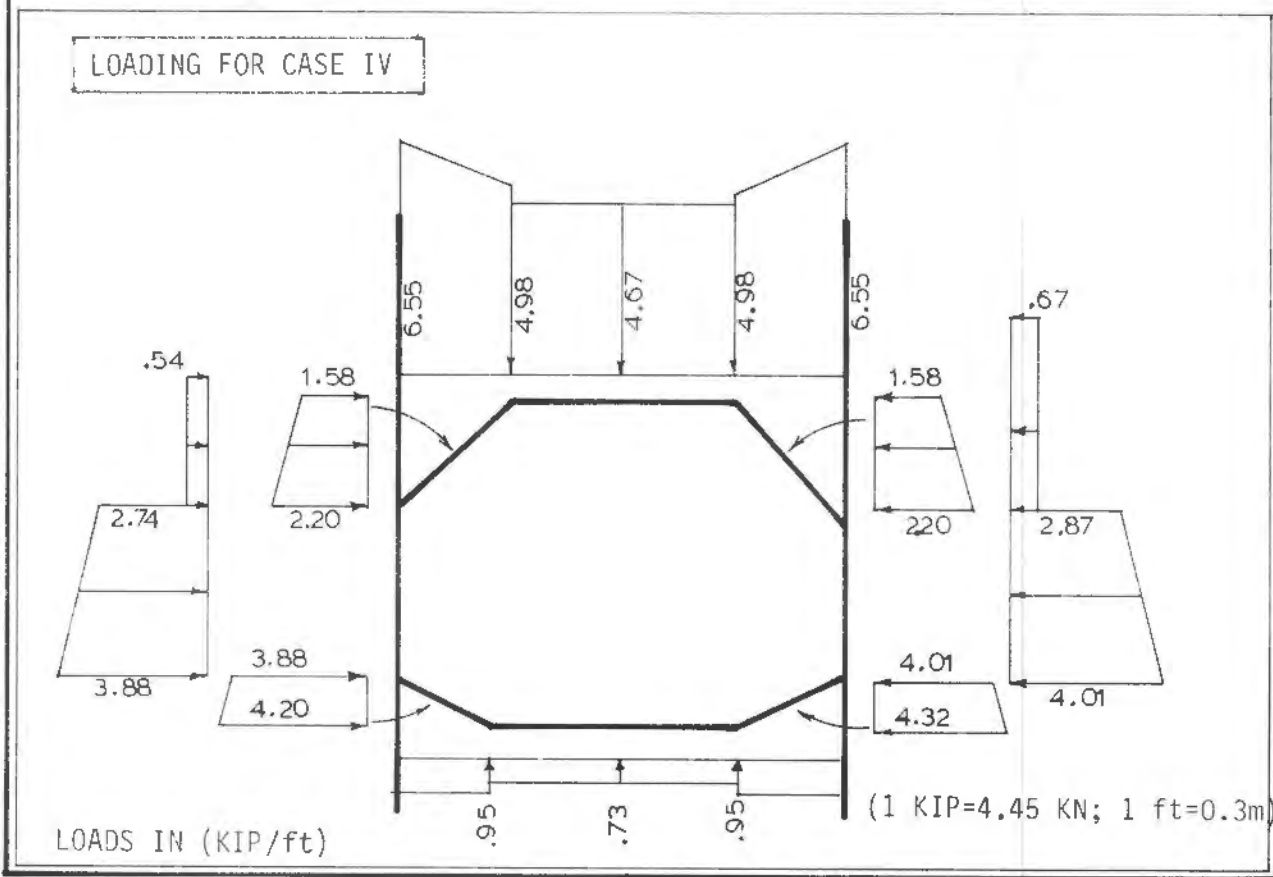
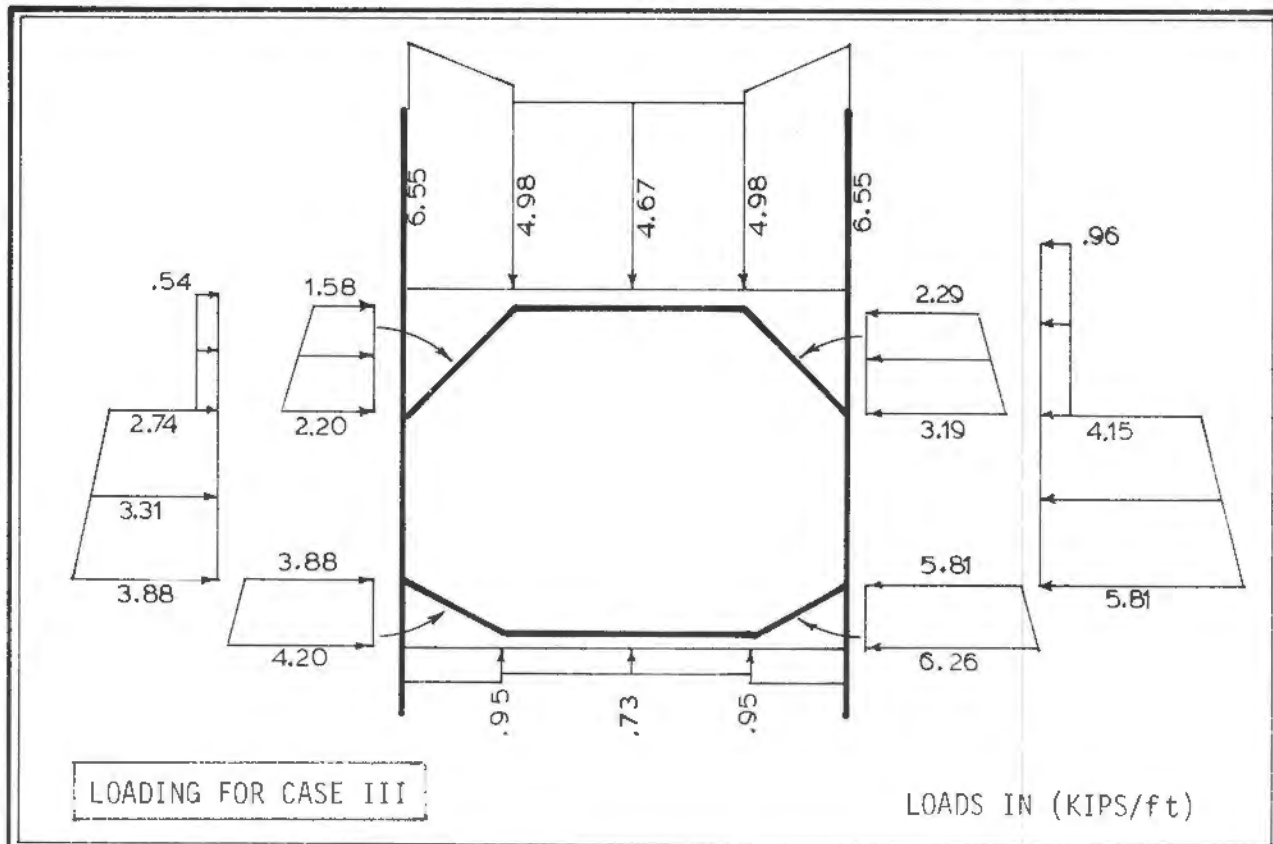
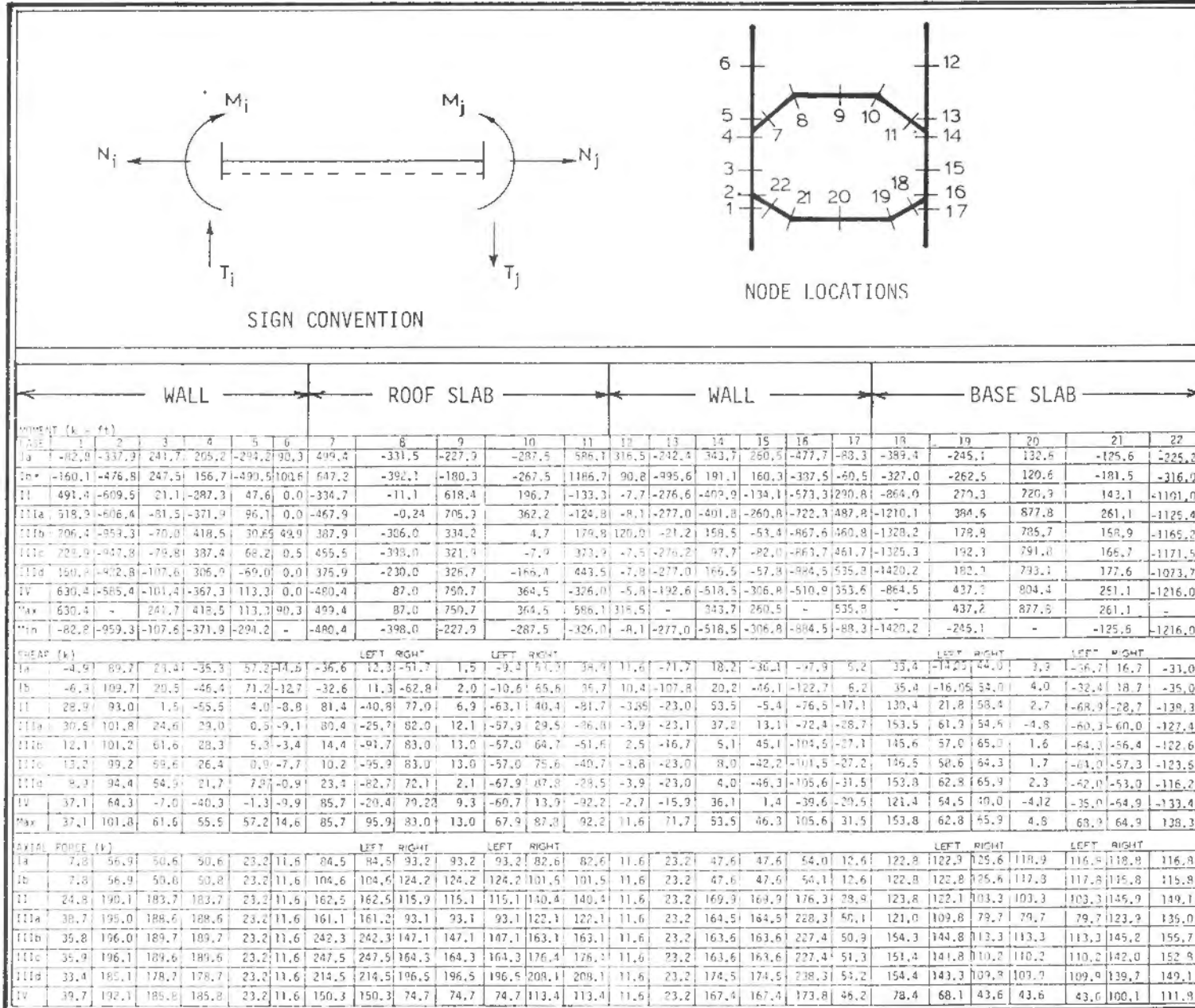


