A STUDY ON APPLICATION OF CUT AND COVER METHOD FOR THE CONSTRUCTION OF METRO RAIL TUNNEL IN DHAKA CITY

BY

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The thesis titled **"A Study on Application of Cut and Cover Method for the construction of Metro Rail Tunnel in Dhaka City"** Submitted by **Kawsar Md. Abdul Waheed, Roll : 040504220(F), Session : April 2005,** has been accepted as satisfactory in partial fulfilment of the requirement for the degree of Master of Science in Civil Engineering (Geotechnical) on 24th May, 2008

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Kawsar

24th May, 2008 Kawsar Md. Abdul Waheed

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ABSTRACT

A feasibility study for a metro rail tunnel for Dhaka was performed in this thesis. A tunnel alignment along the existing surface railroad system from Tongi to Kamalapur was chosen for the feasibility study. Infrastructures close (4 to 40 m from edge) to the existing railway that may be effected due to tunnel construction were identified. Density of building close to the road line is highest between the the Basabo, Moghbazar and Mohakhali areas. However, some unauthorized structures exist close to the rail lines. These are unlikely to create problem during constructions as these may be demolished.

Geotechnical characterization of subsoil condition along the proposed tunnel alignment revealed variability of the subsoil. The subsoil was characterized on the basis of available literature. Considering the soil profile and the soil parameters, four general cases of ground condition has been chosen for analysis of earth retention system for tunnel construction to cover a wide range of ground conditions. Tunnel structure and the station building were chosen as those used for typical metro lines.

Design of the earth retention system revealed that cantilever sheet pile wall will not be suitable for the ground conditions along the rail alignment and for the depth of excavation expected for the tunnel in Dhaka. Earth retention systems consisting of sheet pile, contiguous drilled piles and diaphragm walls with lateral bracings have been designed for the proposed railway tunnel. The effect of nearby structures on the design of the retention system revealed that if the structure is located at 8 ft or beyond, the effect of the building structure on the earth retention system would be negligible.

Among the earth retaining systems, cost of diaphragm wall was found to be the minimum of the three systems i.e, sheet piling, contiguous piling and diaphragm walling. The cost of diaphragm wall for the proposed tunnel ranged from Tk. 29.6 crore to Tk. 46 crore per km. The costs of diaphragm wall is 17 to 21 percent less than those for braced-cuts, and 15 to 23 percent less than those for drilled contiguous pile system. The cost/km of tunneling using diaphragm wall system appear to be 26% less than the cost of Flyover Structure at Mohakhali.

The study revealed that metro tunnel system is feasible for Dhaka city and would appear as an effective solution for the current traffic condition. Diaphragm wall system appeared to be the most effective for construction of the tunnel.

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INTRODUCTION

1.1 General

Traffic jams have appeared as a major problem in Dhaka city over the last decade. Number of population in the city is growing at a geometric rate while the capacity of the road and road network have not been expanded to accommodate the growth. While analysis of the road system make sound decisions about traffic management, better traffic control, more roads and flyovers, more lanes in existing roads is unlikely to eliminate the existing traffic problem. Underground Metro system for mass movement in Dhaka city has never been seriously considered . Subway have been used successfully in the crowded cities of the world such as Tokyo, New York, London etc. In line with mass transit system of all great cities, a metro rail is desirable for Dhaka city to eliminate the traffic congestion and related problems. Metro rail system is believed to be one of the most superior with respect to volume of mass transport, uninterrupted operation, safety, noise pollution and the preservation of cityscape. A tunnel can be constructed by either 'Cut and Cover' method or using 'Tunnel Boring Machine' (TBM). The former is relatively cheap and require simple technology and therefore suitable for tunneling in Dhaka city. However, the method causes disturbance to land traffic and ground deformations around deep excavations which can damage adjacent buildings and utilities. Peck (1969), Goldberg et al. (1976), Mana (1978), Clough and O' Rourke (1990), and Long (2001) have identified factors influencing the ground deformations due to excavations. Stress around tunnel and stability after excavation are described in Matsumoto and Nishioka (1991). Braced-cut, anchored sheet pile, diaphragm wall, shore pile etc are the methods used for soil retention (Bickle et. al , 1997). The diaphragm wall technique was used suitably for Kolkata metro tunnel (Som, 1985). This study is to investigate different earth retention system that can be used for Dhaka soil for construction of a metro tunnel and to explore the feasibility of using metro tunnel in Dhaka.

1.2 Reasons for going underground

Underground medium is a space that can provide the setting for activities or infrastructures that are difficult, impossible, environmentally undesirable or less profitable to install above ground. It offers a natural protection to whatever is placed there. The containment created by the structures protects the surface environment from the risks / disturbances inherent in certain types of activities. Underground space also provides temperature control, noise isolation and environmentally acceptable location for unwanted but necessary facilities. The use of underground space has recently gained popularity in developed countries for civil, industrial, residential and even recreational facilities. In densely populated cities like Dhaka, the contrast between available space and the space for different functions is enormous. Highways congestion and increased daily travel time appeared to pose a threat on daily life. The growing number of vehicles generates daily traffic jams, pollutes air and creates conflicts with the pedestrians. The traffic became a real nightmare, aggravated by ineffective transportation system at the surface. Underground subway system is the only alternative that can solve the problem. Subway system also provide a balanced, coordinated, well-managed and efficient transport system, which is a precondition for the sustainable development and economic growth of Bangladesh.

1.3 Metro Rail for Dhaka City

A feasibility of using underground tunnel and metro rail for Dhaka city from geotechnical consideration has been investigated in this thesis. While the alignment of the metro rail depends on numbers of factors including road traffic, volume, connectively subsoil condition, land use strategy, economic benefits etc. An alignment along the existing surface railroad system from Tongi to Kamalapur has been chosen for the feasibility study. The reason for chosing this line is that it passes through the heart of the city which presently is active as barrier to road traffic as it is crossed at several location by road network. Also since this rail line with its right of way has significant width, any new construction work will not significantly disturb the existing traffic flow. Therefore it is proposed to take the existing surface train line to underground. Figure 1.1 shows aerial views of the existing railroad from Tongi Bridge to Kamalapur Station. Some of the reasons for chosing the metro rail along the existing rail road are :

1) Improvement of the existing railway infrastructure in Dhaka. Metro Rail will significantly increase the passenger capacity of the core section from Kamlapur to Tongi route.
2) Underground railway services to new destinations will improve public transport access in Dhaka city.
3) Shorter journey times as a result of fewer changes needed between mainline and Underground services.
4) Metro Rail can also give more services between Kamlapur and Tongi per hour at peak times and reduced overcrowding on the existing Dhaka and other District bus services.

* White line indicate existing Railroad and Blue line indicate highways

Figure 1.1 : Existing rail Route from Kamalapur to Tongi Bridge (Google Earth)

5) It will also improve dispersal of passengers from the Roads and Highway vehicle at Dhaka.
6) Finally, it will give a significant contribution to helping Dhaka's economy and be provide a catalyst for regeneration in parts of the capital.

1.4 Description of existing railroad

The existing rail track, which is the proposed site for metro rail, starts from Kamalapur railway station and extends towards north as shown in the Figure-1.1. It has two sharp bands one at Khilgaon and the other at Tejgaon, and a gentle curve at cantonment area. This length of track is running approximately from south to north and passes through the busiest portion of the city. In this typical route there exist 18 rail crossing points over the Railway line. Subway line is considered to run parallel to existing rail line within the right of way of the rail lines. Average 30 ft to 35 ft wide right of way exists on each side of two existing parallel railway lines which may be used to construct the metro line. Existing railway line is a meter gauge line that runs almost flat relative to the existing road surface except in few places, where it is slightly elevated.

1.5 Nearby Infrastructures

Nearby infrastructures are identified along the existing rail road to examine if subway construction would affect the existing facilities. The infrastructures are divided into two categories. (1) Residential Building. (2) Non-Residential Building. For safety operation during construction of Metro Tunnel, shortest distance between typical route and infrastructure and the area occupied by the infrastructures are required. During excavation, distance from excavation and the load would influence the horizontal movement and vertical settlement of the infrastructures. Table 1.1 give the details information of nearby infrastructures along the existing railroad. The information about the residential buildings and non-residential buildings are shown in Table 1.2 and 1.3, respectively. These information have been collected from the maps of Survey of Bangladesh and from google earth. The maps along the railroad and picture from google earth are included in Appendix-1. Table 1.4 gives the information of other types of objects which may effect the construction of Tunnel. It shows that water body is present as close as 2m from the track. It is revealed that the buildings are located at distances of

4m to 40 m from the rail line. Numbers of buildings per km are the highest in the Basabo, Moghbazar and Mohakhali portion of the route. Building closest to the line are located in Moghbazar and Mohakhali areas. However, some of the structures too close to the track may be unauthorized and therefore can be removed during tunnel construction. Table 1.1 shows that area occupied by the building structures varies from 9 m^2 to 2800 m^2. The structures occupying smaller areas are small shops located nearby to the track and are mostly unauthorized. The large buildings are usually located further beyond the right of the way of rail track. Figure 1.2 shows existing rail tracks with two roadway intersections.

Table 1.1 : Nearby infrastructures along the rail road

Map No	*SOB ID No	Dis tance From	Dis tance To	Length Of Rail way Line	Buildings on the sides of rail line		Dis. From Near edge of rail line to Build.		Are a of near est Buil ding.	Small est Buil ding Area	Lar gest Buil ding Area
					Left	Right	Left	Right			
				(m)			(m)	(m)	(sqm)	(sqm)	(sqm)
1	173	Saida bad	Kamala pur	3100	170	120	4	5	25	9	413
2	172	Basa bo	Mera dia	2165	142	96	10	3	30	9	750
3	152	Ram na	Mogh bazar	3450	135	145	2	1.22	20	2	2800
4	151	Tej gaon R/A	Moha khali	3045	139	132	2	4	40	12	1925
5	150	Bana ni	Army Sta diam	3500	23	23	8	10	16	16	550
6	169	Khil khet	koala	3110	Nil	21	Nil	6	140	20	140
7	148	Ash kona	Azam pur	3250	82	121	5	5	100	25	560
8	147	Coat bari	Tongi I/A	800	26	26	5	10	25	9	300

*** SOB : Survey of Bangladesh 2003**

Table 1.2 : Nearby Residential Building (From Bsabo To Airport Railway Station)

Map No	Survey of Bangladesh Map ID No	Distance From	Distance To	Length of Rail way Line	No of Building		Dis. From Near edge of rail line to Build.		Area of nearest Build
					Left Side of Rail way Line	Right Side of Rail way Line	Left Side of Rail way Line	Right Side of Rail way Line	
				(m)			(m)	(m)	(sqm)
1	172	Basabo	Meradia	1850	70	54	10	35	158
2	152	Ramna	Mogh bazar	3450	88	55	5	5	263
3	151	Tejgaon R/A	Moha khali	3045	73	69	5	4	125
4	150	Banani	Army Stadiam	3500	16	13	11	11	400
5	169	Khil khet	koala	3110	Nil	3	Nil	7	150
6	148	Ash kona	Azam pur	3250	2	Nil	20	Nil	500

Table 1.3 : Nearby Non-Residential Building (From Basabo To Airport Railway Station)

Serial No	Type of Structure	No of Structure		Area of Structure (sqm)	Nearest dis. of the Structure from the Route (m)
		Left Side	Right Side		
1	School	5	4	225	11
2	Mosque	9	11	688	5
3	Factory	3	1	700	9
4	Madrasa	1	1	282	30
5	Community center	Nil	1	306	40
6	Govt. Office	2	2	1338	10
7	Hospital	1	Nil	150	35
8	Petrol Pump	Nil	2	300	25
9	Hotel	Nil	2	338	10
10	Bank	Nil	2	350	11
11	Police Station	1	Nil	525	20

Table 1.4 : Other type of Objects (From Bsabo To Airport Railway Station)

Serial No.	Types of Objects	No. of Objects		Length (m)		Area (sqm)		Nearst Distance (m)	
		Left Side	Right Side	Left Side	Right Side	Left Side	Right Side	Left Side	Right Side
1	Lake & Pond	3	3	3150	3220	91500	199550	2	5
2	Swamp	1	2	100	1210	1000	72750	7	5
3	Embankment & Vegetation boundary & Highway	1	Nil	3110	Nil	Nil	Nil	Attached	Nil

It shows two tracks of meter gauge covering an average width of 10 m . One meter gauge lane can be continued to function for both way traffic, during construction of subway, while other track would be closed . Thus, width of the working area that would be available is about 8 to 10 m. At some places even a larger space is available that would facilitate the tunnel construction. However, the movement of train on working track may

(a) Mohakhali Rail line Crossing (b) Banani Rail line Crossing

Figure 1.2 : Actual site condition and availability of working space.

cause vibration to the soil that may destabilize the soil near the cut. The issue of the effect of soil vibration has not been considered in this thesis. The site is bounded by some underground utilities, including water pipes and telecommunication cables in the close vicinity of the work area, particularly in the city area from Kamalapur to Banani. Above ground utilities are also available at places such as high tension electricity lines, telephone cables, drainage and sewage pipes lay on both sides. The utilities may be relocated to facilitate tunnel construction.

1.6 Objective and scope

The objective of this thesis is to review construction process of a underground railway system by cut and cover method. Different earth retention systems such as anchored sheet pile, braced cut, bored pile and diaphragm wall are examined for their applicability in Dhaka soil. Construction cost associated with each of the method is also explored to develop a suitable and cost effective method for a metro tunnel in Dhaka City. Specific objectives and possible outcomes of this study are described below:

i. To review use of cut and cover methods for underground tunneling.
ii. To characterize the sub-soil conditions at typical locations within the Dhaka city.
iii. To propose a suitable underground tunnel construction method and tunnel structure for Dhaka city including suggestion for a cost effective soil retention system based on soil condition of the Dhaka city.
iv. To estimate and compare cost for different cut and cover tunneling methods.
v. Estimate cost of metro-tunnel construction using cut and cover method.
vi. Make a comparison of cost for underground tunnel and with flyover structures.
vii. To develop and understanding of the challenges that leads to be focused for tunnel construction in Bangladesh.

1.7 Organization of the thesis

The thesis is arranged in a systematic way to present the above objectives. In chapter-2, reviews of available cut and cover methods such as anchored sheet pile, braced cut, bored pile and diaphragm wall for ground tunneling are described. Chapter-3 discusses about the Geotechnical characterization of the sub-soil conditions along the proposed typical tunnel alignment. Based on the geotechnical characterization four conditions are selected for designing an appropriate earth retention systems. Chapter-4 discusses the typical metro rail systems and selection of a tunnel box and metro station for Dhaka city.

Chapter-5 discusses design of a suitable earth retention system for four conditions and design of Tunnel Box and Underground Railway Station for the Dhaka city. Chapter-6 is divided into three parts. These are : (1) estimation and comparison of cost for different earth retention methods, (2) estimation of cost of metro-tunnel structure (3) comparison of cost for underground tunnel and flyover structure. Finally, conclusion regarding suitability and challenges of applying tunneling in Bangladesh and Recommendations for future studies are discussed in Chapter-7.

CHAPTER 2

REVIEW OF CUT-AND-COVER TUNNELING

2.1 Introduction

Tunnel construction is characterized as "cut-and-cover" construction when the tunnel structure is constructed in a braced, trench-type excavation ("cut") and is subsequently backfilled ("covered") (Bickel et al, 1997). For depths up to 35-45 ft this method is often economical. The general design aspects of cut-and cover tunneling discusses the basic theoretical elements and design framework with which to approach the engineering of deep excavations. Shoring walls are used for earth retention in cut and cover tunneling methods. Steel sheet piling walls are often classified as "flexible" walls. In the same context, continuous concrete diaphragm walls, which are ordinarily much stiffer, are classified as "rigid" or "semi-rigid" walls, depending upon actual stiffness. In addition, considerable emphasis is placed on displacements of adjacent ground and adjacent structures. The issues of common shoring system and design aspects of tunneling by cut-and-cover method has been reviewed. Ground stress on the retaining structure depend on the type of lateral support used. Cantilever wall are those with no lateral support. Earth pressure on the active and passive side of the wall govern the design of the cantilever system. On the other hand, wall with lateral bracings are called braced cut. Braced shoring walls restrict the movement of the soil behind the shoring wall is almost always considerably different from that represented by active pressure, and it cannot be predicted accurately based only on theoretical soil mechanics. Earth pressure diagrams have, however, been developed for the design of shoring systems from data made available from many measurements of strut loads in excavations. These diagrams are commonly referred to as "apparent pressure diagrams." Braced cuts with sheet pile, bored pile and diaphragm wall are usually used for cut and cover construction.

2.2 Common Shoring Systems

2.2.1 Cantilever Steel Sheet Piling

Continuous steel sheet pile are composed of rolled Z-shaped or arch-shaped interlocking steel sections. Because of their greater stiffness and resistance to bending, Z-shaped sections are almost exclusively used as steel sheet piling for cut and cover constructions. Figure 2.1 shows the section of interlocking steel sheet piling. It is typically used in saturated pervious or semi pervious soils and other soils that do not permit the easy placing of lagging. It is used as well in competent sandy soils when ground water is not a concern, if there are sufficiently few utility crossing and other subsurface obstacles and it is found economically advantageous to do so. The sheet piles are driven with either hydraulic type or vibratory type hammers, depending in part on the type of soil. Vibratory hammers are most commonly used in sandy soils.