Figure 86 . cont. Loading Conditions corresponding To The WMATA Design Criteria For The D4b Project

Table 14

Results Of Finite Element Analysis



		WALL						ROOF SLAB						WALL				BASE SLAB					
MOMENT (k - ft)		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22
1a		-82.8	-337.9	241.7	205.2	-294.2	90.3	499.4	-331.5	-227.9	-287.5	586.1	316.5	-242.4	343.7	260.5	-477.7	-88.3	-389.4	-245.1	132.6	-125.6	-225.2
2a		-160.1	-476.8	247.5	156.7	-499.5	101.6	647.2	-392.1	-180.3	-267.5	1186.7	90.8	-995.6	191.1	160.3	-337.5	-60.5	-327.0	-262.5	120.6	-181.5	-316.0
11		491.4	-609.5	21.1	-287.3	47.6	0.0	-334.7	-11.1	618.4	196.7	-133.3	-7.7	-276.6	-409.9	-134.1	-573.3	1290.8	-864.0	270.3	720.9	143.1	-1191.0
111a		518.9	-606.4	-81.5	-371.9	96.1	0.0	-467.9	-0.24	705.7	362.2	-124.8	-8.1	-277.0	-401.8	-260.8	-722.3	487.8	-1210.1	384.5	877.8	261.1	-1125.4
111b		706.4	-959.3	-70.0	418.5	30.65	49.9	387.9	-306.0	334.2	4.7	179.8	120.0	-21.2	158.5	-53.4	-467.6	160.8	-1328.2	178.9	785.7	158.9	-1165.2
111c		228.9	-947.8	-78.8	387.4	68.2	0.5	455.5	-348.0	321.4	-7.9	373.9	-7.5	-276.2	97.7	-82.0	-863.7	461.7	-1325.3	192.1	791.0	166.7	-1171.5
111d		150.8	-432.8	-107.6	306.9	-69.0	0.0	375.9	-230.0	326.7	-166.4	443.5	-7.8	-277.0	166.5	-57.8	-844.5	535.8	-1420.2	182.7	793.1	177.6	-1073.7
1V		630.4	-585.4	-101.4	-367.3	113.3	0.0	-480.4	87.0	750.7	364.5	-326.0	-5.8	-192.6	-518.5	-306.8	-510.9	353.6	-864.5	437.7	804.4	251.1	-1216.0
Max		630.4	-	241.7	418.5	113.3	90.3	499.4	87.0	750.7	364.5	586.1	316.5	-	343.7	260.5	-	535.8	-	437.7	877.8	261.1	-
Min		-82.8	-959.3	-107.6	-371.9	-294.2	-	-480.4	-398.0	-227.9	-287.5	-326.0	-8.1	-277.0	-518.5	-306.8	-884.5	-88.3	-1420.2	-245.1	-	-125.6	-1216.0
SHEAR (k)		LEFT						RIGHT						LEFT				RIGHT					
1a		-4.9	89.7	21.4	-35.3	57.2	14.6	-36.6	12.3	-51.7	1.5	-9.4	50.3	34.9	11.6	-71.7	18.2	-30.1	-47.9	5.2	35.4	-14.2	44.0
1b		-6.3	109.7	20.5	-46.4	71.2	-32.7	-32.6	11.3	-62.8	2.0	-10.6	65.6	35.7	10.4	-107.8	20.2	-46.1	-122.7	6.2	35.4	-16.0	34.0
11		28.9	93.0	1.5	-55.5	4.0	-8.8	81.4	-40.8	77.0	6.9	-63.1	40.4	-81.7	-3.5	-23.0	53.5	-5.4	-76.5	-17.1	130.4	21.8	58.4
111a		30.5	101.8	24.6	29.0	0.5	-9.1	80.4	-25.7	82.0	12.1	-57.9	29.5	-86.8	-3.9	-23.1	37.2	13.1	-72.4	-28.7	153.5	61.2	54.6
111b		12.1	101.2	61.6	28.3	5.3	-3.4	14.4	-91.7	83.0	13.0	-57.0	64.7	-51.6	2.5	-16.7	5.1	45.1	-104.5	-27.1	145.6	57.0	65.0
111c		13.2	99.2	58.6	26.4	0.9	-7.7	10.2	-95.9	83.0	13.0	-57.0	75.6	-40.7	-3.8	-23.0	8.0	-42.2	-101.5	-27.2	136.5	58.6	64.3
111d		8.3	94.4	54.5	21.7	7.2	-0.9	23.4	-82.7	72.1	2.1	-67.9	47.8	-28.5	-3.9	-23.0	4.0	-46.3	-105.6	-31.5	153.8	62.8	65.9
1V		37.1	64.3	-7.0	-40.3	-1.3	-9.9	85.7	-20.4	79.2	9.3	-60.7	13.9	-92.2	-2.7	-15.9	36.1	1.4	-39.6	-20.5	121.4	54.5	40.0
Max		37.1	101.8	61.6	55.5	57.2	14.6	85.7	95.9	83.0	13.0	67.9	87.8	92.2	11.6	71.7	53.5	46.3	105.6	31.5	153.8	62.8	65.9
AXIAL FORCE (k)		LEFT						RIGHT						LEFT				RIGHT					
1a		7.8	56.9	50.6	50.6	23.2	11.6	104.5	104.5	124.2	124.2	124.2	101.5	101.5	11.6	23.2	47.6	47.6	54.0	12.6	122.8	122.8	125.6
1b		7.8	56.9	50.0	50.8	23.2	11.6	104.6	104.5	124.2	124.2	124.2	101.5	101.5	11.6	23.2	47.6	47.6	54.1	12.6	122.8	122.8	125.6
11		24.8	190.1	183.7	183.7	23.2	11.6	162.5	162.5	115.9	115.1	115.1	140.4	140.4	11.6	23.2	169.9	169.9	176.3	28.9	123.8	122.1	103.3
111a		38.7	195.0	188.6	188.6	23.2	11.6	161.1	161.2	93.1	93.1	93.1	122.1	122.1	11.6	23.2	164.5	164.5	228.3	50.1	121.0	109.8	79.7
111b		35.8	196.0	189.7	189.7	23.2	11.6	242.3	242.3	147.1	147.1	147.1	163.1	163.1	11.6	23.2	163.6	163.6	227.4	50.9	154.3	144.8	113.3
111c		35.9	196.1	189.6	189.6	23.2	11.6	247.5	247.5	164.3	164.3	164.3	176.4	176.4	11.6	23.2	163.6	163.6	227.4	51.3	151.4	141.8	110.2
111d		33.4	185.1	178.7	178.7	23.2	11.6	214.5	214.5	196.5	196.5	196.5	208.1	208.1	11.6	23.2	174.5	174.5	238.3	51.2	154.4	143.3	109.8
1V		39.7	132.1	185.8	185.8	23.2	11.6	150.3	150.3	74.7	74.7	74.7	113.4	113.4	11.6	23.2	167.4	167.4	173.8	46.2	78.4	68.1	43.6

*Note: 1a proved more appropriate than 1b, therefore 1b values not considered in summarizing maximum & minimum values.

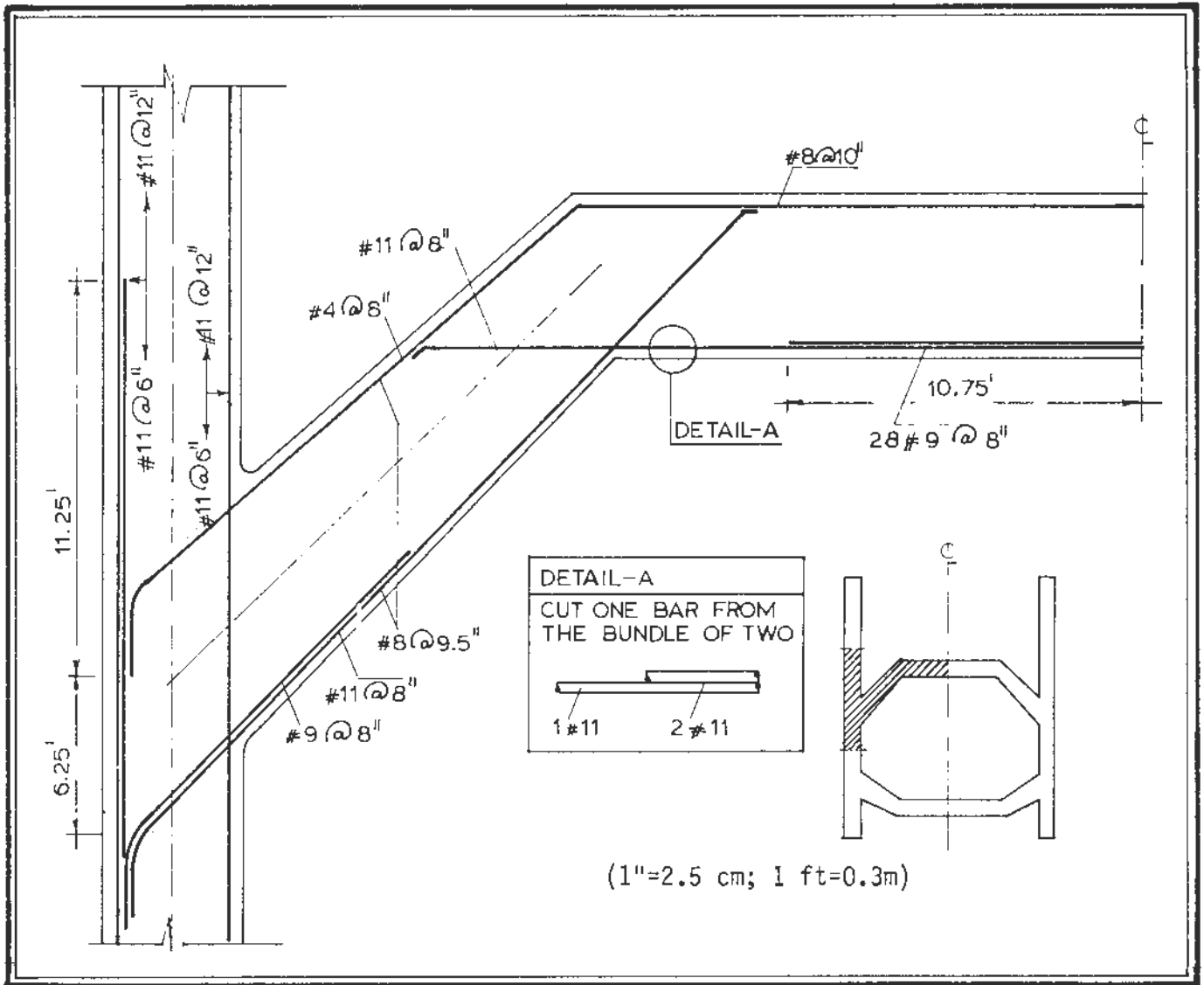


Figure 87. Reinforcing Steel Requirements; For The Upper Portion Of The Alternate Design

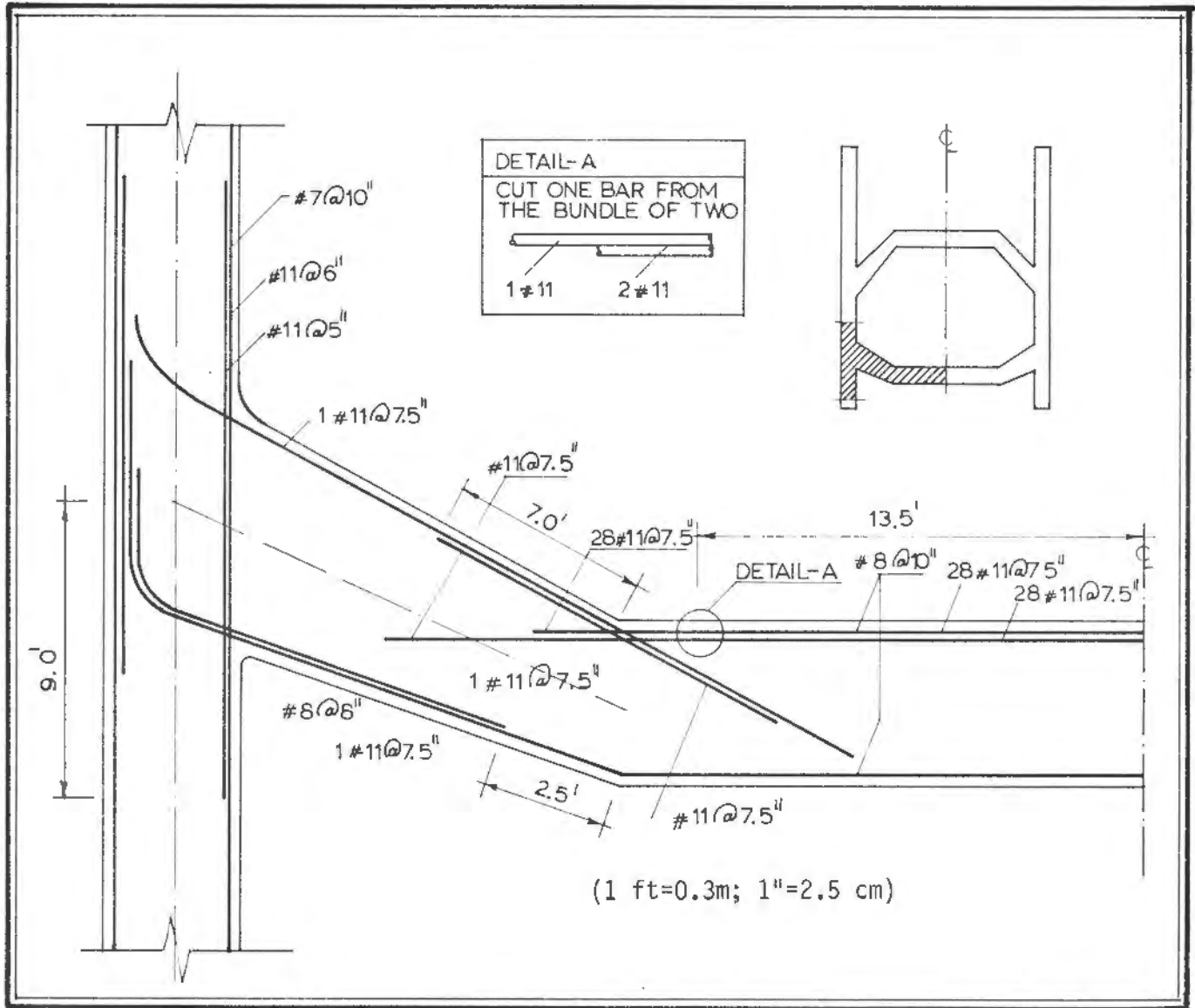


Figure 88: Reinforcing Steel Requirements For The Lower Portion Of The Alternate Design

SLURRY WALL ANALYSIS: NORTH WALL
 (EXCAVATION OF WALL HAS JUST BEEN COMPLETED)

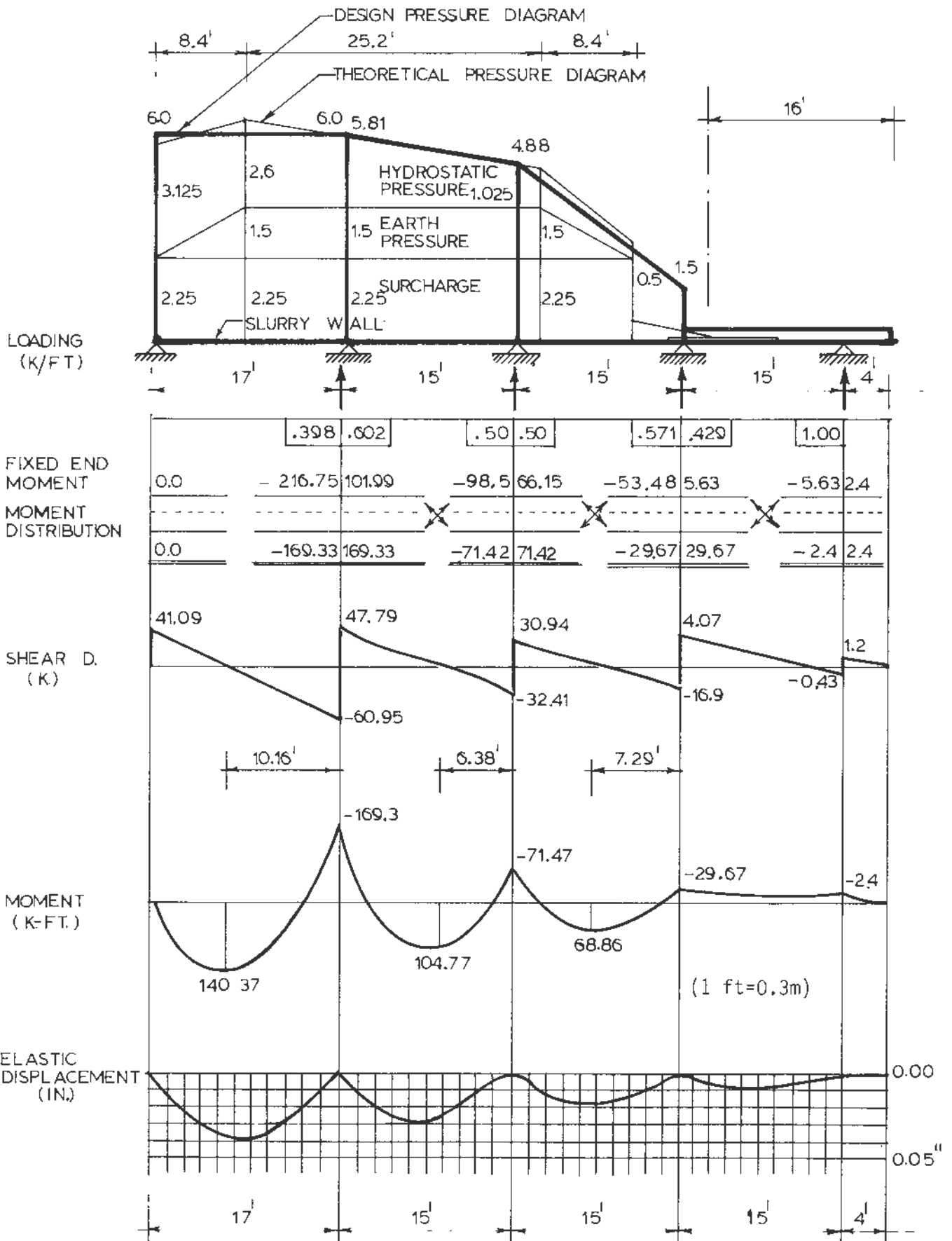


Figure 89. Slurry Wall Analysis

SLURRY WALL ANALYSIS: NORTH WALL

(INVERT SLAB HAS BEEN POURED, THE BOTTOM BRACE IS REPLACED AS SHOWN, AND NEXT BRACE UP IS REMOVED SO THAT ROOF SLAB CAN BE CONSTRUCTED)