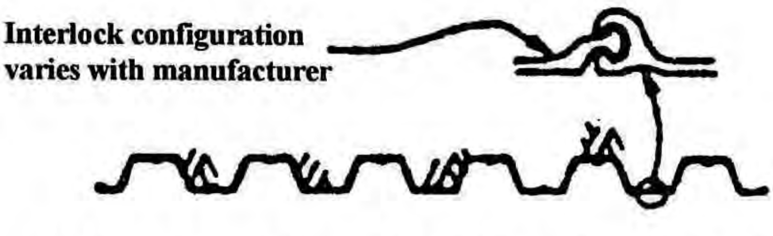


Figure 2.1 : Section interlocking steel sheet piling. (Bickel et al, 1997)

2.2.2 Sheet piling with Tie-Backs

A tie back is a form of support in which the horizontal earth pressure acting on the sheet pile Wall is resisted by an anchor assembly, which in turn deposits its load into soil or rock far enough behind the wall to have no significant effect on the wall. A tie back consists of three principal elements shown in Figure 2.2. These are : (1) an anchor zone, which acts as a

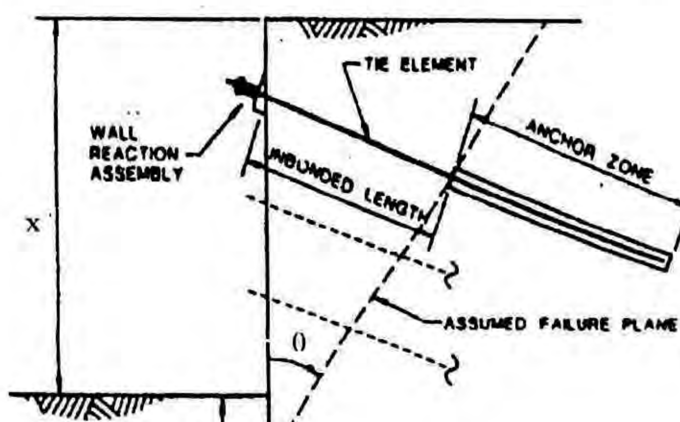
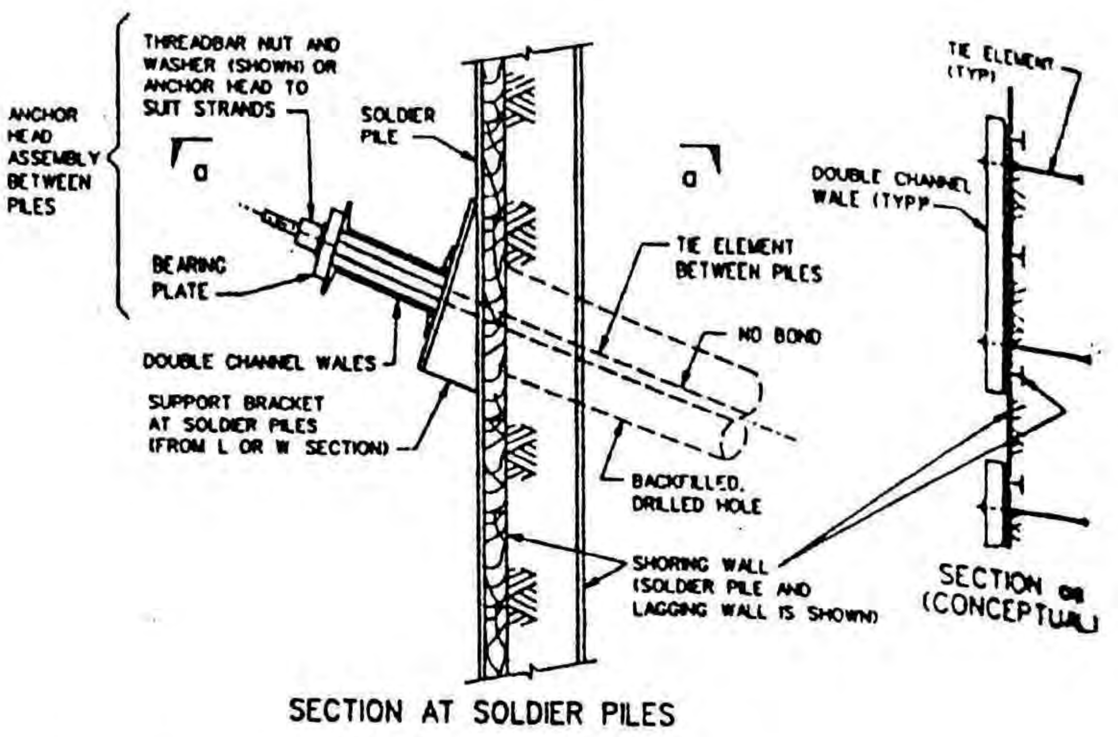


Figure 2.2 : Principal tie-back components. (Bickel et al, 1997)

reaction to horizontal earth pressure on the shoring wall ; (2) a tie element, which transfers the load from the wall to the anchor zone ; (3) a wall reaction assembly at the point of wall support. For non-cohesive soils common practice is to assume $\theta = (45^0 - \varphi/2)$. The zone between the anchor zone and the sheet pile wall is commonly referred to as the "unbonded zone" or "unbonded length". The unbonded length should be that obtained from 0 (Figure 2.3) or 15 ft minimum, which ever is larger. To locate the anchor zone behind the sheet pile wall, the anchor zone is typically placed behind an "assumed failure plane". The tie element for all of the more commonly used tie-backs is either a single thread bar 1-1-3/8 in. diameter or multiple high strength strands, each (in most cases) 0.6 in. in diameter. The tie element are installed in holes drilled from inside the excavation through the shoring wall at an inclination usually in the range of $15\text{-}30^0$ from horizontal for soil anchors. The anchored zone is created by filling the drilled hole throughout the anchor zone with sand-cement grout or neat cement grout, depending upon the type of anchor. Throughout the length the tie element is covered with a plastic tube so that none of the tie-back load is transferred into the ground. Drilled holes for the most common tie-backs range from 8 to 18 in. in diameter; about 12 in. in typical. Grout is placed in the drilled hole. By gravity or at modest pressure when hollow-stem augers are used to drill the hole. The anchor resistance is develop through grout to soil friction. For these conventional tie-backs, working capacities in soils ranging from 70 to 140 kips are common. More sophisticated tie-backs of much higher capacity are also utilized on cut-and-cover projects. In competent soils, Tie-back capacities up to 400 kips or more have been installed (Bickel et al, 1997). Figure 2.3 shows tie-back wall reaction assembly. When tie-backs are utilized as support for shoring walls, excavation proceeds in lifts that correspond to the vertical spacing of the tie-backs. Each succeeding increment of excavation cannot commence until the tie-back at that lift has been successfully post tensioned to its design working load at the wall reaction assembly. Normally, the tie-backs can not be post-tensioned until 5 days after completion of the grouting of the anchor zone. Tie-backs can be considered an alternative to internal bracing when the following conditions exist : (1) There is ample width within the excavation for tie-back installation. (2) Permission is granted by the property owner to install tie-backs in the ground adjacent to the cut. (3) There is no significant piezometric head behind the shoring wall at the level of the tie-back installation. (4) The soil behind the shoring wall is sufficiently competent to permit successful tie-back installation.(5) There are no subsurface obstacles such as deep basements beneath adjacent buildings. Although all these conditions are often present, it is uncommon to find that tie-backs are an economical alternative when the excavation is less than about 65 ft wide.



<u>NOTES</u> : WALE SPACERS OR STIFFNESS, IF ROUNDED, ARE NOT SHOWN, ROUNDED WELDING IS NOT SHOWN.

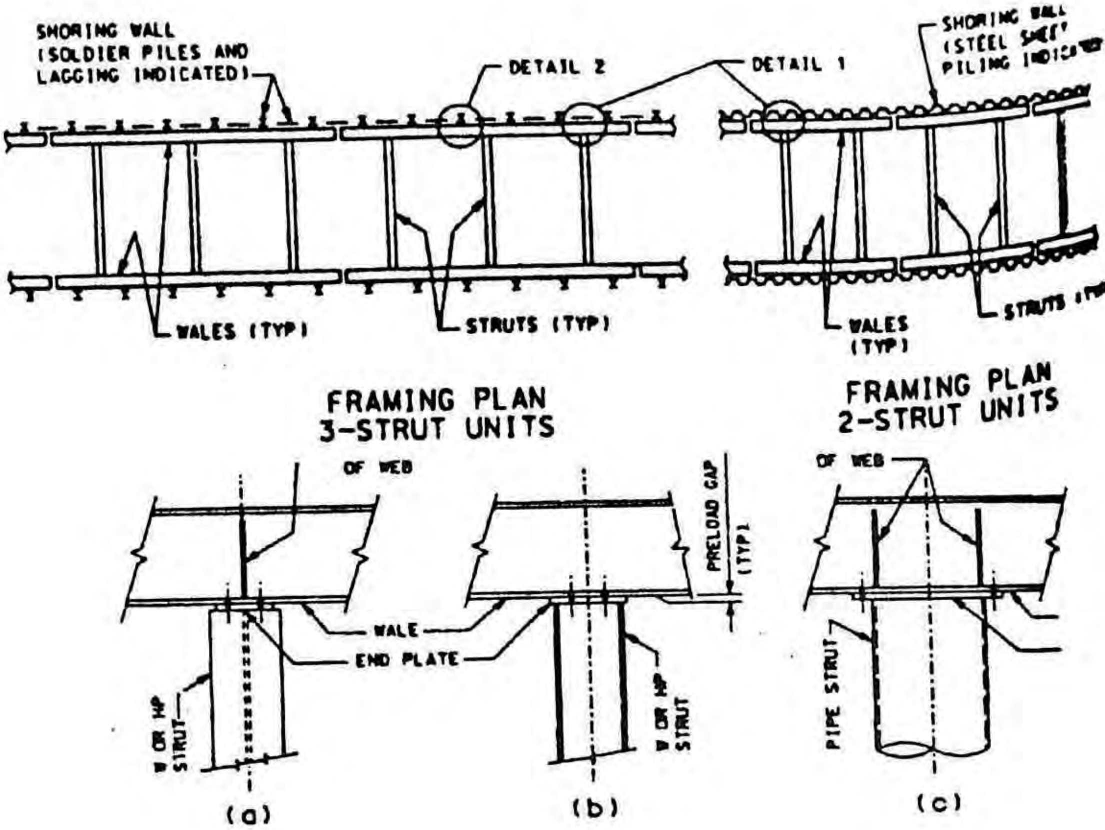
SECTION aa IS DEMONSTRATIVE AND NOT NECESSARY TYPICAL.

ARRANGEMENT SHOWN CAN BE SIMILARLY APPLIED TO ANY SHORING WALL WITH SOLDIER PILES OR TO SHEET PILE WALLS.

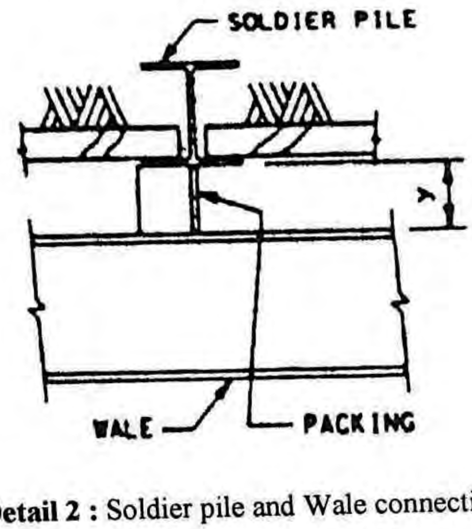
Figure 2.3 : Section showing tie-back wall reaction assembly. (Bickel et al, 1997)

2.2.3 Sheet pile with lateral bracing

Many cut-and-cover excavations are relatively narrow and, as a result, internal bracing composed of multiple tiers of horizontal, structural steel framing is the most common type of shoring wall support used. In a typical excavation the principal components of each internal bracing tier are longitudinal beams, or "wales," and transverse compression members, "or Struts," arrange generally as shown on Figure 2.4. The bracing tiers must be positioned so that they support the shoring wall and permit efficient construction of the permanent structure. Figure 2.5 shows the general sequence of construction operations typically employed during the construction of a subway line structure.

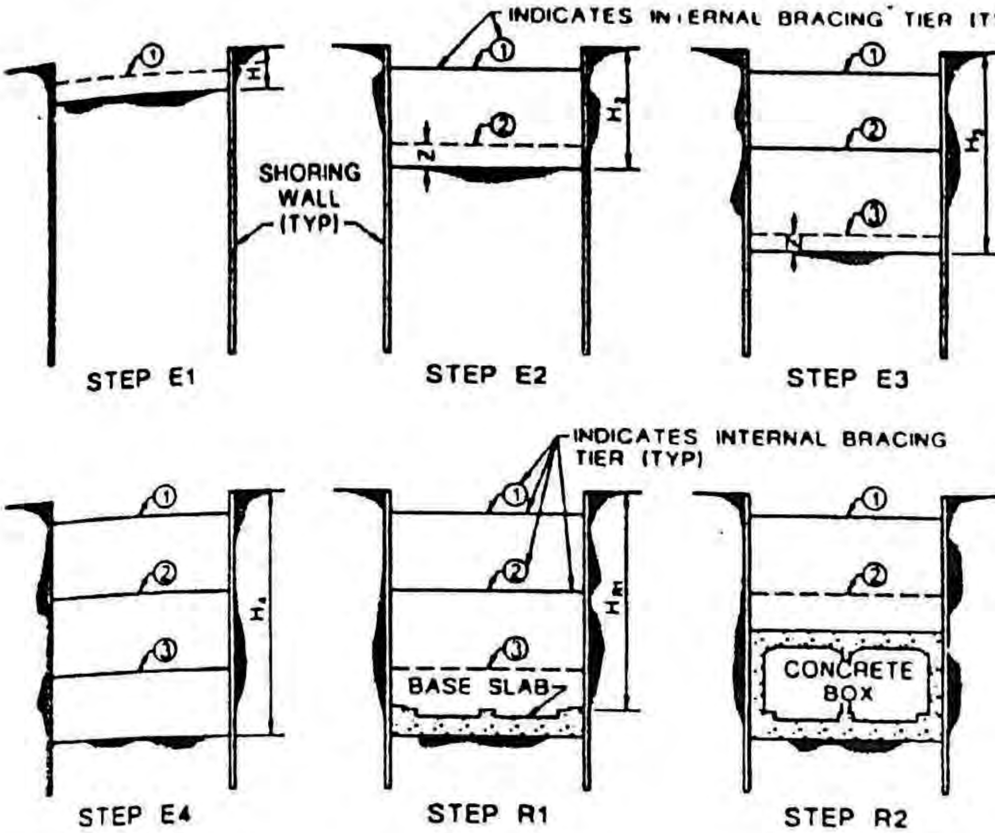


Detail 1 : Common Strut and Wale connections.



Detail 2 : Soldier pile and Wale connections.

Figure 2.4 : Internal bracing framing plans and details. (Bickel et al, 1997)



GENERAL CONSTRUCTION SEQUENCE

STEP E1 EXCAVATION TO DEPTH H_1 AND INSTALL TIER NO 1
STEP E2 EXCAVATION TO DEPTH H_2 AND INSTALL TIER NO 2
STEP E3 EXCAVATION TO DEPTH H_3 AND INSTALL TIER NO 3
STEP E4 EXCAVATION TO DEPTH H_4 (FINAL SUBGRADE)

STEP R1 : (a) PLACE CONCRETE BASE SLAB.
(b) AFTER BASE SLAB HAS AGED ADEQUATELY, REMOVE TIER NO.3

STEP R2 : (a) COMPLETE CONSTRUCTION OF CONCRETYE BOX.
(b) AFTER ROOF SLAB HAS AGED ADEQUATELY, REMOVE TIER NO.2

STEP R3 : (NOT SHOWN) BACKFILL TO DEPTH $H_{1,2}$ AND SUBSEQUENTLY REMOVE TIER NO 1. COMPLETE BACKFILL. IF SHORING WALL IS SOLDIER PILES AND LAGGING OR STEEL SHEET PILES. REMOVE (PULL) SOLDIER PILES OR SHEET PILES IF PERMITTED TO DO SO. COMPLETE SURFACE RESTORATION.