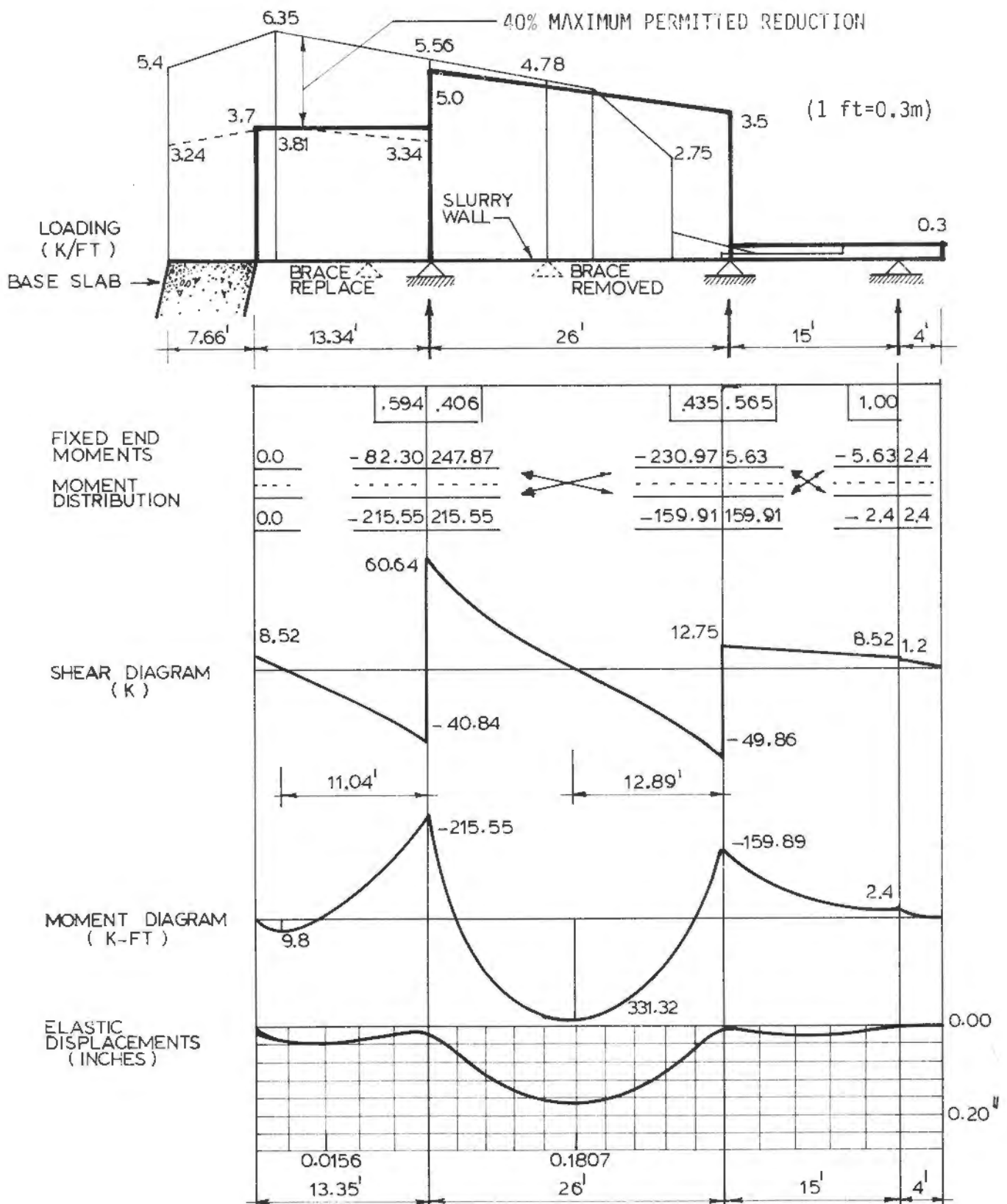


Figure 90. Slurry Wall Analysis

Proper bracing, locating, and removal schemes are required and must be adhered to in order to keep the maximum displacement within specific tolerances. For the D4b project, this tolerance was set at 3/8 in. (0.953 cm) between brace supports during excavation, and 5/8 in. (1.588 cm) between braces and the portion of the permanent structure used for supporting the wall. The node displacements of the slurry wall in the present study are well within these limits.

10.4 Comments Concerning the Alternate Design

During the design process, the writers attempted to duplicate the appearance of the station as built on the D4b project. A picture of the existing station is shown in Figure 91 for comparison with Figure 95 which shows the present design. It is seen that the resemblance is striking and there should be no architectural objections to the present design. Furthermore, the alternate design offers the following additional advantages:



Figure 91. View Of Federal Center S.W. Subway Station

First, the alternative design has smaller exterior width than the existing design while keeping the interior width the same. In addition, a sizeable additional interior space was gained through the use of folded plate as the floor slab. The additional area was estimated to be 229 ft² (21.27 m²). Admittedly this additional space is gained only through the additional expenses of deeper excavation; however, any additional space can be put into functional use. We remark in passing that this is carried out on the assumption that the top-most point of the folded plate must be kept on the same level as the flat slab. If this is not necessary, the the top surface of the lower portion of the folded plate corresponds to the top surface of the flat slab, this extra excavation would not be needed. For details, see Figure 92.

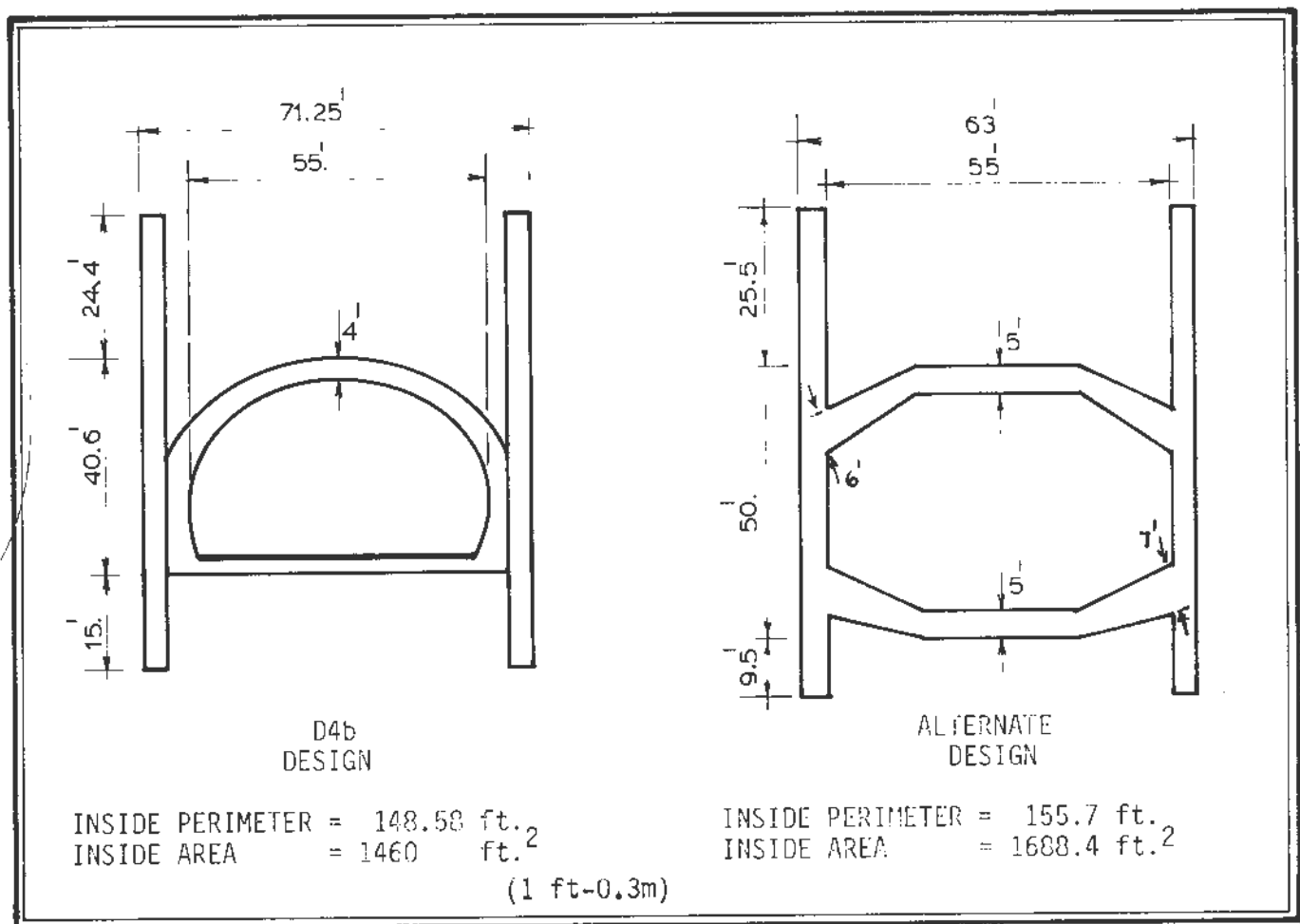


Figure 92. Station Dimensions

Second, the proposed scheme facilitated the platform design and track placement shown graphically in Figure 93. Optimum use of the folded plate called for side platforms (Type I) allowing trains to travel in the most open portion of the station. For patron convenience, however, the designers wanted a centrally positioned platform (Type II). In both cases, openings under the platforms can allow for utility pipe and electrical cable trays.

Third, the proposed design offered a wide choice of roof arrangements. Available options are shown in Figure 94. With Option A, the station roof simply remained as cast, i.e., a plain concrete finish. Option B permitted the roof to be cast with a waffle pattern, if such a finish is desired, for acoustic or architectural reasons. Such a waffle pattern could easily be cast in place during roof fabrication, using readily available forms. Option C provided a false roof suspended from the structural roof. The false roof has been utilized on other stations in WMATA's subway system and, in the alternate design scheme allowed additional space for pipes or electrical runs.

10.5 Cost Analysis

During the design phase of the D4b project, a conventional temporary earth support system, soldier pile and lagging, was considered for the excavation phase of the project. The structural engineers considered supporting the bordering buildings with underpinning which, they soon realized, would be quite extensive and costly. To avoid the underpinning requirements, an alternate design was suggested which called for the use of temporary slurry walls as rigid cofferdams during excavation and station construction. When the project was opened for bids, the designers allowed contractors to submit separate bids on Option A (a conventional flexible wall cofferdam) and Option B (the rigid slurry wall cofferdam). The lowest bid was for Option B, which was then implemented in the project. The economic advantage of a rigid wall system over conventional flexible systems was amply demonstrated by a calculated saving of more than \$2 million in the bid price (see Table 2 of Chapter 2).

An accurate indication of the advantage of using slurry walls in the final structure can be demonstrated by comparing the final costs of the D4b design with those of the alternate design developed by the writers. The basis for such an analysis is the bid price list from the actual D4b project.

For this cost comparison, various items (concrete, rebar, etc.) were directly identified in the bid summary package obtained from WMATA. For the alternate design, i.e., the permanent slurry wall, those costs were assigned by consensus of the Board of Consultants. The dollar value for the false roof was taken from the engineer's estimate of \$35 per sq ft in 1979 for a similar roof construction on another WMATA station. All prices are based on the 1972 value of the dollar, so that the dollar value of the false roof was determined by dividing the \$35 figure by the yearly inflation rate since 1972.

The cost comparison, therefore, is straightforward and accurate. Table 15 summarizes this analysis for a 600 ft station and shows that the alternate design, which incorporated slurry walls as part of the permanent station structure,

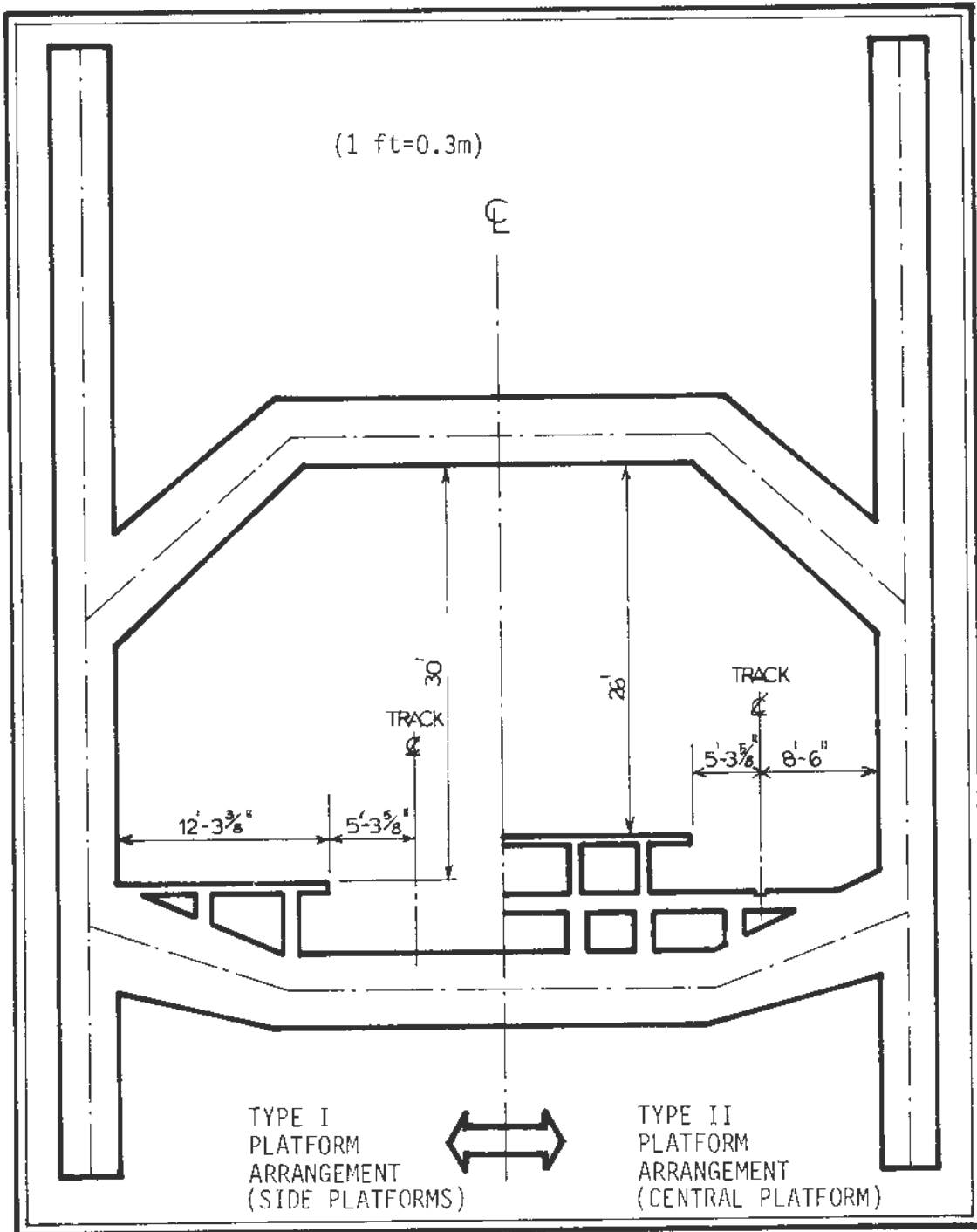


Figure 93. Alternate Design Allows Choice Of Platform Arrangement

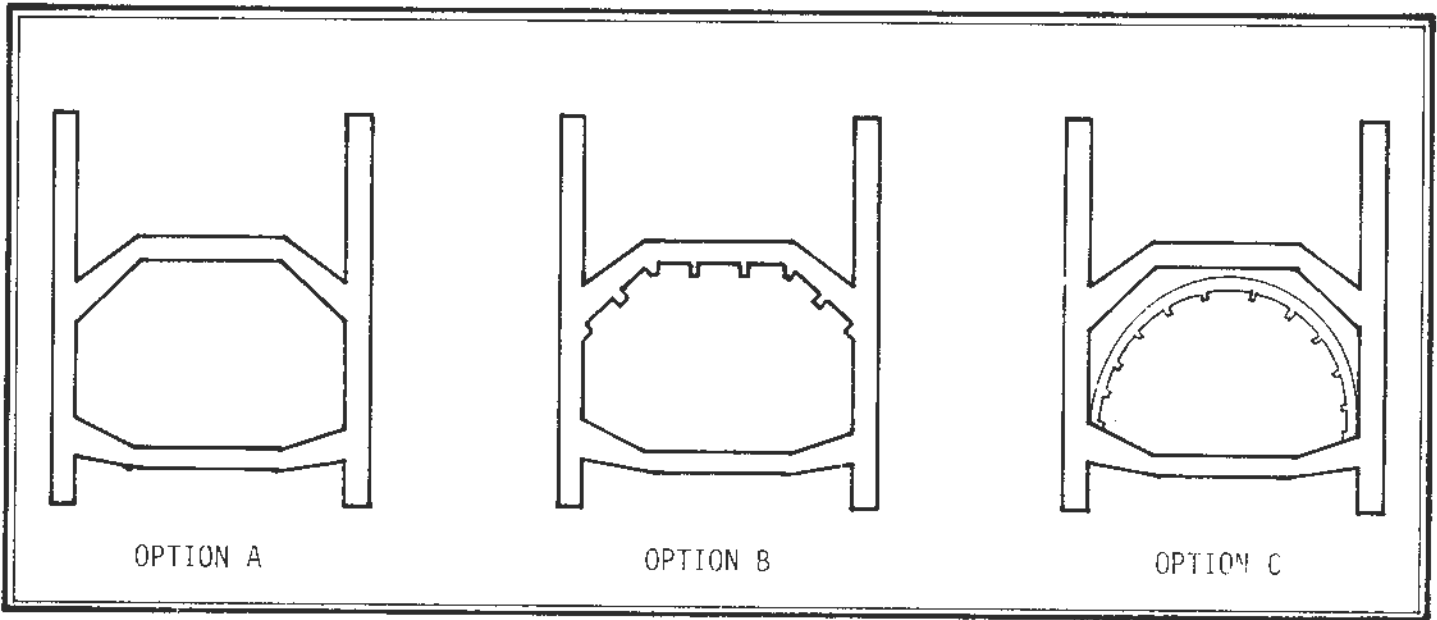


Figure 94. The Various Choices Of Roof Arrangement Available With The Alternate Design