Figure 2.5 : General construction sequence for braced-cut construction. (Bickel et al, 1997)

During the excavation stage, vertical spacing of bracing tiers is often specified to be a maximum of 15-16 ft, sometimes 12 ft when it is crucial to minimize adjacent ground settlement. The maximum depth of cut in any excavation step is usually specified to be 3 ft below the centerline of the next bracing tier to be installed (dimension Z, Figure 2.5). The amount of settlement of the ground adjacent to the cut is generally considered to be largely a function of vertical spacing of bracing tiers and shoring wall stiffness. It is sometimes better to increase wall stiffness to permit larger vertical spacing of bracing tiers, when the larger spacing is needed to avoid interference of the bracing with the reinforced concrete construction. Occasionally, it is not feasible to avoid such interference, and different removal techniques or supplementary bracing methods are required. During the bracing removal stage, the shoring wall does not depend upon the soil below subgrade for support, and larger vertical shoring wall spans can ordinarily be permitted. Struts in the internal bracing framing need to be spaced far enough apart so that excavating equipment can operate efficiently. Strut spacing is usually in the range of 10-15 ft, but larger spacing (up to 25 ft) is sometimes used to permit more clearance for construction activities. However, such large spacing is often very costly because of the much heavier wales that result, and it can be undesirable as well because of the inward wall movement that accompanies the increased wale deflection. Figure 2.4 shows two framing plans representative of common internal bracing framing. Horizontal force from the shoring wall is transferred to the wale at each soldier pile, at each sheet pile web, or in the case of slurry walls, at heavy steel bearing plates embedded in the slurry wall at its inside face. The bearing plates are fastened to the reinforcing cage when the slurry wall is constructed. The shoring walls cannot be placed with sufficient accuracy to permit the wales to bear directly on the soldier piles, sheet piles, or slurry wall bearing plates. The gap between these wall elements and the wale is typically filled with a structural "packing" (Figure 2.4). The wales are supported by structural steel brackets ("lookouts") mounted on the soldier piles, sheet piles, or slurry wall bearing plates. Many framing concepts different from the typical framing shown in Figure 2.4 are employed. Irregular framing is usually required for irregularly shaped cut-and-cover excavation. Secondary framing may be required to brace the weak axis of struts in the wider excavations. When slurry wall panels can be constructed so that they are installed symmetrically about the longitudinal centerline of the cut-and-cover excavation, slurry wall panels are sometimes braced directly by struts, thus eliminating the need for wales.

2.2.4 Drilled Pile or Bored Pile Walls with lateral bracing

"Drilled pile walls" refers to walls formed by abutting cast-in-place concrete or reinforced concrete piles, concrete and soldier beams placed in drilled holes, or combinations of these concepts. Drilled holes range in diameter from about 2 to 4 ft. Depending on soil and groundwater conditions, the excavation can be made with or without casing, either in dry or slurry-stabilized holes. Cast-in-place concrete or reinforced concrete piles placed in a single line or row, tangent, nearly tangent, or slightly overlapping with each other, have been called " contiguous," "secant," or (sometimes) "tangent" pile walls. However, in recent years the configuration shown on Figure 2.6 has been utilized on several cut-and-cover projects and is probably more common (Bickel et al, 1997). With this configuration, considerable more strength can be built into the drilled pile wall.

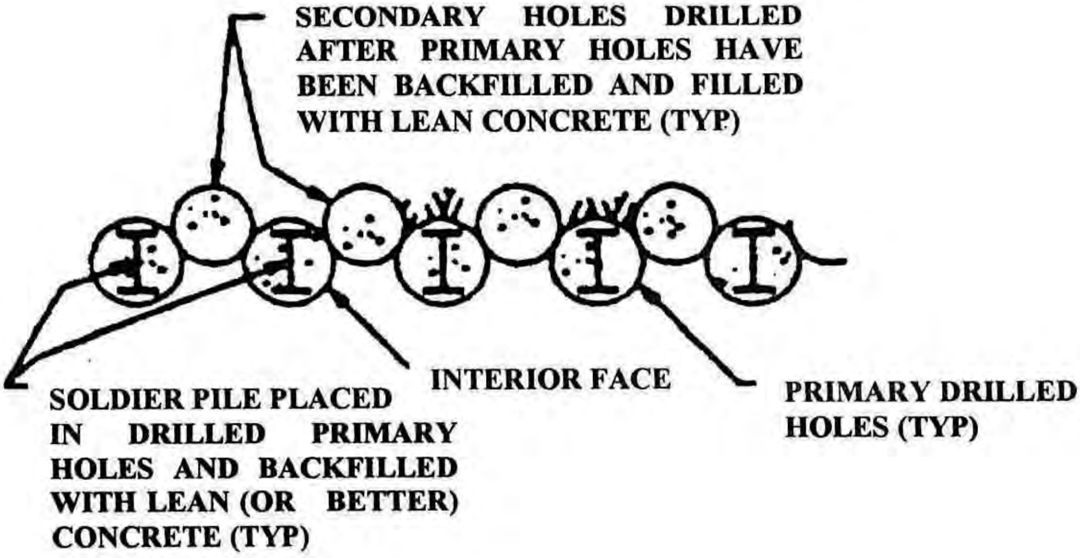


Figure 2.6 : Plan or section of Drilled Pile Wall. (Bickel et al, 1997)

2.2.5 Diaphragm Walls with lateral bracing

In shoring system design and construction, the term diaphragm wall refers to continuous shoring walls that are reinforced concrete, a combination of concrete and structural steel, or similar systems. The walls are constructed from the ground surface and are ordinarily designed so that they are, for construction purposes, watertight. The more common diaphragm walls, and their applications, are discussed briefly below.

The diaphragm wall usually refers to a reinforced concrete wall placed in a deep trench usually 2-3 ft wide. The wall is constructed in increments, or panels. Panel lengths have ranged from 7 to 20 ft, but in cut-and-cover construction they are usually 10-15 ft long. The panel is excavated with a special clamshell type bucket. The sides of the panel excavation are stabilized by filling the panel with a bentonite slurry and maintaining the level of the slurry at or near the ground surface throughout the excavation. Upon completion of the panel excavation, a preassembled steel reinforcing "cage" is lowered into the slurry-filled panel. Concrete is then placed in the panel by tremie techniques, displacing the slurry. It is important (almost always) in slurry wall construction that the joints between panels be watertight. The most common type of joint is formed with a circular end pipe. Figure 2.7 illustrates the joint configuration formed by this method.

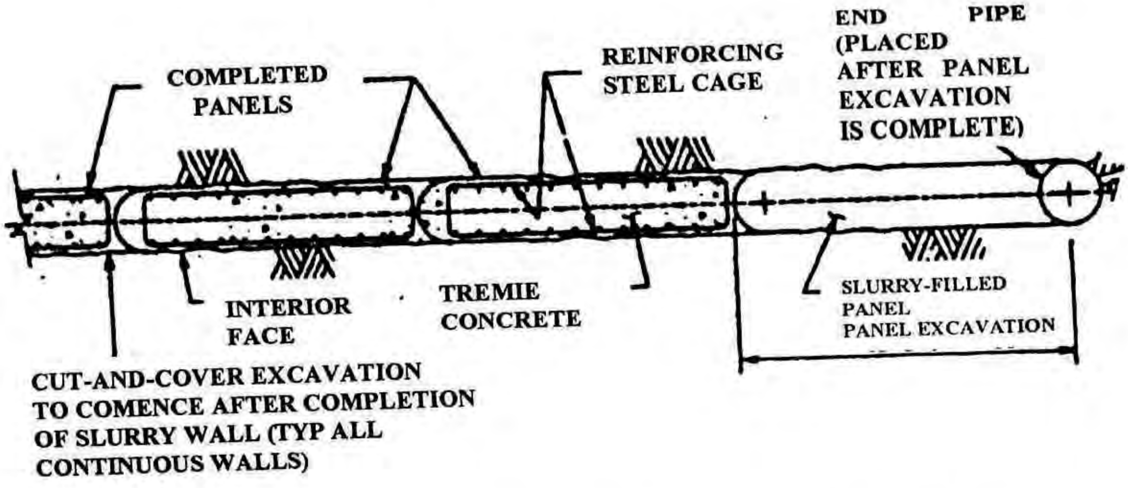


Figure 2.7 : Plan or Section of Typical Slurry Wall. (Bickel et al, 1997)

The end pipe is a steel tube inserted at one end of the excavated panel as a stop for tremie concrete. Some time after the start of the tremie concrete pour, the end pipe is rotated to break bond; it is subsequently slowly extracted to produce a formed, semicircular joint, which can be cleaned when the next panel is excavated. The procedure shown in Figure 2.7 indicates a continuous operation in which one panel at a time is constructed, setting the end pipe at the leading edge. Another procedure is to construct alternate "primary" panels, setting end pipes at both ends. The resulting "secondary" panels between primary panels are then constructed. There are many variations to the more common procedures described. Slurry walls can be constructed in soil to depths exceeding 180 ft. For cut-and-cover construction, slurry walls deeper than about 100 ft are not common, (Bickel et al, 1997). Diaphragm Walls are used generally where it is required that the shoring wall be water tight and where, at the same time, more wall stiffness or resistance to bending is needed than can be provided by heavy steel sheet pile sections. Diaphragm Walls have been constructed in virtually all soil types, but usually in very soft to medium clays, saturated silts, or saturated, loose silty or clayey sand. They are usually constructed where surface settlement adjacent to the cut must be minimized. The stiffer of the diaphragm walls have been specified in many urban cut-and-cover projects to obviate the need for under pinning adjacent buildings. The construction of diaphragm walls cause much less noise and vibration than does the driving of sheet piles, and a diaphragm wall may be chosen over a sheet pile in some cases on this account alone. Where a diaphragm wall is to be used, the choice of the type will depend primarily on the required stiffness and resistance to bending and shear, actual subsurface conditions, and cost.

2.3 Shoring Wall Deformation

2.3.1 Cantilever Walls

Sheet pile wall can be rigid and flexible depending on the stiffness of the wall and my deform differently during excavation. Figure 2.8 shows the possible range of deformations for perfectly rigid walls and for walls displaying flexure. Basically the range of behavior includes translation and either rotation about the base or rotation about the top. In addition, wall deformation will include some bulging as a result of flexure the amount of bulging depending upon the stiffness of the wall support system.

2.3.2 Braced Cut

The upper portion of internally braced walls is restrained from undergoing large horizontal movement especially when braces are pre stressed and are installed at or close to the surface. This produces the typical deformation mode as shown in Figure 2.9. The degree of rotation will depend upon the toe restraint below the bottom of the excavation.

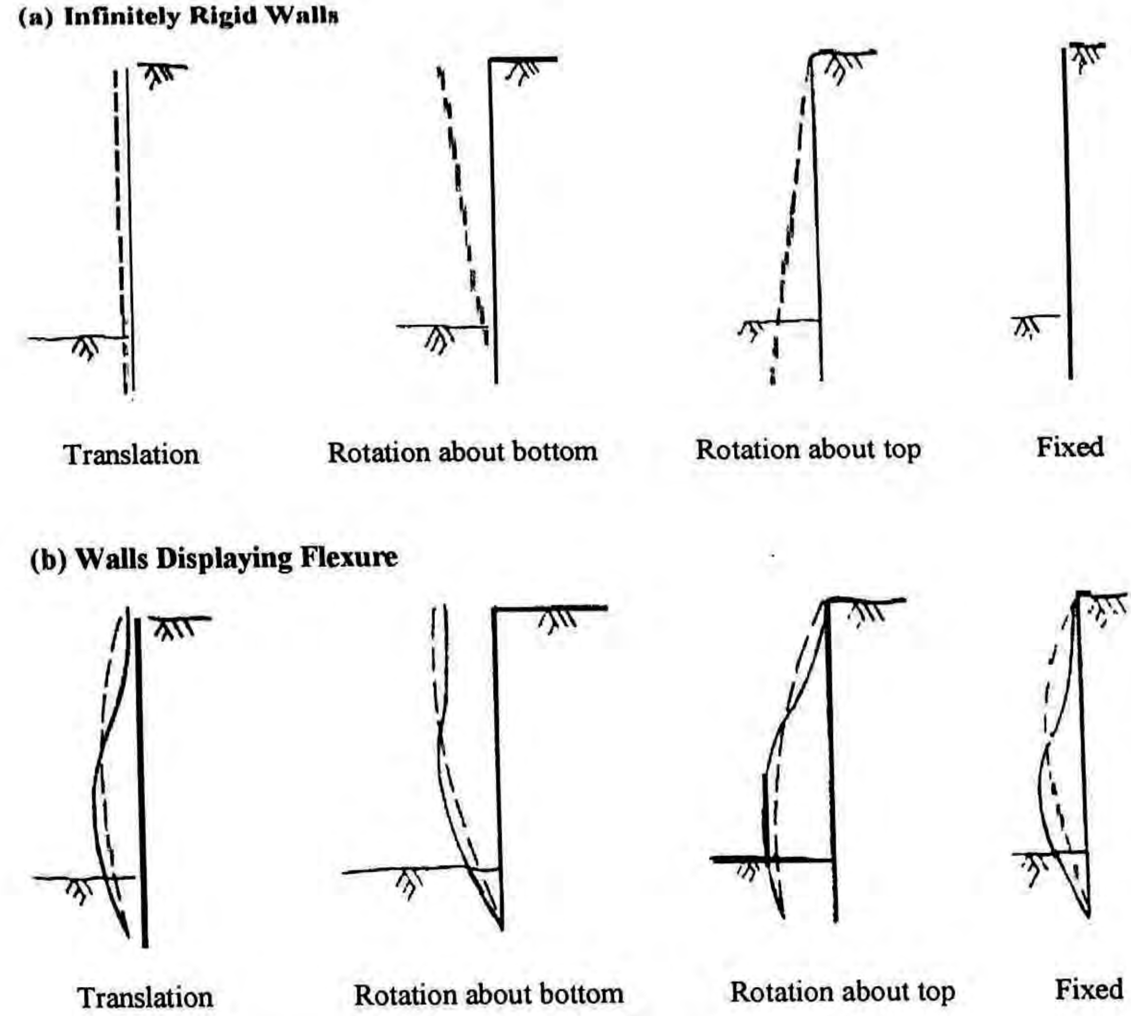


Figure 2.8 : General deformation modes. (Goldberg et. al, 1976)

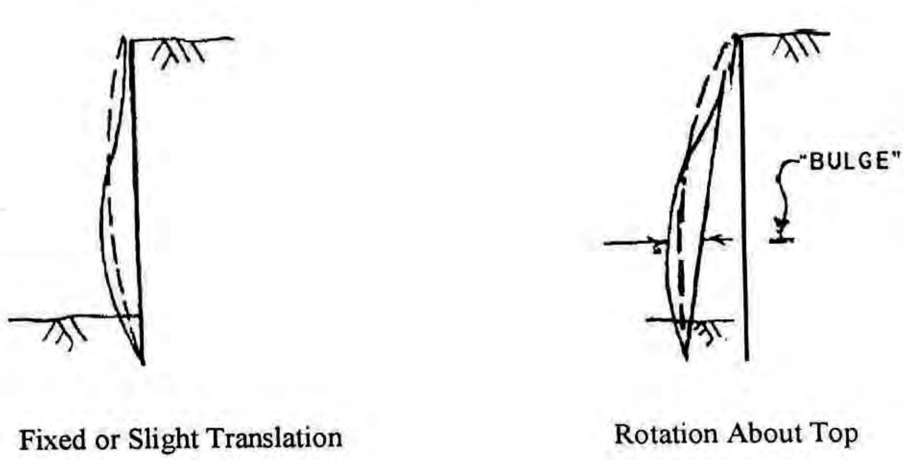


Figure 2.9 : Typical deformation of Braced-cuts. (Goldberg et. al, 1976)

Fixed or Slight Translation Rotation about bottom

Figure 2.10 : Typical deformation of tied- back (Goldberg et. al, 1976)

2.3.3 Tied-Back Walls

If the top of the tied-back wall remains fixed, then the deformation mode is similar to that of an internally braced wall (Figure 2.10). On the other hand, settlement of the wall, partial yielding of the ties, gross movement of the soil mass, or shear deformation of the soil mass may result in inward movement of the top and rotation about the bottom as shown in Figure 2.10. Nendza and Klein (1974) attributed the deformation mode of Figure 2.10, to a combination of shear deformation, which contributed to inward movement of the top, and flexure, which contributed to the bulging effect. If the soil mass embodied by the tiebacks deforms somewhat as a unit, the pattern would be similar to that shown in Figure 2.11. Here, the top moves inward toward the excavation and the earth mass mobilizes internal shear. Such a deformation mode is not true for all situations, but is very likely in cases of an unyielding base and with the bottom of the wall restrained from outward movement. Overall, the deformation mode of a tied-back wall is complex in that various factors develop in different ways.