Table 15

Cost Comparison Between The Actual D4b Design And
Alternate Design

ITEM	UNIT	UNIT PRICE (\$)	D4b DESIGN		CAI'S ALTERNATE DESIGN			
			QUANTITY	COST	QUANTITY	OPTION A COST	OPTION B COST	OPTION C COST
EXCAVATION	CY	10.00	95,859.95	958,599.50	86,166.67	861,666.70	861,666.70	861,666.70
BACKFILL	CY	4.00	50,777.78	203,111.11	34,500.00	138,000.00	138,000.00	138,000.00
SLURRY WALL	LS			3,152,275.98		3,376,773.59	3,376,773.59	3,376,773.59
BRACING	LS			200,956.94		200,956.94	200,956.94	200,956.94
CONCRETE :								
UPPER ARCH	CY	85.00	6,825.00	580,125.00				
LOWER ARCH	CY	85.00	4,148.00	352,580.00				
BASE SLAB	CY	50.00	10,768.00	538,400.00				
REINFORCING STEEL :								
ARCH	LB	0.25	7,224,857.00	1,806,214.25				
BASE SLAB	LB	0.25	2,677,545.00	669,386.40				
CONCRETE :								
ROOF SLAB	CY	50.00			7,457.78	372,889.00		372,889.00
	CY	85.00			7,457.78		633,911.30	
BASE SLAB	CY	50.00			7,395.56	369,778.00	369,778.00	369,778.00
REINFORCING STEEL :								
ROOF SLAB	LB	0.25			1,107,450.00	276,872.50	276,872.50	276,872.50
BASE SLAB	LB	0.25			2,009,664.00	502,416.00	502,416.00	502,416.00
PRECAST WAFFLE ARCH	SF	21.00			58,320.00			1,224,720.00
TOTAL				8,461,649.18		6,099,342.73	6,360,365.03	7,324,062.73
SAVINGS OVER D4b :						2,362,306.45	2,101,284.15	1,137,586.45

*BASED ON 1972 DOLLARS FROM D4b BID SUMMARY SHEETS

CY = CUBIC YARD
LB = POUND

LS = LUMP SUM
SF = SQUARE FOOT

(1 lb=4.45N; 1 ft=0.3m; 1 yd=0.9m)

provided a calculated savings of as much as \$2.3 million over the original D4b design. Option C, which included a false hung waffle roof that would most nearly replicate the actual as-built station, offered an anticipated savings of over \$1 million. However, alternate design Options A & B offered comparable station interiors with additional space at appreciable savings (both over \$2 million) giving engineers and architects a valid basis for considering their use.

CHAPTER 11

CONCLUDING REMARKS

Results of this study have shown that slurry walls used as permanent structures provide a viable alternate to conventional temporary structures, such as, the flexible wall cofferdam, soldier pile lagging, etc. during construction. Advantages of permanent slurry walls are enumerated below:

- a. Provide rigid support to the adjacent soil and minimize the ground movement.
- b. Provide an adequate support for the surcharge load from adjacent structure and eliminate the need for underpinning.
- c. Act as a watertight diaphragm thereby reducing the need for dewatering (provided that the slurry wall is keyed into the underlying impervious layer).
- d. Keep the water table more or less constant throughout the life of the structure and thus prevent excessive consolidation under, as well as surrounding, the structure, when subsoil consists of clay. This prevents excessive settlement.
- e. Provide a number of environmental advantages during construction which, include quieter operation, less traffic interruption, etc.

Slurry walls, however, are not a panacea to solve all deep foundation problems, either structural or architectural. Slurry wall, as a structure, is costly. To justify its adoption the tangible and intangible savings associated with its use, e.g., dewatering and underpinning costs, etc., must be weighed to offset the initial capital outlay. In some cases, however, it was chosen purely by default in that other alternatives of construction methods were ruled out for non-economical reasons. One well known example is the Harvard Square Station in Cambridge, Massachusetts, where many historic sites had to be preserved at any cost.

To enhance the usefulness as well as economical advantages of slurry wall, it is desirable to use it both as temporary and later, as a part of the permanent structure. This can be done with a proper design procedure. It is instructive to list the following observations for the consideration of the slurry wall designers.

- a. Properly designed, slurry wall can be both economical and functional. In the redesign of the Federal Center Station, it was shown that a permanent slurry wall could have been designed and constructed thereby providing a combined savings of more than \$4 million, i.e., the use of slurry walls as a temporary earth support did result in a \$2.3 million savings, and the use of slurry wall as part of the permanent structure could have resulted in an additional \$2 million savings.

- b. In common with any structure of high degree indeterminacy, the station cross-section must be carefully chosen so that the moment allocation at different locations will be equitable. In particular, the geometrical configuration has a great deal of influence on the magnitude of the critical moments. It was shown that curvilinear profile in the roof and/or the floor slabs can substantially reduce the critical moments at the joints.
- c. The critical moments of a rigidly-jointed station frame generally occurs at the joints between the slabs and the slurry walls. The moments are of such great magnitude, that slurry wall thickness and rebar requirements become outside the limits of practicability.
- d. For ordinary dimensions of subway stations, center columns spaced about 15 ft apart are recommended.
- e. Moment releasing joints or hinged joints should be considered for use in joining the floor slab with the slurry walls. For reasons of stability, the joint between the roof slab and the slurry wall should be left rigid. To reduce the sidesway, due to unequal lateral load, counterforts should be added to the slurry wall and/or the slabs, as appropriate.
- f. The common design criteria which requires an unequal lateral pressure of soil on the foundation structure is overly conservative and unfairly penalizing. The adoption of any subway work should be carefully weighed and investigated.
- g. Prestressed slurry wall did not show any distinct advantage in this particular design study. Because of costs involved, its use is not deemed justifiable.
- h. Double wall or composite wall construction is advantageous on technical grounds, except for the fact that there is no standard method to design the shear connectors between the tremie concrete and the in-situ concrete. This question is still open and is a suitable area for further research.
- i. In lieu of bracing, permanent tiebacks can be used to advantage, in terms of both performance and economics, provided easements can be obtained from the owners of the adjacent areas.

LIST OF RELATED PUBLICATIONS

REPORT NUMBER

1. FHWA-RD-73-40 "CUT-AND-COVER TUNNELING TECHNIQUES - VOL. 1
A STUDY OF THE STATE OF THE ART"

SVERDRUP & PARCEL AND ASSOC., INC.

FEBRUARY 1973

ABSTRACT:

This study of cut-and-cover tunneling techniques in urban areas considers environmental quality, geotechnical investigation and analysis, ground support, ground water control, permanent structure, restoration, cost considerations and major problems. Both United States and foreign techniques are reviewed.

This volume summarizes the study findings. In addition to describing alternate methods of construction, it identifies key problems, needed construction improvements, and future research goals.

The other volume of this report is FHWA-RD-73-41, entitled "Appendix."

- FHWA-RD-73-41 "CUT-AND-COVER TUNNELING TECHNIQUES - VOL. 2 -
APPENDIX."

This volume consists of summaries of significant articles, summaries of recent noise-control legislation, and the Ontario, Canada, expropriations act.

The other volume of this report is FHWA-RD-73-40, entitled "A Study of the State of the Art".

2. FHWA-RD-74-57 "PROCEEDINGS OF WORKSHOP ON CUT-AND-COVER TUNNELING:
PRECAST AND CAST-IN-PLACE DIAPHRAGM WALLS CONSTRUCTED
USING SLURRY TRENCH TECHNIQUES"

D'APPOLONIA, DAVID J.; D'APPOLONIA, E.; NAMY, DOMINIQUE

JANUARY 1974

ABSTRACT:

The workshop, conducted September 21, 1973, reviewed various aspects of the design, construction, and performance of traditional sheeting systems and examined, in detail, recently developed technology for precast and cast-in-place diaphragm walls constructed using slurry trench techniques. It included recommendations for construction management and innovative contracting concepts for reducing construction costs and time.

Approximately 60 percent of the cost of a tunnel built by cut-and-cover methods is associated directly with the cost of the wall itself. The main advantages of diaphragm walls installed by the slurry trench method over conventional sheeting or soldier pile and lagging type construction are:

1. The same wall can be used as the temporary retaining wall and the permanent structural wall.
2. The movements associated with diaphragm wall construction and excavation inside the walls are small.
3. Diaphragm walls are watertight compared to conventional sheeting.
4. The construction operations are more compatible with the urban environment.

REPORT NUMBER

3. FHWA-RD-75-128 "LATERAL SUPPORT SYSTEMS AND UNDERPINNING - VOL. I -
DESIGN AND CONSTRUCTION

FHWA-RD-75-129 VOL. II

FHWA-RD-75-130 VOL. III

GOLDBERG, DONALD T.; JAWORSKI, WALTER E.; and
GORDON, M. DANIEL

APRIL 1976

ABSTRACT:

This volume is a convenient reference on the design and construction of lateral support systems and underpinning which are often required in conjunction with cut-and-cover soft ground tunneling. The design recommendations and construction methods described herein are a summary of the more detailed information presented in the companion volumes of this study. Included in this volume are discussions of displacements, lateral earth pressure, ground water, passive resistance, stability analysis, bearing capacity, soldier piles, steel sheeting, diaphragm walls, bracing, tiebacks, underpinning, grouting, and freezing. An overview compares the relative costs of the construction methods used in lateral support systems and underpinning.

VOL. II - DESIGN FUNDAMENTALS

ABSTRACT:

This volume provides current information and design guidelines on cut-and-cover tunneling for practicing engineers. The main emphasis is on the geotechnical aspects of engineering. Included in this volume is a state-of-the-art summary of displacements and lateral pressure. Other topics are basic concepts of soil mechanics, ground water in open cut, passive resistance, design aspects of lateral earth pressure, stability analysis of sheeted excavations, bearing capacity of deep foundations, and construction monitoring. Detailed explanations of design methods and literature citations are included.

VOL. III - CONSTRUCTIONS METHODS

ABSTRACT:

This volume provides specific design recommendations, design considerations, and construction techniques for the construction of lateral support systems and underpinning. The design considerations are presented for each technique or method (soldier piles, steel sheeting, diaphragm walls, internal bracing, tiebacks, underpinning, grouting, and freezing). The factors affecting the design or implementation of these schemes are discussed. Construction techniques are presented, and literature references are provided for those seeking even greater detail. An overview of the construction methods compares the applicability of the techniques and the construction costs of each.

4. FHWA-RD-75-131 "CONCEPTS FOR IMPROVED LATERAL SUPPORT SYSTEMS"

GOLDBERG, DONALD T.; JAWORSKI, WALTER E.;
AND GORDON, M. DANIEL

APRIL 1976

ABSTRACT:

Recommends improved designs for further research. Emphasis is upon developing procedures to understand and control displacements adjacent to deep excavations. Concepts include both analytical procedures and construction techniques.

Suggested methods to improve performance are (1) Use of stiff walls and pressurized wells in cohesive soil; (2) Prevention of ground loss in "running ground"; (3) Tiebacks and internal bracing combined in a hybrid support system which will provide superior performance over each system alone; and (4) Earth reinforcement concepts, both horizontal and vertical, designed to diminish the deformability of the retained soil mass.

Finally, test sections are proposed to evaluate the concepts advanced in the report.

5. FHWA-RD-76-28 "CUT-AND-COVER TUNNELING - VOL. I - CONSTRUCTION METHODS, DESIGN AND ACTIVITY VARIATIONS"

FHWA-RD-76-20 VOL. II

FHWA-RD-76-30 VOL. III

WICKHAM, G.E.; AND TIEDEMANN, H.R., MAY 1976

ABSTRACT:

This report presents the results of a study to develop an analytical method for evaluating and optimizing cut-and-cover tunneling operations. The method is based on the results of a series of multiple estimates prepared by contractor type of basic resource estimating, rather than published unit prices. Major variables are type of structure, type of ground support, type of bracing, depth of excavation, and depth of water table.

Volume 1 contains detailed descriptions of the study situations considered, the methodology to be employed, design criteria used, alternate methods of performing each construction activity, and a discussion on methods of cost analysis.

Volume 2 contains the basic production cost data used, results of the multiple estimates produced, a discussion on quantifying construction disruption, analysis of all results obtained, and comparison of traditional and under-the-roof construction. Samples of detailed estimates are included in the Appendix.

The third Volume, FHWA-RD-76-30, contains summary cost analyses for 15 different situations. They give the unit costs of all labor, equipment and material resources required to complete a project, and are typical of the 176 cost estimates prepared for this study. Computer printouts giving complete detailed cost data developed for four of the estimates have been submitted separately.

6. FHWA-RD-76-113 "PREFABRICATED STRUCTURAL MEMBERS FOR CUT-AND-COVER TUNNELS - VOL. 1 - DESIGN CONCEPTS"

FHWA-RD-76-114 "VOL. 2 - THREE CASE STUDIES

MARCH 1977

ABSTRACT:

This report explores the possibility of improving cut-and-cover tunnel construction in urban areas by the use of prefabricated structural members. Various shapes and materials are examined and methods of incorporating these shapes are described. Types of loadings required and design methods are shown.

The study concludes that the use of prefabricated members, particularly precast concrete members, is feasible and offers opportunities for significantly reducing surface disruption time. It shows construction methods for the use of precast wall members placed in fluid (slurry) trenches, and precast, prestressed members.

The use of prefabricated members for tunnel approaches and other depressed roadways is investigated in Case Study 2.

7. "APPLICATIONS OF SLURRY WALLS IN CIVIL ENGINEERING PROJECTS"
GILL, SAFDAR A. PAPER PRESENTED AT ASCE CONVENTION & EXPOSITION
CHICAGO, ILL. OCT. 1973 PREPRINT #3355

ABSTRACT:

This paper discusses some of the applications of the slurry wall system and where no other system would be economically competitive. The examples show the immense versatility of the system for good ground conditions as well as for difficult subsoils. A brief description of construction procedures and design principles is also given.

8. "BEHAVIOR OF SHALLOW FOOTINGS NEAR A DIAPHRAGM WALL"
JOHNSON, EDMOND G.; GIFFORD, DOUGLAS G.; AND HALEY, MARK X.
PAPER PRESENTED AT THE ASCE FALL CONVENTION & EXHIBIT - 1977
PREPRINT #3112

ABSTRACT:

This paper presents the pertinent geotechnical aspects of the design, construction and field monitoring of performance during construction of a deep foundation project for a major high rise building in Boston, Mass. in 1975. Of particular interest are the observations of lateral wall movements and vertical building settlements at various stages of excavation and bracing. These data are compared with the observations and trends noted by others for similar conditions, together with an assessment of the various technical components which contributed to the observed performance.

9. "DESIGN AND PERFORMANCE OF EXCAVATIONS AND TUNNELS IN SOFT CLAY:
A STATE-OF-THE-ART"

CLOUGH, G. WAYNE; AND SCHMIDT, BIRGER
PAPER PRESENTED AT 1977 INTERNATIONAL SYMPOSIUM ON
SOFT CLAYS, BANGKOK, THAILAND. TO BE PUBLISHED IN THE
BOOK, "SOFT CLAY ENGINEERING" BY ELSEVIER.

ABSTRACT:

Some clay deposits in urban areas have been written about so extensively as to become landmarks -- San Francisco Bay Mud or Boston Blue Clay, for example. The basics of our knowledge of excavations and tunnels in clay comes from Chicago Clay, where K. Terzaghi and R. B. Peck made the first fundamental geotechnical studies of performance of these problems. The prospect of more extensive construction using new techniques such as slurry wall and the slurry mole places pressure on the geotechnical engineer to improve his technology for dealing with construction in soft clays. This paper is devoted to the state-of-the-art as regards design and performance of excavations and tunnels.

10. "DIAPHRAGM WALLS & ANCHORAGES"

EDITOR: DARWENT, THELMA J.
PROCEEDINGS OF THE CONFERENCE ORGANIZED BY THE INSTITUTION OF
CIVIL ENGINEERS AND HELD IN LONDON, 18-20 SEPTEMBER 1974

ABSTRACT:

"Diaphragm Walls and Anchorages", the proceedings of a conference held by the Institution of Civil Engineers, contains 25 original papers and wide-ranging discussion reports. Contributions on diaphragm walls cover research work, design criteria, choice of construction methods, specification, recommendations for site practice, records of wall monitoring, and practical considerations affecting construction. Site problems and remedies are discussed. There are descriptions of many projects using various systems of wall construction. On the rapidly developing subject of ground anchors, comprehensive field data are presented and current

practice is summarized. Attention is given to individual anchors and groups, formation in soil and rock, and testing and behavior characteristics.

11. "FIELD MEASUREMENTS OF LATERAL EARTH PRESSURES ON A CANTILEVER
RETAINING WALL"

COYLE, H. M.; BARTOSKEWITZ, R. E.; MILBERGER, L. J.;
AND BUTLER, H. D.
PREPARED FOR PRESENTATION AT THE 53RD ANNUAL MEETING OF
THE HIGHWAY RESEARCH BOARD - JANUARY 1974

ABSTRACT:

The findings presented in this paper are developed from a research study on "Determination of Lateral Earth Pressure for Use in Retaining Wall Design". In this paper the pressure acting on a typical cantilever retaining wall is measured and compared with theoretical pressures determined by Rankine and Coulomb theory.

Terra Tec pneumatic and Geonor vibrating wire pressure cells are used to measure lateral earth pressures. Procedures used to calibrate the pressure cells are presented and discussed. Measurements of the lateral movement of the wall are made during and after backfilling. Data are presented covering a period of fourteen months. These data include graphs of earth pressure and wall movement versus time and graphs of pressure distribution versus depth. Engineering properties of the backfill materials are determined and presented. Earth pressures are computed based on the Rankine and Coulomb active case and compared with measured pressures. A significant finding is that the measured pressures near the base of the wall are higher than the active pressures. They are nearly equal to the at-rest pressures which are possible as a result of the small movements which occurred at the base of the wall.

12. "FINITE ELEMENTS IN GEOMECHANICS"

EDITOR: GUDEHUS, G. GERMANY

ABSTRACT:

This book is based on the International Symposium held at Karlsruhe in September 1975. It is devoted to the use of finite element methods in geomechanics which covers the area of rock and soil mechanics. It is concerned with the science of (macroscopic) displacements and forces in the technically relevant part of the earth's crust and deals with problems of interest to civil engineers, mining engineers, engineering geologists, etc.

13. "LATERAL STRESSES IN THE GROUND AND DESIGN OF EARTH-RETAINING STRUCTURES"
SPONSORED BY SOIL MECHANICS AND FOUNDATIONS DIVISION, ASCE,
CORNELL UNIVERSITY, NEW YORK, JUNE 1970

ABSTRACT:

This volume represents papers delivered at an ASCE Specialty Conference in 1970. Topics address subjects including:

1. Geotechnical Considerations of Lateral Stress
2. Methods of Estimating Lateral Loads and Deformations
3. Design of Earth Retaining Structures for Dynamic Loads
4. Braced Excavations
5. Lateral Pressures on Rigid Permanent Structures
6. Design, Construction, and Performance of Cellular Cofferdams

14. "A METHOD FOR PREDICTING MOVEMENTS OF CROSSLOT BRACED WALLS IN CLAY"
CLOUGH, G. WAYNE; MENA, A. I.; AND MAYU, PHILLIPE H.
PRESENTED AT THE ASCE FALL CONVENTION AND EXHIBIT,
SAN FRANCISCO, CALIFORNIA PREPRINT 2923

ABSTRACT:

This paper derives a procedure for prediction of wall and soil movements for excavations into homogeneous clay deposits supported by crosslot braced walls. A methodology is developed within the context of trends of behavior established by combining the results of field performance and finite element analyses. This dual approach is necessary in that the field data by itself provides little or no information on the effect of changes in design details. The finite element technique provides the controlled environment within which effects of variations in design details can be judged.