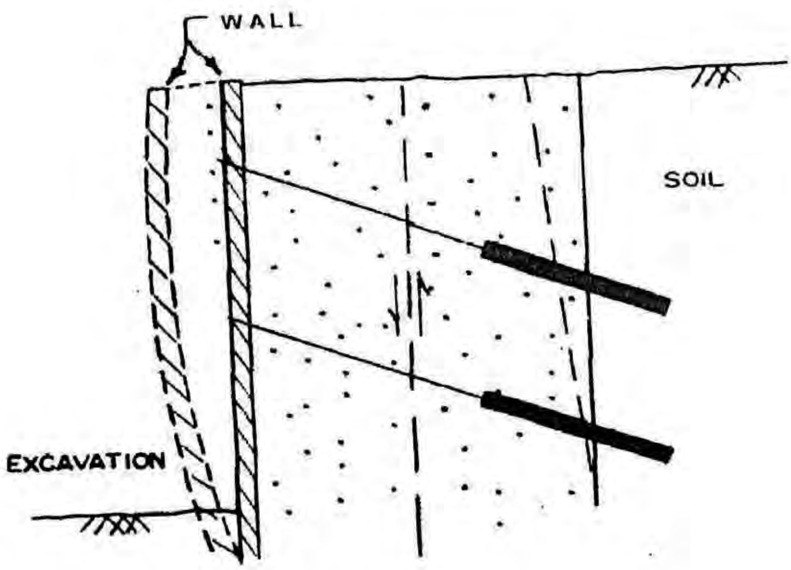


Figure 2.11 : Internal shear development, horizontal shift at top relative to bottom. (Goldberg et. al, 1976)

2.4 Construction effects

2.4.1 Effects on excavation

It is well known that construction procedures can have a significant effect on the performance of excavations. Lowering of the ground water level either by pumping or by seepage into the excavation can result in significant settlements. These settlements could be associated with consolidation of the soil or, in the case of granular soils, the piping of soil into the excavation. Poor installation techniques for tiebacks or struts can lead to surface settlements. Tiebacks should be carefully drilled to minimize the soil removed from holes. Also, any voids remaining after the tieback is installed should be filled with grout. Struts, rakers, and wales should be tightly wedged and preloaded to prevent movement of the wall. In addition, hard wood or steel wedges should be used for shimming to reduce movements caused by crushing. Earth beams when used to provide temporary support before installing a strut have been observed to be of little value in preventing wall movement. Cole and Burland (1972) and Hansbo, Hofmann, and Mosesson (1973) report cases where earth beams did little to restrict wall movement. Even though the entire support system may be in place, the sides of the excavation may continue to creep inward with time. This problem appears to be particularly acute in tied-back walls in very stiff to hard clays. There is also some evidence to indicate that lagging in soldier pile walls tends to pick up more load with time in all soils. Excessive bulging or even failure of some lagging has been observed.

2.4.2 Effects on nearby structures

The displacement occurring in the structures and soil mass adjacent to an excavation will have an effect on the choice of wall and the remedial or preventive measures required to protect the structures . The greater the amount of movement, the greater will be the protective measures required. In sands and gravels and very stiff to hard clays, the wall type does not cause deformation on nearby structures significantly. However, in softer cohesive soils the stiffer support systems (concrete diaphragm walls) need to be used to limit movements to a much greater extent than the more flexible soldier pile or steel sheet pile walls. If displacements can be reliably predicted, particularly with distance behind excavations, the effect that the movements will have on structures can be evaluated. While the magnitude of settlement is a useful indicator of potential damage to structures, the amount of settlement change with horizontal distance (settlement profile) is actually of greater significance. This fundamental concept is related to the concept of differential settlement, as opposed to gross settlement. Peck (1969) provided a normalized plot of ground settlement and the influence zone for braced-cut as shown in Figure 2.12. The figure shows the influence zone can be as far as 2 times the depth of excavation depending upon type of soil. Long-term settlement as high as 7 in (180 mm) was observe at a distance of 22 ft (6.5 m) from the face of the wall. Som (1985) observed the influence zone up to 2 times the excavation depth and the maximum settlement adjacent to the not exceeding 1% of the depth of cut for Kolkata Metro Tunnel where the soil condition is similar to Dhaka soil at some places (Figure 2.13). Figure 2.14 shows the building settlement due to Braced-Cut at Kolkata (Som, 1985). Some buildings showed small and major damages in the form of cracks in walls and floors, while some structures suffered moderate to sever damages during construction of Kolkata Metro tunnel. Thus, it appears that settlement of nearby building structure can be one of the major challenges for construction of metro tunnel in Dhaka. Detail analysis of the settlement of the nearby structure was not within the scope of this thesis.

2.5 Tunnel Structures

2.5.1 Subway Line Structures

In cut-and-cover construction between stations, the subway tracks are usually enclosed in a reinforced concrete double box structure with a supporting center wall or beam with columns

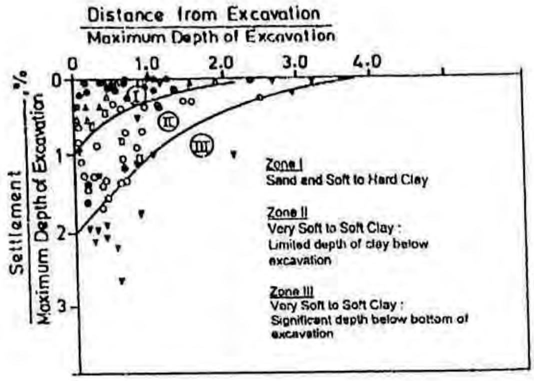


Figure 2.12 : Ground settlement for braced-cut for sheet pile support. (Peck,1969)

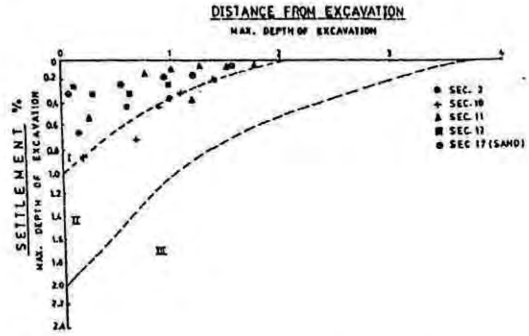


Figure 2.13 : Variation of Ground Settlement at different Test Sections in Kolkata (Som,1969)

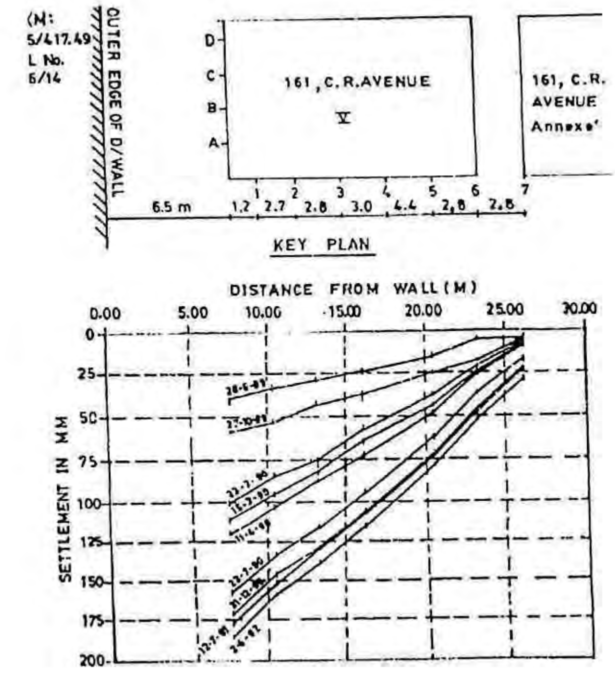


Figure 2.14 : Building Settlement due to Braced-Cut (Som,1985)

(Biclel et al, 1997). These tunnel structures are commonly referred to as "line" structures. The track centers are normally located as close together as possible. In a typical double box section, each train way will have a clear width of about 14-15 ft, depending upon width of vehicle and clearances to be provided for equipment and many ways. The number of opening depends on the number of tracks or lanes. Transit or Railway tunnels are usually ventilated by the piston action of the train, and except for fire protection they do not require air ducts. In Kolkata most of the subway box for the Metro has been built by cut-and-cover construction. The method essentially consists of putting two rows of vertical walls in the soil and suitably propping them against each other by steel struts as the excavation is made when the final excavation level is reached the subway box is cust and backfilling done to restore the original ground surface as the struts are progressively removed. The walls are normally 10m apart in the running sections and 19m at station sections. Where the cut is made wider to accommodate the platform and other services, the depth of excavation was varied from 12m to 15m (Som, 1985). A further discussion about tunnel structure is given in Chapter 4.

2.5.2 Subway Stations

Station structures include the trainway for trains, boarding and off-boarding platforms, stairs, escalators, fare collection areas and service rooms. If the line structure is two circular tunnels constructed by tunneling methods, the station may be designed with a single centre platform or side platform. If the line structure is a cut-and-cover double box subway line structure, the station is usually of the side platform type, except at terminals where center platform are normally used to commonly with system standards (Bickel et al, 1997). Detail discussion of Tunnel Structures is also given in Chapter 4.

2.6 Structural Design

The tunnel structure must be designed to have structural capacity sufficient to resist safely all loads and influences that may be expected over the life of the structure. The principal loads to be resisted are ordinarily the long-term development of water and earth pressures, dead load including the weight of the earth cover, surface surcharge lo, and live load. The load usually considered in design codes (i.e, AASHTO) are : (1) Dead Load (DL), (2) Live Load (LL), (3) Impact (I),(4) Centrifugal Force (CF), (5) Rolling Force (RF), (6) Longitudinal Braking and Tractive Force (LF), (6) Horizontal Earth Pressure (E), (7) Buoyancy (B), (8) Flood (FL), (9) Shrinkage Force (S), (10) Thermal Force (T). In locations where there is a potential for significant seismic activity, earthquake forces (EQ) must be added.

2.6.1 Dead Load

The dead load to be considered for the design of cut-and-cover structures normally consists of the weight of the basic structure, the weight of secondary elements permanently supported by the structure, and the weight of the earth cover supported by the roof of the structure and acting as a simple gravity load. The design unit weight for the earth cover should not be taken as less than 120 pcf for dry fill or less than 130 pcf for moist fill.

2.6.2 Live Load

Live loads may include pedestrian loading as well as subway vehicle loads. However, in a tunnel of one-story high, pedestrian live load, subway vehicle live load, impact load, and other dynamic loads are ordinarily transmitted through the invert slab directly to the supporting ground and, as a consequence, have little or no effect on the proportioning of the structural elements of the tunnel. These loads will normally affect the basic structural design only if the tunnel is two or more stories in height. Cut-and-cover subway structures must also be designed to support surface traffic loading or other live loading. For structures below a depth of 8 ft, a uniform live load of 300 psf is commonly used for this purpose. For structures having less than 8 ft of earth cover, common practice is to design the roof of the subway structure for the more severe of the following two conditions: (1) Actual depth of cover plus superimposed HS 20-44 wheel load distribution is accordance with AASHTO requirements. (2) An assumed future cover of 8 ft plus a uniform live load of 300 psf (Bickel et al, 1997).

2.6.3 Horizontal Earth Pressure

Horizontal earth pressure may be considered to be lateral pressure due to both retained soil and retained water in soil when water is present. Horizontal earth pressure may include the effect of surcharge loading resulting from adjacent building foundation loading, surface traffic loading, or other surface live loading. All of these components of earth pressure must be evaluated both in terms of present conditions and future conditions. Where future changes could adversely affect the subsurface structure, needed protective

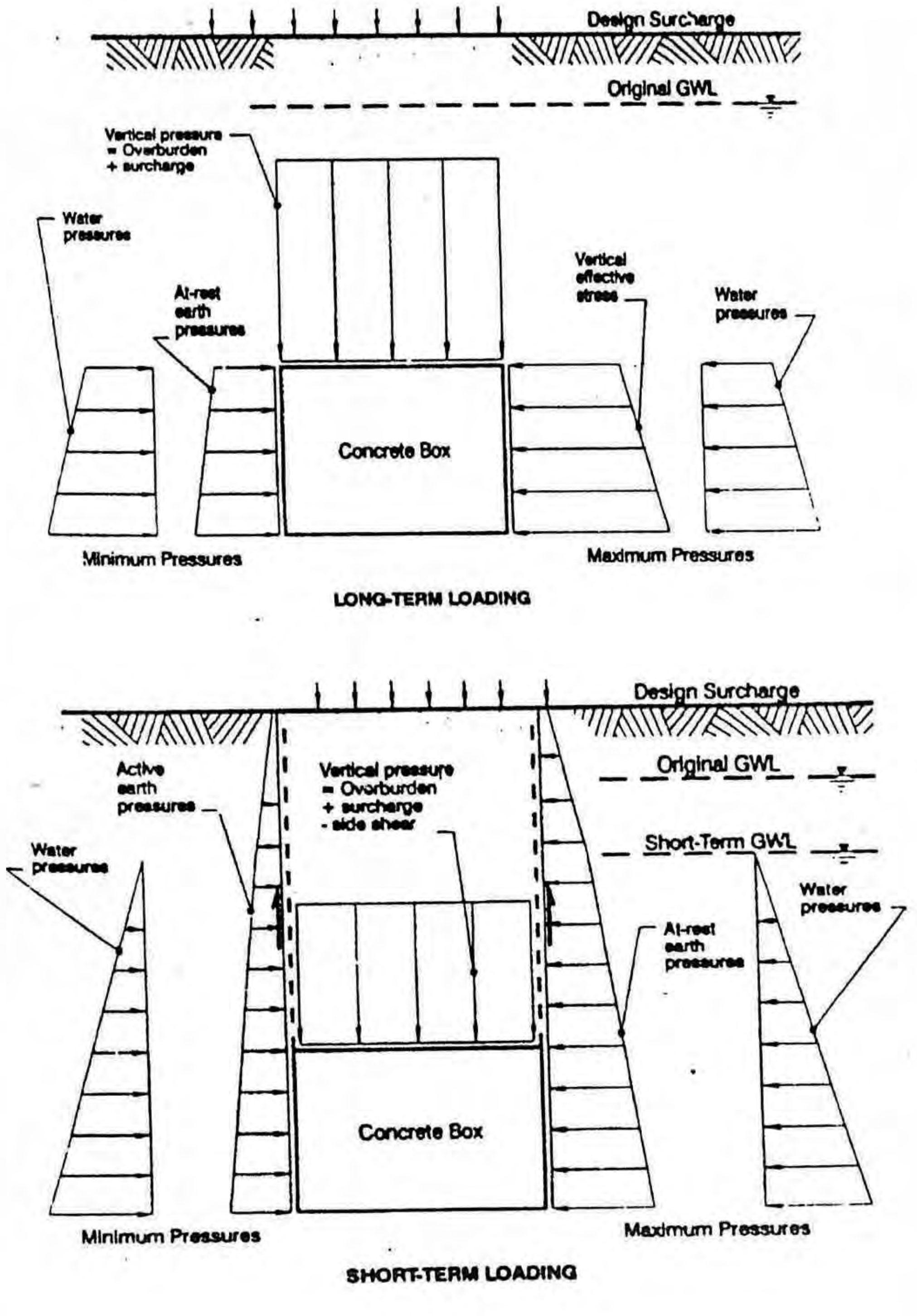


Figure 2.15 : Loadings diagrams for the design of concrete box structure (Bickel et al, 1997)

measures to mitigate the adverse effects might not be foreseen and might be extremely costly to add to an existing structure. Figure 2.15 shows typical earth pressures on a concrete tunnel box.

2.6.4 Other loads

(1) Buoyancy : When the groundwater table lies above the bottom of the invert or base slab of a subsurface structure, an upward pressure on the bottom of the base slab, equal to the piezometric head at that level, must be accounted for. For a rectangular box, this upward pressure multiplied by the width of the base slab is the buoyant force (B) per lineal ft of structure. When the reliable minimum weight of the structure plus the fill above the structure (DL min.) exceeds B by an adequate factor of safety (FS), the structure is considered stable against uplift due to B.

(2) Flood (FL) : Where there is a potential for river floods or other flooding that could add loads to subsurface structures, the design of the structures should allow for this loading as required by the particular type of structure and the conditions affecting each location.

(3) Shrinkage and Thermal Forces : Between transverse joints in cut-and-cover tunnel structures constructed of reinforced concrete, shrinkage forces and thermal forces are accounted for by the longitudinal reinforcement in the walls, roof, and invert slab. The stresses produced by these forces are typically normal to the principal stresses caused by DL, LL, and I and, therefore, do not enter into frame analysis of the structure.

(4) Earthquake Forces : Major codes that address the seismic design of surface structures in the United States contain no provisions for underground structures. The general view is that underground structures are much less affected by seismic motion than are surface structures. Although this view is substantiated by limited observations of the performance of underground structures during seismic events, some severe damage has been reported. In areas identified as subject to significant seismic activity, it is therefore necessary to determine the extent to which earthquake forces (EQ) should be considered in the design. In making this determination, the importance of the structure, the consequences of damage due to EQ, the type of soil in which

the structure is founded and the potential for liquefaction, and the location of potentially active faults at or in the vicinity of the site should all be considered.

2.7 Loading Cases

Cut-and-cover tunnel structures retain earth but are not free to yield significantly. Nevertheless, there is a need to consider unbalanced and other a typical loading on these structures. The particular cases to be analyzed will depend on the type of structure, its location, the type of ground in which the structure is founded, the location of the ground water table, and local factors. All reasonable foreseeable temporary and permanent loading cases that would affect the design of the structure should be investigated. The system design criteria developed by Washington Metropolitan Area Transit Authority for reinforced box and station section specify that, as a minimum, the following three basic loading cases be investigated : Case I : Full vertical and Long-term horizontal load, Case II : Full vertical load, Long-term horizontal load on one side, and Short-term horizontal load on other side, Case III : Full vertical load with short-term horizontal load neglecting ground water pressure on both sides.