15. "NUMERICAL METHODS IN GEOMECHANICS"
EDITOR: DESAI, C. S.

ABSTRACT:

This volume contains 96 papers presented and discussed at the Second International Conference on Numerical Methods in Geomechanics held at Virginia Polytechnic Institute and State University, Blacksburg, Virginia, U.S.A., in June, 1976.

Sponsor: Engineering Foundation Conferences
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1. Geotechnical Engineering Division,
American Society of Civil Engineers
2. German Geotechnical Society
3. Department of Civil Engineering, Virginia
Polytechnic Institute and State University

15. "A REVIEW OF DIAPHRAGM WALLS"
INSTITUTE OF CIVIL ENGINEERS, LONDON, ENGLAND 1977

ABSTRACT:

This volume is the record of a seminar held in September 1976 to discuss "Diaphragm Walls and Anchorages". Paper topics included:

1. General analysis of available methods for designing diaphragm walls and anchored walls.
2. Current design practice applied in differing ground conditions for diaphragm walls, contiguous piles and anchorages.
3. Practice in the detail design applications of diaphragm walls and contiguous piling.
4. Practice in the detail design applications of anchorages.
5. Design and control of bentonite/clay suspensions and concrete in diaphragm wall construction.
6. Ground anchors: installation techniques and testing procedures.
7. Slurry trench practice for diaphragm walls and cut-offs.

16. "UNDERGROUND CONSTRUCTION IN FLUID TRENCHES"
XANTHAKOS, PETROS P.
AS PRESENTED AT THE NATIONAL EDUCATION SEMINAR - APRIL 1974

ABSTRACT:

This book presents a comprehensive statement on the analysis, design, construction, supervision and inspection of structural elements in fluid trenches. The contents include a review of the State-of-the-Art; the analysis of trench stability and the supporting action of fluids; the development of fluid technology and the control of fluid systems; the presentation of excavating systems and a discussion of common organizational and construction problems; concrete fundamental and construction details; the analysis and design aspects with emphasis upon analytical modeling; applications and examples from actual construction; specifications and inspection procedures; and the need for future research.

17. "UNDERGROUND CONSTRUCTION PROBLEMS, TECHNIQUES & SOLUTIONS"

ABSTRACT:

This is an assembly of 12 papers presented at a Seminar sponsored by the Urban Mass Transportation Administration of the Department of Transportation held in Chicago, Illinois in October, 1975. Topics included:

- Review of Underground Construction in Chicago
- Site Conditions and Environmental Factors
- Contractual and Legal Aspects of Underground Construction
- Underpinning for Transportation Tunnels
- Review of Foreign Practices in Underground Construction
- Dewatering and Grouting as Supplementary Ground Engineering Techniques
- Precast Concrete Tunnel Liners
- Situations in Underground Construction Where Braced Excavations are Appropriate
- An Overview of Construction in Slurry Trenches
- Analysis, Design, and Installation of Tiebacks and Ground Anchors
- Construction Fundamentals of Diaphragm Walls - Part 1
- Construction Fundamentals of Diaphragm Walls - Part 2

18. "SLURRY WALLS FOR BART CIVIC CENTER SUBWAY STATION"

THON, J. GEORGE; AND HARLAN, RICHARD C.
JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS
SEPTEMBER 1971 PAPER NO. 2361, VOL. 97 NO. SM9

ABSTRACT:

Site conditions, laboratory tests, design, construction techniques, ground-water control, and observation and stress measurements made during construction of a subway station in a 78-ft deep excavation in a heavily developed area for the San Francisco Bay Area Rapid Transit System, are described. The excavation was supported by Soldier Pile Tremie Concrete (SPTC) walls consisting of permanent concrete walls cast-in-situ in slurry stabilized trenches and braced at three permanent and two temporary strut levels. The excavation was made in pervious sands, clayey silts, and layers of compressible peat. To minimize excessive settlements of adjacent buildings, the water table outside the excavation was controlled by recharge wells and checked by observations on piezometers. Measured pressures and inward wall deflections during and immediately after completion of the excavation are compared with predicted lateral earth pressures.

19. "COFFERDAM FOR BARTD EMBARCADERO SUBWAY STATION"

ARMENTO, WILLIAM J.

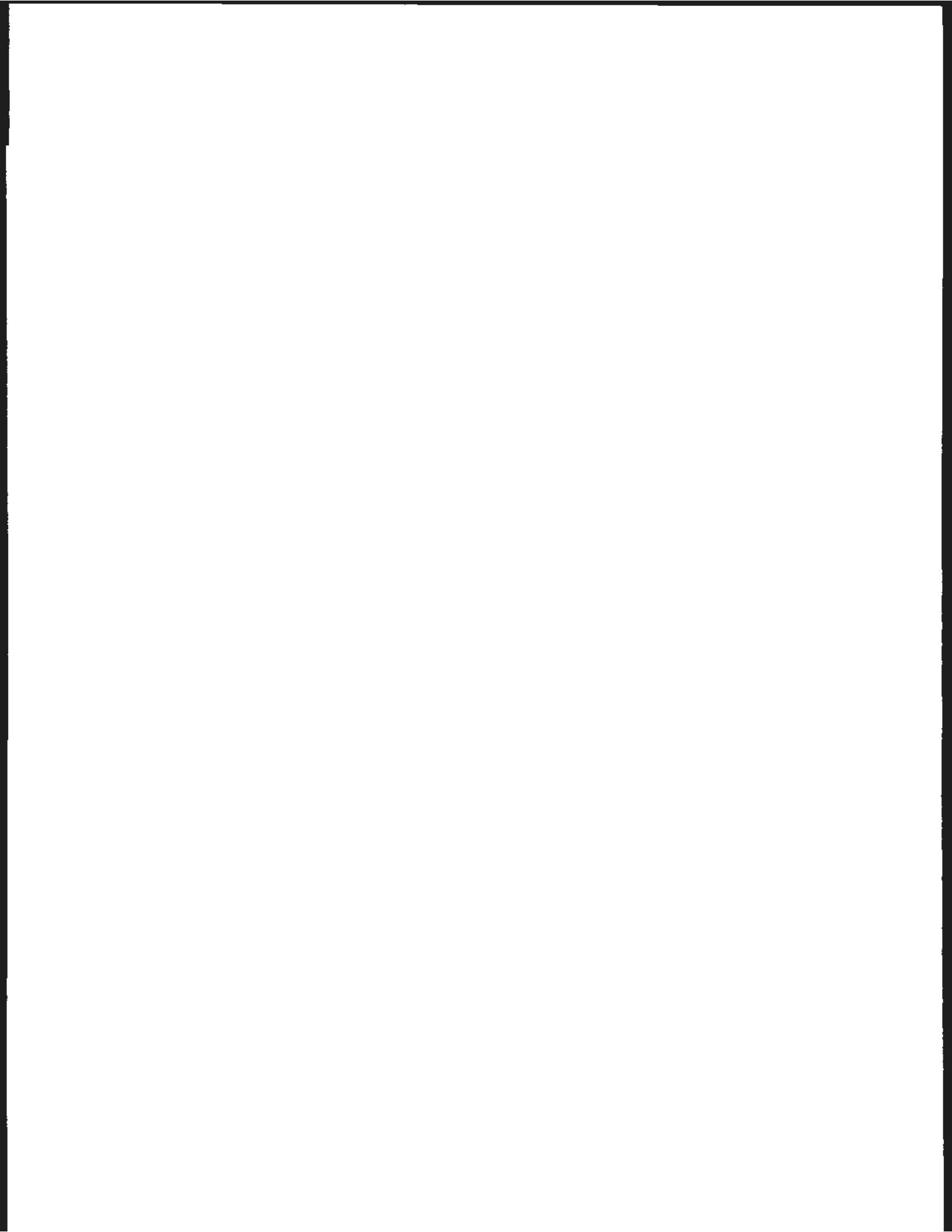
JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS
OCTOBER 1973 PAPER 10075, VOL. 99 NO. SM10

ABSTRACT:

Inward deformations of cofferdam walls during deep excavation in soft soils are inevitable but steps can be taken to minimize these movements. An excavation 1,160 ft (354m) long by 55 ft (17m) wide by 70 ft (21m) deep in a busy downtown San Francisco street, flanked by major buildings, was performed for the BARTD Embarcadero station successfully in soil strata that contained varying depths of soft clay locally known as "recent Bay mud". To provide an impervious rigid cofferdam wall of adequate strength and reasonable thickness a soldier-pile and tremie concrete (SPTC) system of sheeting was selected. Successive excavation cuts below prescribed bracing levels were held to a practical minimum. Struts were preloaded to reduce compression deformations. Inclinoimeters recorded deflections of the walls while strain gages enabled determination of strut loads. The thicker the layer of soft clay the greater the inward movements of the walls notwithstanding the corresponding increases in wall sizes.

The first part of the document discusses the importance of maintaining accurate records of all transactions. It emphasizes that every entry, no matter how small, should be recorded to ensure the integrity of the financial data. This includes not only sales and purchases but also expenses and income. The document provides a detailed list of items that should be tracked, such as inventory levels, customer orders, and supplier invoices. It also outlines the procedures for recording these transactions, including the use of specific forms and the assignment of responsibilities to different staff members. The goal is to create a clear and consistent system that allows for easy access and review of the data at any time.

The second part of the document focuses on the analysis of the recorded data. It describes various methods for identifying trends and patterns in the financial performance. This includes comparing current results with historical data and industry benchmarks. The document also discusses the importance of regular reporting and the use of visual aids, such as charts and graphs, to present the information in a clear and concise manner. The goal is to provide management with the information they need to make informed decisions about the business's future. The document concludes with a summary of the key points and a call to action for all staff members to adhere to the established procedures and maintain the highest standards of accuracy and integrity.



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