CHAPTER 3

GEOTECHNICAL CHARACTERIZATION

3.1 Introduction

Geotechnical characterization is required to develop generalized soil profiles, to investigate the variation of the soil properties with depth, and to establish approximate correlation among different geotechnical properties of the sub-soil. Soil types and properties generally dominate underground construction and the required earth retention systems. For the purpose of metro construction the soil in the top 30 m (100 ft) is of immediate relevance (Som,1985). The geotechnical characterization presented in this chapter focused on characterization of a soil profile along a Metro tunnel alignment assumed for Dhaka City. Construction of a metro tunnel has been assumed along the existing rail road from Tongi to Kamalapur.

Bashar (2000) characterized subsoil of Dhaka through collecting about 300 sub-soil investigation reports consisting of data of 674 boreholes from different drilling companies, civil consulting firms and other organizations of different places of Dhaka Metropolitan area and through conducting soil tests. Majority of the borings were drilled upto depth of 50 ft to 60 ft and a few borings were drilled upto 100 ft. Dhaka city map, bounded by longitude $90^{\circ}20'$ to $90^{\circ}27'$ E and $23^{\circ}41'$ to $23^{\circ}53'$ N, has been divided into grids by four longitudinal grid lines along north-south direction and five cross grid lines along East-West direction. These grid lines were spaced at 2 minutes interval. All borehole site location points were then inserted on the Dhaka city map to develop the soil profile. Serajuddin et al. (2001) also conducted a systematic characterization of Dhaka City, dividing the city into ten different zones. Data of Bashar (2000) and Serajuddin et. al (2001) were used in this research to characterize the subsoil for construction of metro tunnel in Dhaka.

3.2 Geological and Geotechnical Condition of Dhaka City

The subsoil of Dhaka generally consists of Madhupur clay formation of Pleistocene age overlain by a fine to coarse grained Dupi Tila sand formation of Pliocene age. High and low land alluviums of recent age cover the formation at some places. The alluvium deposit consists of stream deposits, natural levee and back slope deposits, swamp deposits and inter-stream deposits. Table 3.1 shows the strartigraphic succession of subsoil in and around Dhaka city (after Alam, 1988).

Dhaka Terrace, constituting of the southern portion of the Uplifted Pleistocene Madhupur Tract, generally covers the whole area of the city. At the surface across the uplifted track is the Madhupur clay with thickness varying from 20 ft to over 100 ft. The Madhupur clay surface is over 730,000 years old and consists of tough overconsolidated redish brown to grey silty clay (Ahmed et al. 1999). The elevation of the Dhaka terrace ranges from about 6m to 12m (20 to 40 ft) above surrounding floodplain. At the margins of the Madhupur Tract are the recent flood plain deposits of the river Turag, Buriganga, Balu and Tongi. The Madhupur Clay overlies fine to coarse grained micaceous, quartzo-fledspathic sands of the Dupi Tila Formation. These sands are approximately over 300 to 400 ft thick in Dhaka. At the top of the Dupi Tila Formation are fine silty sands which grade downwards into fine/medium grained sands and medium/coarse grained sands with gravels toward the base.

Table 3.1: Stratigraphic succession in and around Dhaka city (After, Alam, 1988)

Age	Formation	Lithology	Thickness
Holocene	Alluvium	Low land alluvium consisting of river bed deposit of medium to fine sand and silty sand, High land alluvium consisting of silt, sand and clay	0 – 40 ft
Pleistocene	Madhupur Clay	Red slightly to heavily overconsolidated plastic clay with silt or fine sand	20 ft to over 100 ft
Pliocene	Dupi Tila sand	Medimum to dense yellowish brown sand or sandstone	Over 300 ft

Karim et. al (1993) divided the city into three geomorphological units as: I) the central high area, II) complex high and low areas, and III) complex low area, as shown in Figure 3.1. The central high area comprises of Madhupur Clay and is covered by indigenous surface soils in places. The area covers from extreme part of Tongi to the south part of

Dhaka city. Toward the east and to the west at some places of the central area, a complex landform of high and low areas have developed. It is characterized by saddles of Madhupur clay alternating with gully type depressions that have been filled at many places. The ground condition of the complex high and low area is medium stiff red clay, uncompacted red soil, soft organic clay and municipal waste fill. Depth of the upper complex soil condition ranges from 6 ft to 16 ft (Karim et al., 1993). Around outskirt of Dhaka-Tongi area, the river Buriganga, Turag, and Balu drain a complex of low lying alluvial plains. This complex is formed of natural levees, point bars, sand bars, abandoned channels, other floodplain deposits and other depressions. The soil in this area consists of organic clay-silt, silt and fine sand. Figure 3.1 shows that the existing railway line from Tongi to Kamalapur station generally lies on the central high area (Area I) consisting of Madhupur formation, except a small part on the South-East. Part of the track on South-East (Kamalapur and Khilgaon area) fall in the complex high and low area (Area II).

Table 3.2: Index properties of Dhaka Clay (After Uddin, 1990)

Properties	Range of values
Liquid Limit	39-50
Plastic Limit	18-25
Plasticity Index	18-29
Clay content	15%-35%
Sand content	0%-11%
Silt content	35 %-85%
Water content	17%-37%
Activity	0.6 to 2.0
Group symbol according USCS	CL and CH

The Madhupur clay of Pleistocene age in the central area and other area is slightly to heavily overconsolidated clay with overconsolidation ratio of 2.7 to 3.0. Consistency of this Dhaka clay appeared to vary from soft to very stiff with the undrained shear strength ranging from 260 psf to 4000 psf (12.5 to 200 kPa), Bashar (2000). The soil is active clay showing expansive behavior when come in contact with water. Table 3.2 shows the typical index properties of the Dhaka clay, after Uddin (1990). Depth of the clay layer appeared to vary form 20 ft to over 100 ft. This reveals that a metro tunnel construction along the existing rail line would pass through the clay and underlying sand stratum in the central high area. However, the excavation on the South-East would face a complex ground of high and low land at shallow depths (6 ft to 16 ft). Detail geotechnical characterization of Dhaka Clay along the existing rail road is discussed further in the following sections.

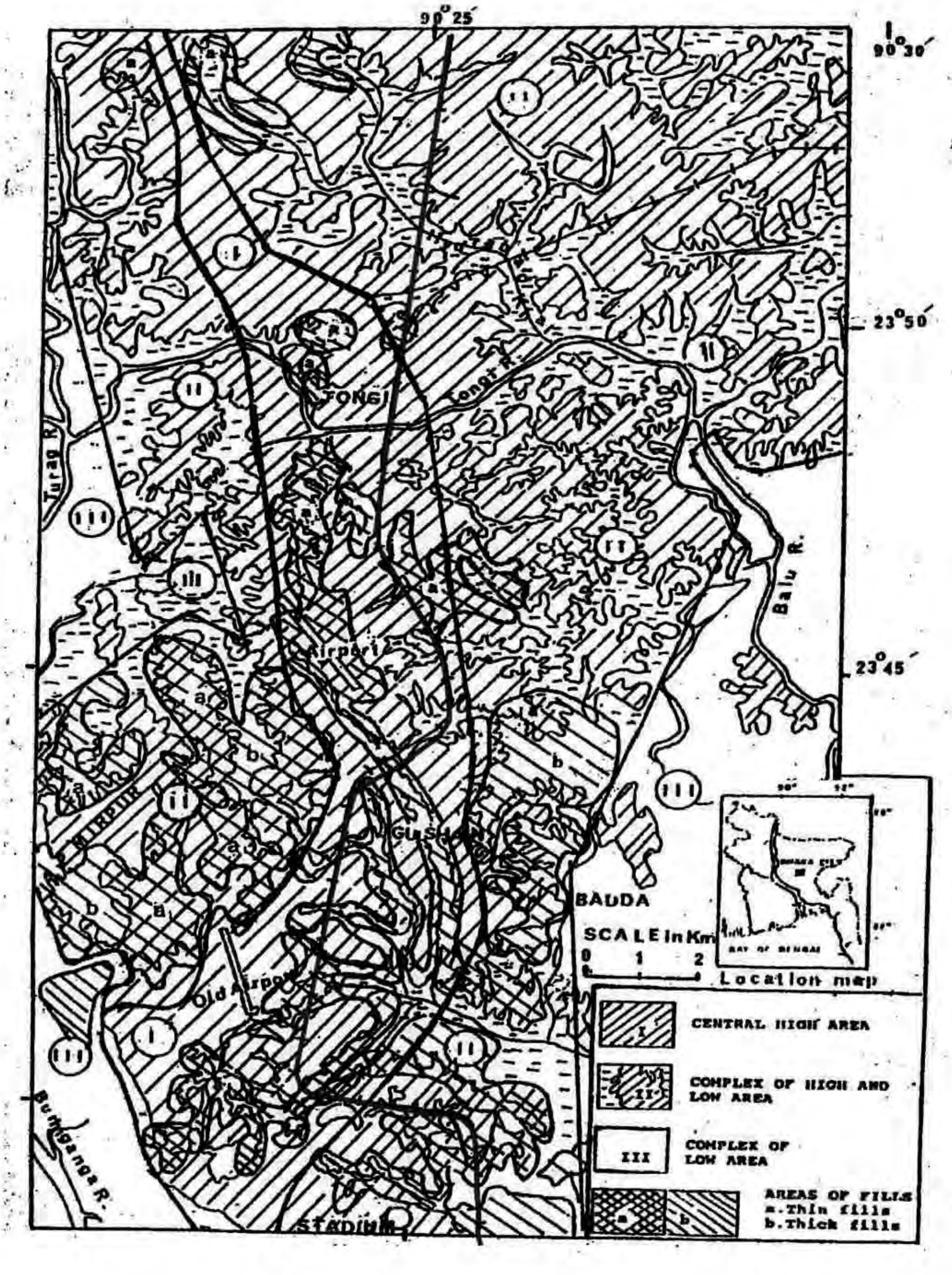


Figure 3.1: Geomorphological units of Dhaka (Karim et al., 1993)

3.3 Geotechnical Characterization by Serajuddin et al (2001)

Serajuddin et al. (2001) conducted a systematically study for geotechnical characterization of the subsoil of Dhaka city and summarized the results including those from many geotechnical investigations carried out for small to large scale public and private construction projects in the city over last three decades. Results of about 249 undisturbed and 3500 disturbed soil samples from 203 boreholes of varying depths in different areas of the city were analyzed and presented in Serajuddin et al. (2001). Ten heavily populated zones within the city where major infrastructures developed over the last few decades, were considered for the geotechnical characterization. Figure 3.2 shows the 10 zones considered starting from North (Uttara Model Town) upto South part of the city (Jatrabari). Areas covering in each zone are summarized in Table 3.3. Figure 3.3 shows the subsoil profiles from the boreholes of different zones. The average N-values from Standard Penetration Tests (SPTs) for every 3 m depth of sandy soils strata and the range of the average unconfined compression strength, q_u of the undisturbed cohesive samples taken from the different sites are shown along the sides of the typical borehole in Figure 3.3. Unified Soil Classification System (USCS) symbols have been used to describe the soil in the stratum. Strata depths are shown with respect to the existing ground level (EGL). It is evident from the soil profile (Figure 3.3) that a deep sand layer is present in the region underlying the upper clay and/or silt layer of various thicknesses as expected from the geological profile discussed earlier. The thickness of the clay and silt layer appears to vary from 5 m to greater than 20 m (16 ft to 66 ft) from the boreholes in Figure 3.3.

The thickness of the clay layer is almost uniform (varied between 5 m and 8m) on the south, but increases toward the north (Area 1). The soil in the upper clay layer varied from light brown, reddish brown to red in color and medium to very stiff in consistency. N-values in the clay layer varied from 5 to 20. Table 3.4 shows the geotechnical properties of the upper cohesive layer for each of the zone considered. The underlying sand layer is predominantly medium to dense sand (N values varied from 10 to above 50), except in the northern zone (Zone 1) where N values are sometime less than 10. Unconfined compression strength of the clay varied from 7 psi (50 kPa) to 69 psi (250 kPa), resulting in the undrained shear strength between 25 kPa (520 psf) and 125 kPa (2600 psf).

Table 3.3 : Areas Covering different zones of Dhaka City

Zone No.	Areas included
Zone 1	Uttara Model Town Area
Zone 2	Banani Gulshan Model towns & Mahakhali Commertial Area
Zone 3	Tejgaon Commercial Area
Zone 4	Mohammadpur, Lalmatia and Dhanmondi Residential Areas
Zone 5	Sher-e-Bangla Nagar Area
Zone 6	Eskaton, Mogbazar & Minto Road
Zone 7	Nilkhet, Palashi & Chankharpul Areas
Zone 8	Segun Bagicha, Purana Paltan, Topkhana Areas
Zone 9	Motijheel & Dilkusha Commercial Area
Zone 10	Jatrabari Area

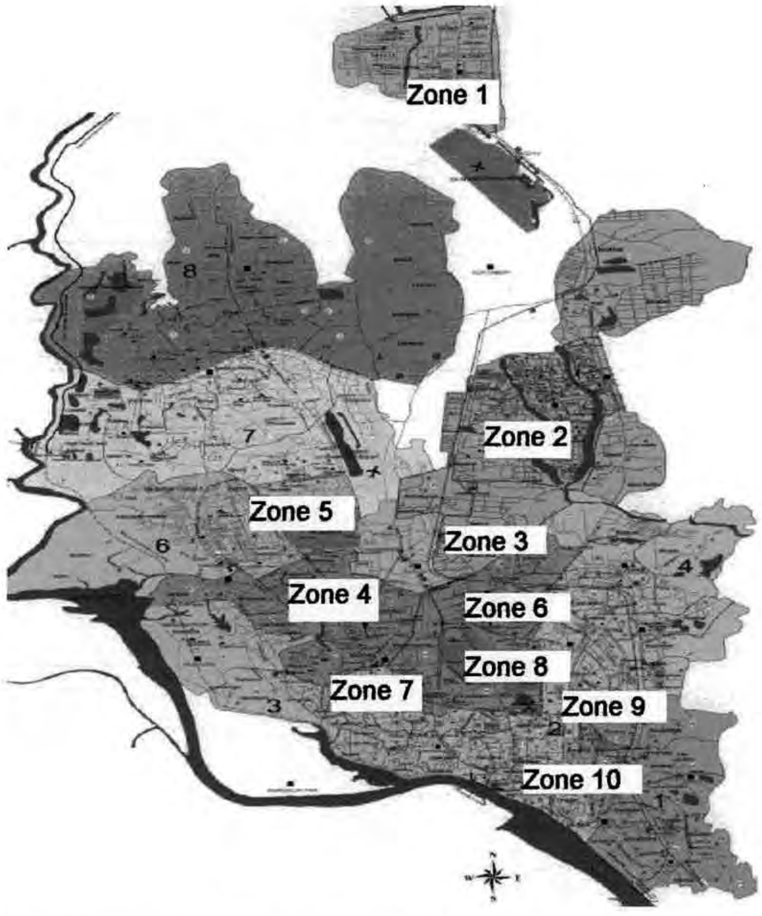


Figure 3.2 : Map showing different zones of Dhaka City (after Serajuddin et al. 2001)

Figure 3.3 indicates that depths of clay stratum can vary significantly even in an area. For example, Zone 1 (Uttara model town) indicated by soil profiles 1 to 5, shows that high plastic clay stratum can vary from depths of 5m (16 ft) (profile 4) to as deep as greater than 20m (66 ft) (profile 5). Profile 2 does not show any evidence of the clay layer, but start with a plastic silt layer. Thus, a site-specific geotechnical investigation would be required for implementation of any engineering project in any of the areas. However, a general conclusion can be drawn as the upper 5m (16 ft) to 8m (26 ft) of Zone 1 contain high plastic material (plastic silt to highly plastic clay) for qualitative assessment of soil profile within the region. A similar soil pattern was obtained in the other zones (zones 2 to 10), with various depths of the cohesive layer.

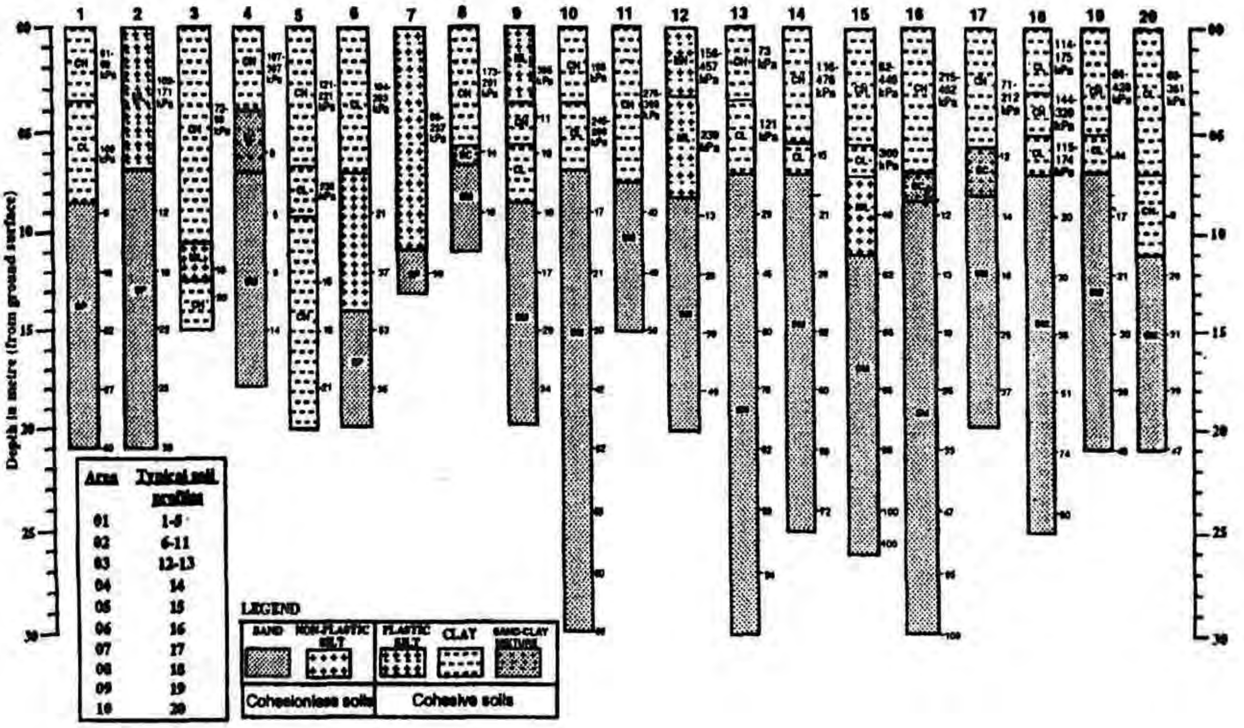


Figure 3.3 : Some typical borehole profiles of Dhaka City (after, Serajuddin et al. 2001)

Table 3.4 : Overview of the soil properties of different regions, After Serajuddin at al. (2001)

Areas	Particle finer than (%) 0.075 (mm)	0.002 (mm)	Water content W_N (%)	LL	PL	Gs	q_u (psi)	e_o	OCR
Zone 1	65-99	14-33	20.4-33.7	30-73	8-41	2.67-2.71	10.44-53.18	0.626-0.884	1.18-4.71
Zone 2	61-99	11-40	19.5-30	37-62	12-36	2.67-2.72	9.85-57.87	0.542-1.013	1.36-4.32
Zone 3	65-99	11-40	18.7-38.5	34-63	10-33	2.67-2.76	6.18-66.25	0.603-0.845	1.38-4.60
Zone 4	61-99	11-30	18.2-29.5	36-63	12-35	2.67-2.70	16.81-69.04	0.657-0.864	1.47-4.40
Zone 5	89-100	15-23	16.8-25.2	52-65	25-35	2.68-2.7	11.92-64.97	0.596-0.700	2.14-4.18
Zone 6	84-99	13-24	18.0-29.2	34-59	12-31	2.68-2.75	11.81-67.01	0.652-0.746	1.45-2.90
Zone 7	80-100	14-29	19.2-26.3	51-66	21-35	2.67-2.71	10.28-42.86	0.572-0.740	1.84-3.89
Zone 8	61-93	13-25	20.5-25.6	38-65	18-31	2.67-2.73	14.23-47.72	0.529-0.786	1.59-4.71
Zone 9	78-91	10-17	22.0-28.6	38-54	14-26	2.67-2.71	20.3-62.07	0.579-0.652	1.96-2.93
Zone 10	77-90	20-22	21.4-34.2	46-60	20-28	2.69	8.63-52.34	0.573-0.588	1.55-2.07

3.4 Geotechnical Characterization by Bashar (2000)

Bashar (2000) collected geotechnical data and conducted field tests to characterize subsoil of Dhaka city. The city was divided into grids as shown in Figure 3.4. In this study, four grid lines along North-South direction and five grid lines along East-West direction have been considered. Thus, in both directions, a total of 9 grid lines have been taken. The grids in the North –South direction and East-west direction were spaced at 2 minutes interval. The longitude and latitude of the nine gridlines are as follows: (i) Gridline A-A' : 90°21'E (ii) Gridline B-B' : 90°23'E (iii) Gridline C-C' : 90°25'E (iv) Gridline D-D' : 90°27'E (v) Gridline 1-1' : 23°43'N (vi) Gridline 2-2' : 23°45'N (vii) Gridline 3-3' : 23°47'N (viii) Gridline 4-4' : 23°49'N and (ix) Gridline 5-5' : 23°51'N. In North-South direction, four grids marked as A-A', B-B', C-C', D-D' and in East-west direction, there are five grid lines, designated as 1-1', 2-2', 3-3', 4-4' and 5-5'. There are altogether 20 grid points. Each grid covered a width of about 3.68 km. Thus each grid point covered an area of about 3.68 km x 3.68 km (i.e., 13.54 km^2). The co-ordinate and location of points are presented in Table 3.5. Soil data were sorted on the basis of applicable study area grid line and locations. Boreholes within the vicinity to the relevant grid were picked for the grid points. In order to characterize the grid section, the representative data were identified by visual inspection. Data are then used for plotting of soil profile and contour maps. As expected from the geological features of Dhaka, a clay layer with variable thickness was observed from the boreholes, which was underlain by a sand formation. Figure 3.5 shows the depth of clay layers from the existing ground level. It is revealed from the figure that the depth of the clay layer is generally 20 to 30 ft, with localized variations at some places. The topography of Dhaka city is shown in Figure 3.6. It can be seen from Figure 3.6 that the elevations on Dhaka city vary from level 5.850m PWD to 7.790 m PWD. Elevation of surrounding areas varies from 2.940m PWD to 5.850 PWD. Apart from some depression areas, pond channel or low land, the ground level of Dhaka metropolitan area can be taken as almost at the same elevation.

Soil investigation data of Bashar (2000) has been reviewed with a goal to develop a soil profile along the assumed tunnel alignment. As mentioned earlier, the tunnel alignment is assumed along the existing rail road from Tongi to Kamalapur. A geological profile is a graphical representation of underground condition along a given typical line on the ground surface. The arrangement of various soil layers can be best shown in the form of

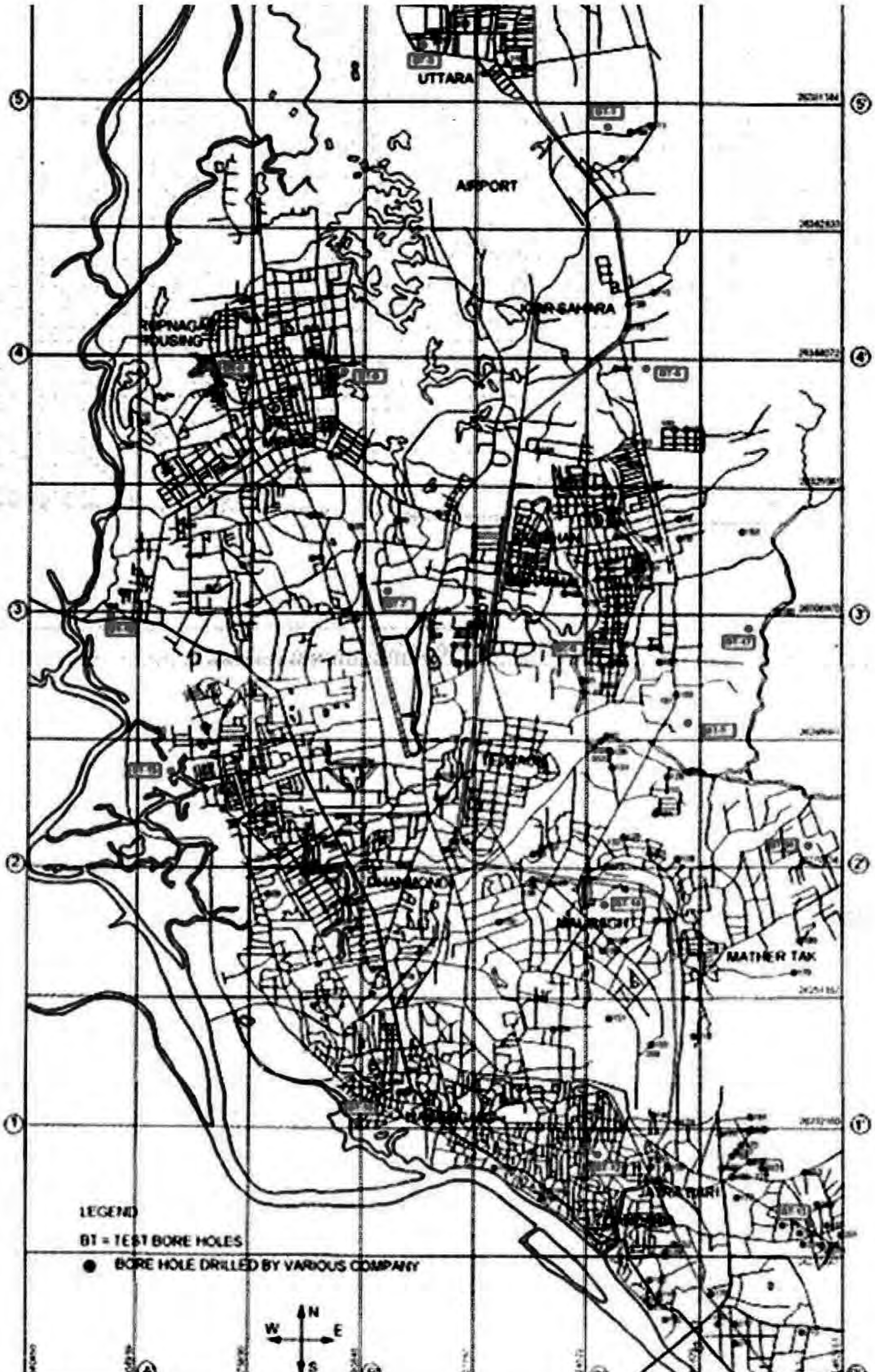


Figure 3.4 : Map of Dhaka city showing Borehole locations (after Bashar, 2000)

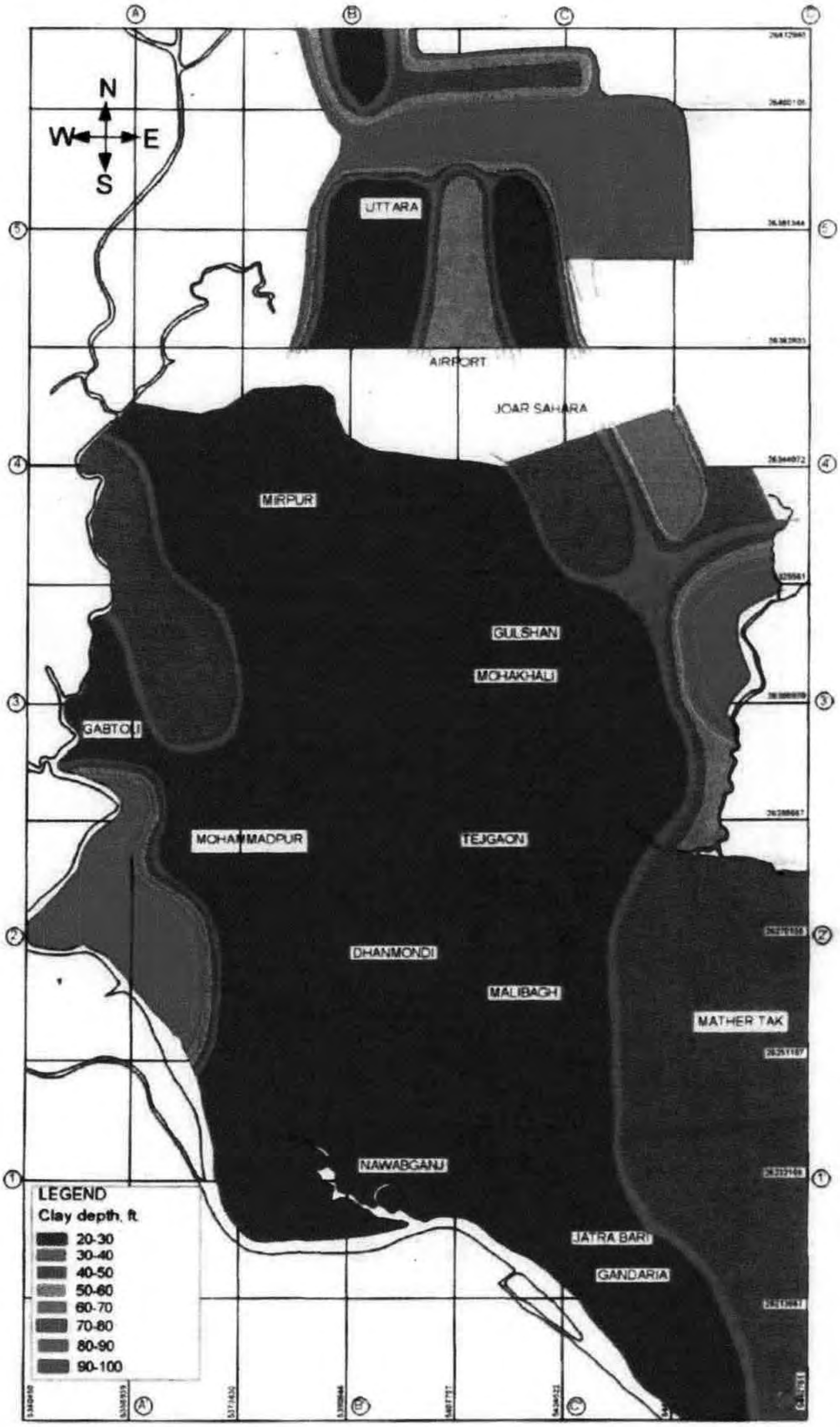


Figure 3.5 : **Grid Point Locations and Depth of Clay Layers from existing ground level**
(After Bashar, 2000)

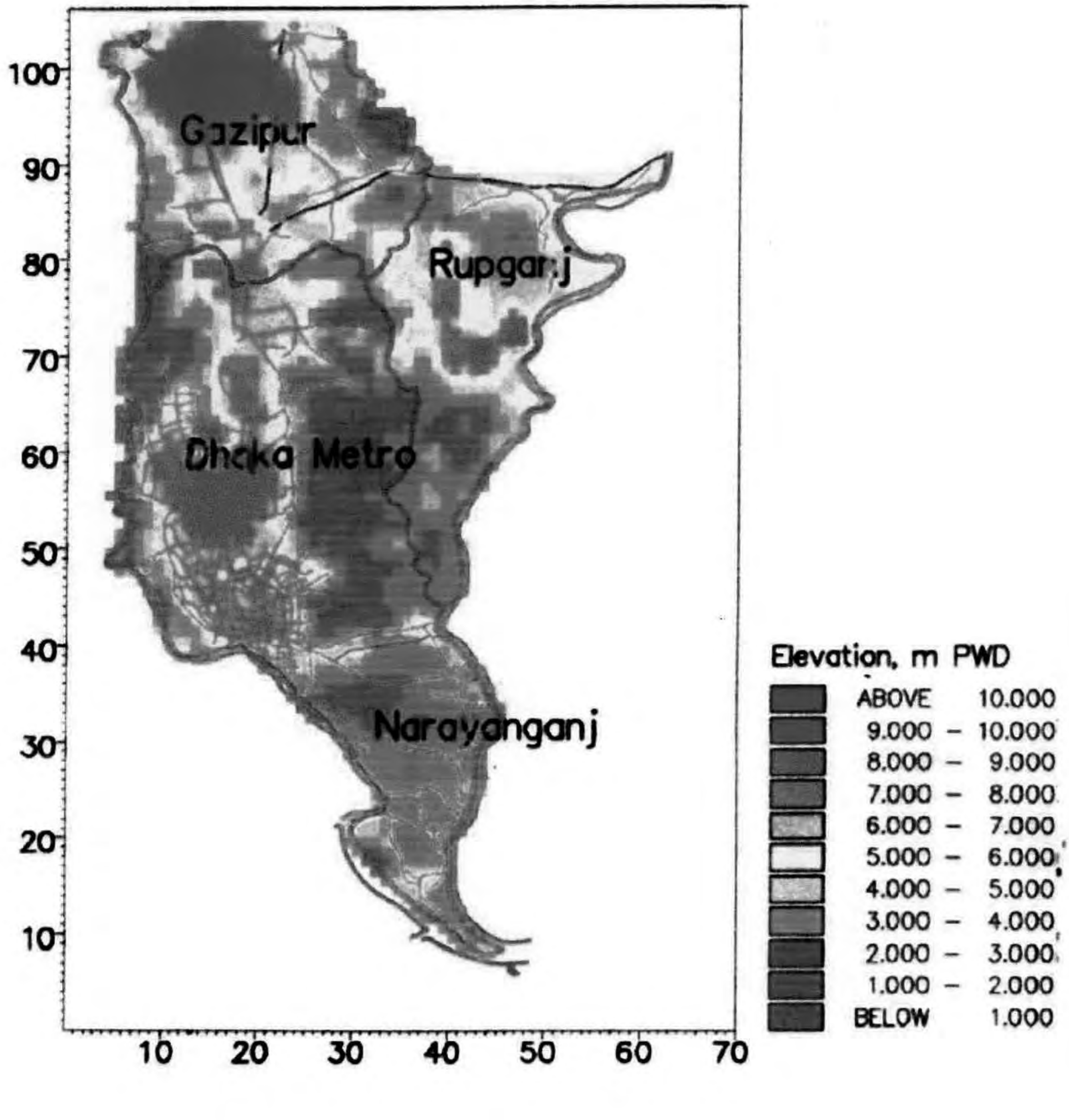


Figure 3.6 : Topograpy of Dhaka City (After SWMC, 1998)

geological profile or soil profile. In order to clearly show the various soil layers, an exaggerated vertical scale has been used. However, the exact condition of geological profile as compared to the actual soil condition depends upon the nature of ground and the spacing of the borings. If the soil conditions are erratic, the arrangement of various layers may differ considerably from the interpolation. In that case, the soil profile should be used considering that point.

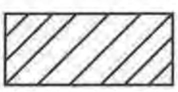
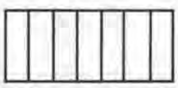
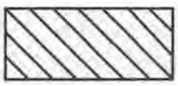
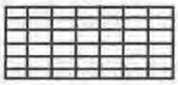
Table 3.5: Co-ordinate Values at Grid Points

Grid Point	Co-ordinates		Borehole location
	Longitude in km	Latitude in km	
A-2	535	2627	Basila
A-3	535	2630	Cotbari,Mirpur
A-4	535	2634	Botanical Garden, Mirpur
B-1	539	2623	Lalbagh
B-2	539	2627	Kalabaghan
B-3	539	2630	Kafrul
B-4	539	2634	Mirpur, Sector-11
B-5	539	2638	Bhasantech
C-1	542	2623	Wari
C-2	542	2627	Malibagh chowdhurypara
C-3	542	2630	Gulshan
C-4	542	2634	Joarasahra
C-5	542	2638	Dakhinkhan
D-1	546	2623	Kajla, Jatrabari
D-2	546	2627	Matherteck
D-3	546	2630	Satarkul

For the purpose of plotting profile along typical tunnel alignment, it is necessary to collect data of the points situated near to the Railway line. For soil profile through Kamlapur to Tongi Railway line, the soil profile data of grid line, C-C' and 2-2' of Bashar (2000) will be used, since the grid point on these gridlines are closer to the rail line. Section 2-2' passes through very close to the East-West part of the rail line. Figure 3.7 and

Figure 3.8 show the soil profile along grids C-C' and 2-2', respectively. To establish soil profile, soil layers are classified on the basis of the following factors; (1) Soil grain (2) Soil type (3) Soil consistency and (4) Soil density. The coarse grained soil (more than 50% of materials is coarser than No. 200 sieve size) are classified into five classes, designated as S1, S2, S3, S4 and S5 (Bashar,2000). The descriptions of the coarse grained soil and their symbols are presented in Table 3.6.

Table : 3.6 : Classification of Soil Strata Composed of Coarse Grained Soil (Bashar, 2000)

CLASS	SYMBOL	DESCRIPTION TERM	N-VALUE FROM SPT
S1		VERY LOOSE	0-4
S2		LOOSE	4-10
S3		MEDIUM DENSE	10-30
S4		DENSE	30-50
S5		VERY DENSE	>50

The fine-grained soil (more than 50% of materials in finer than No.200 sieve) layers have been classified into six classes mainly on the basis of consistency and SPT (Bashar,2000). These are designated as C1, C2, C3, C4, C5 and C6. The descriptions of fine grained soil and soil group symbols are presented in Table 3.7.

Figure 3.7 shows that depth of the clay layer is less toward the south and increases on the north, as those observed in Serajuddin et al. (2001). Depth of the clay on the south is around 20 ft, while the depth on the north was found as 78 ft at Joar Sahara and greater than 100 ft at Dakhin Khan. However, the deep clay layer on the north is stiff to very stiff, facilitating excavations. For the East-West part, Figure 3.8 reveals that the depth of the clay layer to the east of Kawran Bazar is around 40 ft. Borehole to the east of Kawran Bazar has been considered, since the existing rail road is situated to the east.

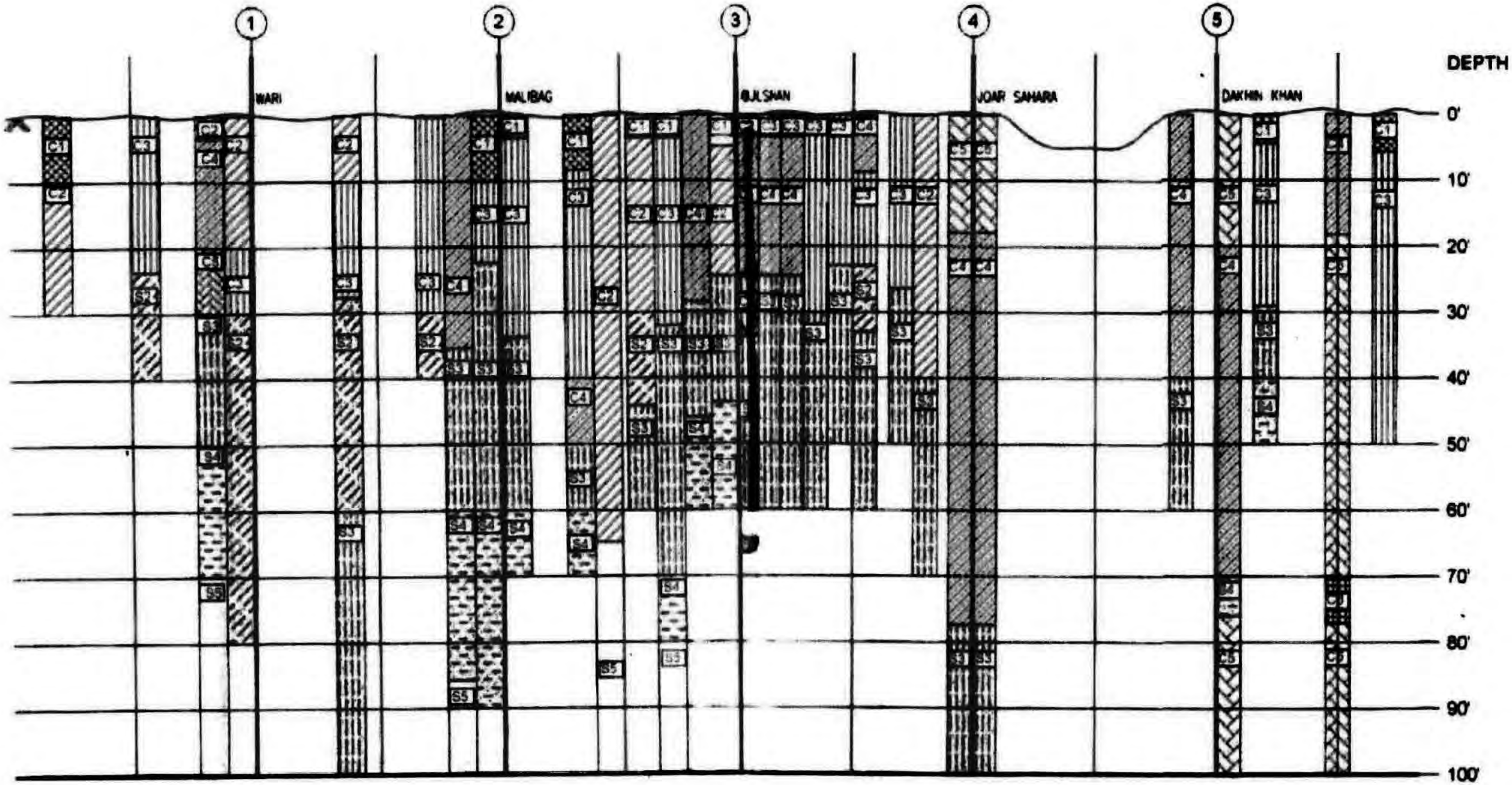


Figure 3.7 : Soil Profile Through Grid C-C' (After Bashar, 2000)

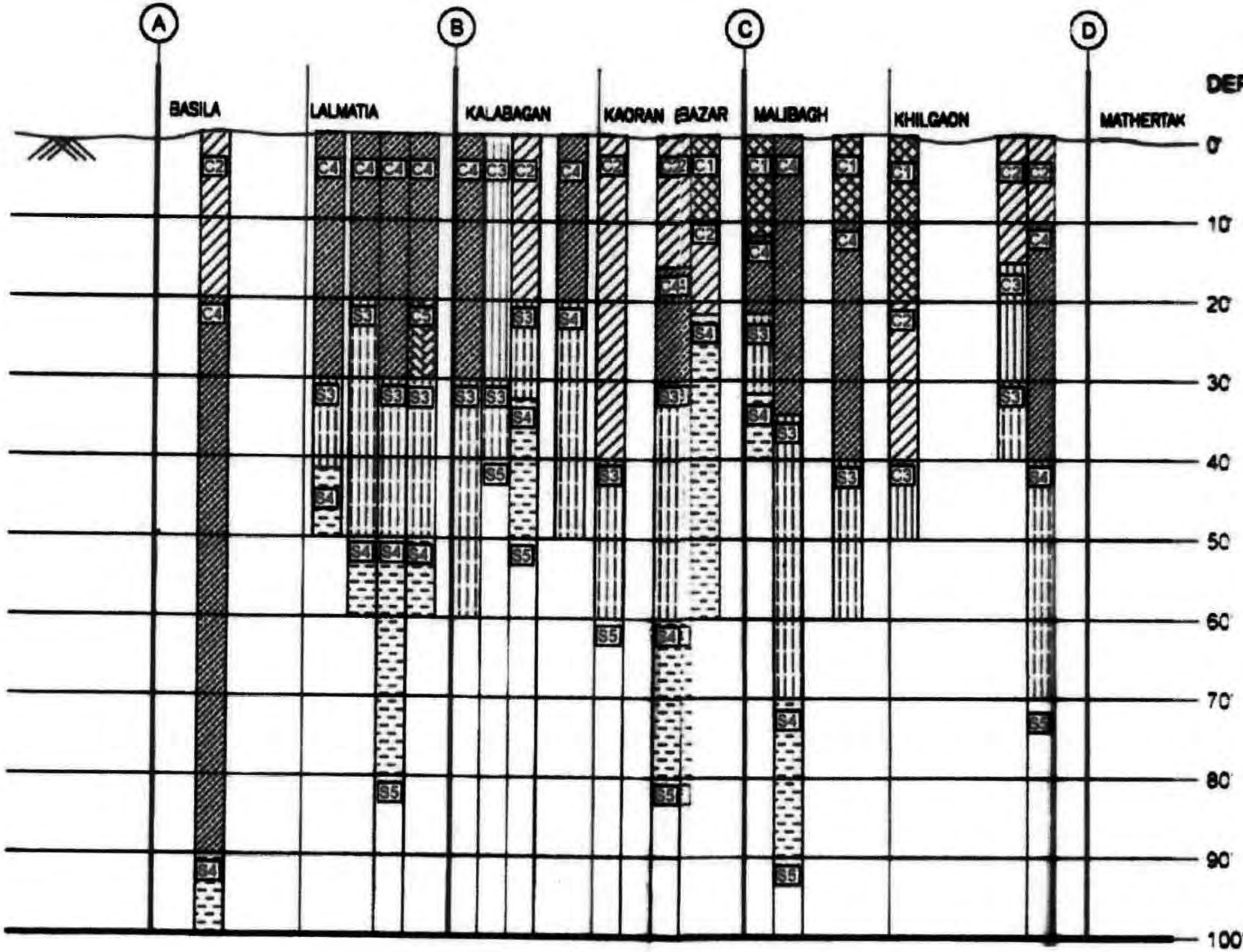


Figure 3.8 : Soil Profile Through Grid 2-2' (After Bashar, 2000)

Table : 3.7 : Classification of Soil Strata Composed of Fine Grained Soil

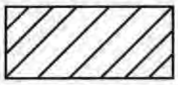
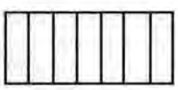
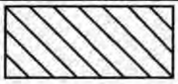
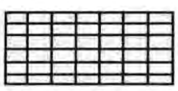
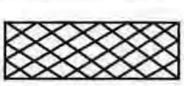
CLASS	SYMBOL	DESCRIPTION TERM	N-VALUE FROM SPT
C1		VERY SOFT	0-2
C2		SOFT	2-4
C3		MEDIUM STIFF	4-8
C4		STIFF	8-15
C5		VERY STIFF	15-30
C6		HARD	>30

Table 3.8 presents a summary of the range of the parameter for the grid points C-1, C-2, C-3, C-4, C-5, 2-C and 2-D that are within the portion of the existing rail road. In the top layers (depth upto 30 ft) of grid locations C-1 to C-5, soft to very stiff clays have been identified (Figure 3.7). N-values, LL, PI and w_n in these locations were found to vary from 2 to 26, 35 to 56, 17 to 36 and 17% to 37%, respectively. The values q_u, C_c and e_o ranged from 14 psi to 53 psi, 0.05 to 0.18 and 0.52 to 0.8, respectively. At large depths, upto 100 ft, sandy soil exist which have been classified as S3 and S4.

3.5 Soil profiles along Tunnel alignment

The longitudinal soil profile along typical locations from Kamlapur Rail Station to Tongi Bridge that covers the area of Uttarkhan, Dakhin Khan, Uttra, Khilkhet, Joar Shahara, Nadda, Kalachandpur, Baridhara, Gulshan, Badda, Rampura, Tejgoan, Maghbazar, Malibagh, Khilgaon, Mathertak, and Kamlapur was estabilished from relevant borehole. As mentioned earlier, gridlines C-C' and 2-2' of Bashar (2000) lies close to the alignment and therefore the borehole data from those locations were used.

Table : 3.8 : Range of soil parameters within the Grid Points C-1, C-2, C-3, C-4, and C-5

Grid Point	Depth (ft)	N-Value	LL (%)	PI (%)	w_n (%)	Sand (%)	Silt (%)	Clay (%)	q_u (psi)	e_o
C-1	0-30 30-100	6-26 21-50	35-55 -	17-26 -	18-21 -	3-7 70-90	65-75 5-35	6-20 -	21-26 -	0.63-0.75 -
C-2	0-30 30-50	2-15 42-50	48-56 -	24-36 -	17-35 -	4-12 87-89	66-68 11-13	20-28 -	14-23 -	0.56-0.70 -
C-3	0-30 30-80	9-13 11-37	37-47 -	13-23 -	18-25 -	6-28 80-96	63-76 4-20	8-18 -	31-35 -	0.52-0.57 -
C-4	0-25 25-100	5-16 11-27	40-54 -	15-28 -	20-37 -	6-20 81-87	68-70 13-19	12-27 -	18-33 -	0.62-0.70 -
C-5	0-28 28-50	3-15 20-50	40-42 -	20-32 -	17-23 -	6-28 84-93	66-78 7-16	7-20 -	24-53 -	0.56-0.80 -

Soil profile has been established along tunnel alignment arranging layers in accordance with the description provided in the Table 3.6 and 3.7. Figure 3.9 shows soil profile through the assumed tunnel alignment. As applicable in other grid profiles, cohesive stratifications have been found at top, which are subdivided into two to three layers, most of which layers are characterized as soil types C3 and C4, corresponding to medium stiff to stiff consistency. At relatively large depths, usually granular soil layers have been encountered except at some locations. Soil S3 and S4 layers, corresponding to medium to dense sand, have prevailed throughout longitudinal section. It is to be noted that at Uttarkhan and Dakhinkhan areas, cohesive layers were encountered upto depth of 30m (100 ft). No sandy layers could be located up to this depth. The cross profile along typical locations starting from Khilgaon Railgate to Kawran Bazar is also shown in Figure 3.9 (based on grid line 2-2' of Bashar, 2000) to cover the railway line. It has been observed that the top layers are usually cohesive up to a maximum depth of about 40 ft, which consists of clay types C1, C2 and C4. Thus, surface soil of very soft to soft consistency was encountered, as expected from the geological profile. The area lies

CLASS	SYMBOL	DESCRIPTION TERM	N-VALUE FROM SPT
S1		VERY LOOSE	0-4
S2		LOOSE	4-10
S3		MEDIUM DENSE	10-30
S4		DENSE	30-50
S5		VERY DENSE	>50

CLASS	SYMBOL	DESCRIPTION TERM	N-VALUE FROM SPT
C1		VERY SOFT	0-2
C2		SOFT	2-4
C3		MEDIUM STIFF	4-8
C4		STIFF	8-15
C5		VERY STIFF	15-30
C6		HARD	>30

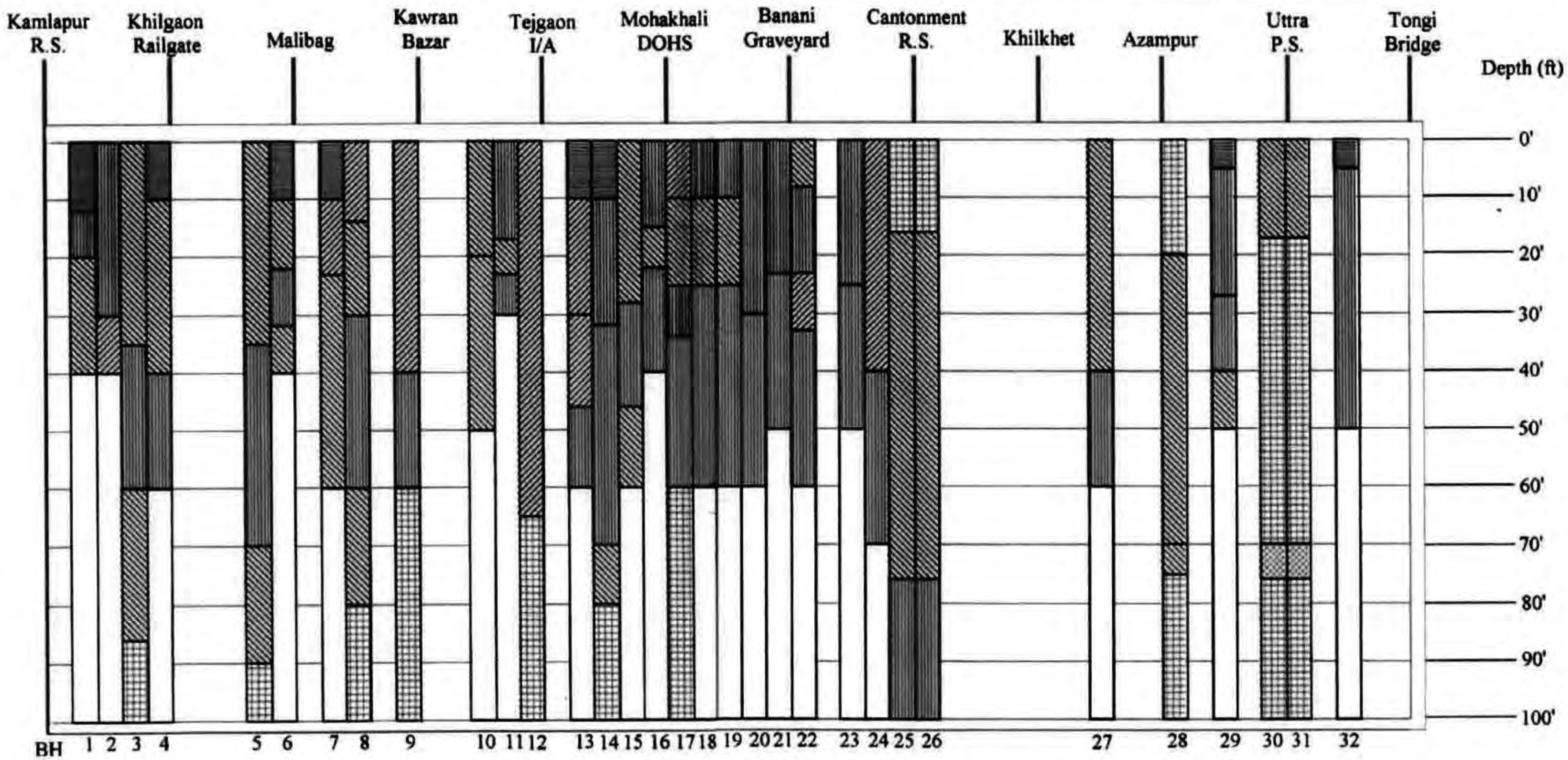


Figure 3.9 : Soil Profile Through Kamalapur to Tongi Bridge

on high to low alluvium zone (Karim et al., 1993). However, the soft clays (C1 or C2) are limited up the depths of 10 to 12 ft, underneath of which medium to stiff clay exits. The underlying layers are granular deposits, which consist of several sandy layers. The sand types are mainly S3, S4 and S5 types with medium to very dense consistency.

Based on the borelog information presented in Figure 3.9, typical soil conditions along railroad are assumed as shown in Table 3.9. In order to arrange the data in a tabular form, this route is divided into 11 zones. There are minimum 1 to maximum 5 boring has been accomplished in each zone. The worst and the best soil conditions are selected according to the class and depth of upper clay layer revealed from the Boreholes. The depth of boring has been varies from 40 ft to 100 ft. When the boring depth is limited up to 40 ft, and no sand layer was encountered within this depth then the cell in the table is kept blank.

Table : 3.9 : Typical soil condition along the tunnel route

Location	Soil description from Boreholes					Assumed soil profile	
	B. Hole (Fig.3.9)	Clay layer		Sand layer		The Worst case	The Best case
		Class	Depths,ft	Class	Depths, ft		
Kamlapur to Khilgaon Railgate	BH : 1	C_1+C_3+C_4	12+8+20	-	-	**Clay:** 40 ft with C_1, C_2 and C_3 as 12ft, 8ft and 20ft **Sand:** S_3 beyond 40 ft	**Clay:** 35 ft C_4 **Sand:** S_3 beyond 35 ft
	BH : 2	C_3	30	S_2	10		
	BH : 3	C_4	35	S_3+S_4+S_5	25+26+14		
	BH : 4	C_1+C_4	10+30	S_3	20		
Khilgaon to Malibag	BH : 5	C_4	35	S_3+S_4+S_5	35+20+10	**Clay:** 22 ft with C_1 and C_4 as 10ft and 12ft **Sand:** S_3 beyond 22 ft	**Clay:** 35 ft C4 **Sand:** S_3 and S_4 as (35ft and beyond)
	BH : 6	C_1 +C_4	10+12	S_3+S_4	10+8		
Malibag to Kawran Bazar	BH : 7	C_1+C_2	10+13	S_4	37	**Clay:** 40 ft C_2 **Sand:** S_3 and S_5 as (20ft and beyond)	**Clay:** 30 ft with C_2 and C_4 as 14ft and 16ft **Sand:** S_3 and S_4 as (30ft and beyond)
	BH : 8	C_2+C_4	14+16	S_3+S_4+S_5	30+20+20		
	BH : 9	C_2	40	S_3+S_5	20+40		
Kawran Bazar to Tejgaon I/A	BH : 10	C_4	20	S_4	30	**Clay:** 65 ft C_2 **Sand:** S_5 beyond 65 ft	**Clay:** 20 ft C_4 **Sand:** S_4 beyond 20 ft
	BH : 11	C_3+C_4	17+6	S_3	7		
	BH : 12	C_2	65	S_5	35		
Tejgaon	BH : 13	C_1+C_2	10+20	S_2+S_3	16+14	**Clay:** 30 ft with	**Clay:** 27 ft C_4

Table 3.9 Contd.

I/A to Mohakhali DOHS	BH : 14	C_1+C_3	10+22	$S_1+S_4+S_5$	38+10+20	C_1 and C_3 as 10ft and 20ft **Sand:** S_2 and S_3 as (16ft and beyond)	**Sand:** S_3 and S_4 as (18ft and beyond)
	BH : 15	C_4	27	S_3+S_4	18+15		
	BH : 16	C_3+C_4	15+7	S_3	18		
Mohakhali DOHS to Banani Graveyard	BH : 17	$C_2+C_4+C_3$	10+15+9	S_3+S_5	26+40	**Clay:** 34 ft with C_1, C_4 and C_3 as 10ft, 15ft and 9ft **Sand:** S_3 and S_5 as (26ft + beyond)	**Clay:** 25 ft with C_3 and C_4 as 10ft and 15ft **Sand:** S_3 beyond 25 ft
	BH : 18	C_3+C_4	10+15	S_3	35		
	BH : 19	C_3+C_4	10+15	S_3	35		
	BH : 20	C_3	30	S_3	30		
	BH : 21	C_3	23	S_3	27		
Banani Graveyard to Cant.	BH : 22	$C_4+C_3+C_2$	8+15+10	S_3	27	**Clay:** 34 ft C_2 **Sand:** S_3 beyond 40 ft	**Clay:** 77 ft with C_5 and C_4 as 17ft, and 60ft **Sand:** S_3 beyond 77 ft
	BH : 23	C_3	25	S_3	25		
	BH : 24	C_2	40	S_3	30		
	BH : 25	C_5+C_4	17+60	S_3	23		
Cant. to Khilkhet	BH : 26	C_5+C_4	17+60	S_3	23		
Khilkhet to Azampur	BH : 27	C_4	40	S_3	20		
Azampur to Uttra	BH : 28	$C_5+C_4+C_5$	20+50+5	S_4	25	**Clay:** 26 ft with C_1 and C_3 as 5ft and 21 ft **Sand:** S_3 and S_4 as (14ft and beyond)	**Clay:** 75 ft with C_5, C_4 and C_5 as 20ft, 50 ft and 5 ft **Sand:** S_4 beyond 75 ft
	BH : 29	C_1+C_3	5+21	S_3+S_4	14+10		
	BH : 30	$C_4+C_5+C_6+C_5$	17+53+6+24	-	-		
Uttra to Tongi Bridge	BH : 31	$C_4+C_5+C_6+C_5$	17+53+6+24	-	-	**Clay:** over 50 ft C_1 and C_3	**Clay:** over 100 ft with C_4, C_5 and C_6
	BH : 32	C_1+C_3	5+45	-	-		

To estabilish the soil parameters of the soil encountered along the profile, test results and available correlation have been used. Correlation between unconfined compressive strength(q_u) and N-value obtained from SPT were found by McEarthy (1977), Boweles (1988), Serajuddin (1996) and Bashar(2000) is q_u = kN, where k is the proportionality factor. Serajuddin and Chowdhury (1996) suggested the average value of k = 16 for Dhaka metropolitan city when q_u is expressed in kPa. When expressed in ksf, the relationship becomes

$$q_u = N/3 \text{ ksf} \quad \ldots\ldots (3.1)$$

McCarthy (1977) has shown the following correlation between q_u and N :

$$q_u = N/2.5 \text{ ksf (for silty clay)} \quad \ldots\ldots (3.2)$$

and

$$q_u = N/2 \text{ ksf (for clay)} \quad \ldots\ldots (3.3)$$

Bowles (1988) suggested a correlation between q_u and N as

$$q_u = N/4 \text{ ksf} \qquad (3.4)$$

Relationship between q_u and N expressed by Bashar (2000) is as follows :

$$q_u = N/2.77 \text{ ksf} \qquad (3.5)$$

Table 3.10 expresses the estimated parameters for fine grained soil. This table includes the types of clay which is classified from C1 to C6, the value of undrained cohesion (C_u), void ratio (e), water content (w), specific gravity of soil solids (G_s), dry unit weight (γ_d), saturated unit wt (γ_{sat}), effective unit wt (γ'). The estimated minimum, average and the maximum cohesion values are included in the table.

Table 3.10: Values of different soil parameters composed of Fine Grained soil

Class	Undrained Cohesion C_u (psf)			Void Ratio, e	Water Content, w (%)	Specific Gravity of Soil Solids, G_s	Dry Unit Weight, γ_d(lb/ft^3)	Saturated Unit wt, γ_{sat}(lb/ft^3)	Effective Unit wt, γ'(lb/ft^3)
	Min^m C_u	Max^m C_u	Typical C_u						
C1	300	864	400	0.88	31	2.84	94.2	123.4	61.0
C2	800	1400	900	0.83	27	3.07	104.7	132.9	70.5
C3	1656	1872	1600	0.79	26	3.04	105.9	133.5	71.1
C4	2232	2520	2400	0.75	24	3.13	111.5	138.3	75.9
C5	2592	3816	3000	0.62	21	2.95	113.6	139.5	77.1
C6	3888	4000	4000	0.58	19	3.05	120.4	143.3	80.9

* 1 psf = 0.048 kpa

Table 3.11 presents the soil parameter for coarse-grained soil estimated from the SPT N-values. This table includes the types of sand, which is classified from S1 to S5, the value of angle of internal friction (φ), void ratio (e), water content (w), specific gravity of soil solids (G_s), dry unit weight (γ_d), saturated unit wt (γ_{sat}), effective unit wt (γ') according to the class of sand.

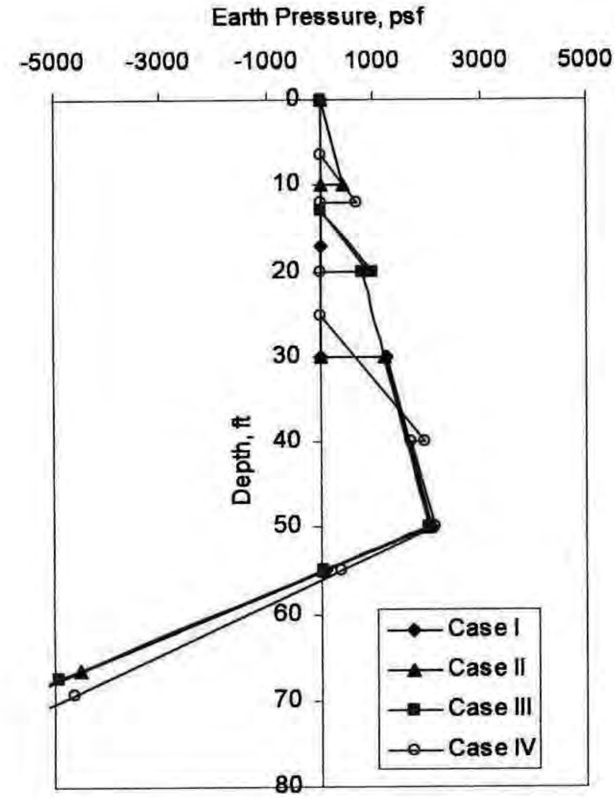
Table 3.11: Values of different soil parameters Composed of Coarse Grained Soil

Class	Angle of internal friction, φ°	Void Ratio, e	Water Content, w (%)	Sp.gr. of soil solids, G_s	Dry Unit Weight, γ_d(lb/ft^3)	Saturated Unit wt, γ_{sat}(lb/ft^3)	Effective Unit wt, γ'(lb/ft^3)
S1	15	0.88	31	2.84	94.2	123.4	61.0
S2	20	0.83	27	3.07	104.7	132.9	70.5
S3	30	0.79	26	3.04	105.9	133.5	71.1
S4	35	0.75	24	3.13	111.5	138.3	75.9
S5	40	0.62	21	2.95	113.6	137.5	75.1

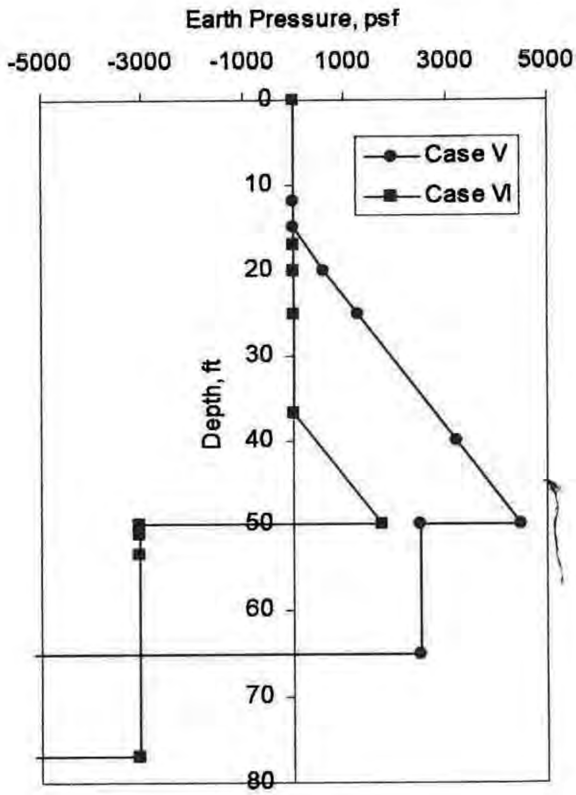
Table 3.12: Soil cases considered for analysis of earth retention system

Ground case	Depths (ft)	Soil type	Sat. Unit wt. (lb/ft^3)	Estimated strength parameters	Comments
Case I	0-30	C_4	135	Cu = 2400 psf	Typical case
	> 30	S_3/S_4	134	$\phi = 32^{\circ}$	
Case II	0-10	C_1	123	Cu = 400 psf	Typical case with soft clay
	10-30	C_4	135	Cu = 2400 psf	
	> 30	S_3/S_4	134	$\phi = 32^{\circ}$	
Case III	0-20	C_2	130	Cu = 900 psf	Shallow clay
	> 20	S_3/S_4	134	$\phi = 32^{\circ}$	
Case IV	0-12	C_1	123	Cu = 400 psf	A complex case
	12-20	C_2	130	Cu = 900 psf	
	20-40	C_3	132	Cu = 1600 psf	
	> 40	S_3	132	$\phi = 30^{\circ}$	
Case V	0-65	C_2	130	Cu = 1000 psf	Worst case
	> 65	S_4	136	$\phi = 35^{\circ}$	
Case VI	0-17	C_5	130	Cu = 3000 psf	The best case
	17-77	C_4	131	Cu = 2400 psf	
	> 77	S_3	132	$\phi = 30^{\circ}$	

Considering the soil profile and the soil parameters discussed above, six general cases of ground condition has been chosen for analysis of earth retention system for construction of tunnel in Dhaka city (Table 3.12). Rankin earth pressures expected on a retaining structure under these soil conditions are compared in Figure 3.10. Earth pressure with the depth of clay layer less than or greater than the expected excavation depth (50 ft) are plotted separately in the figure. It is revealed that the earth pressure is almost same for the soil conditions in Case I through IV, except some variation at shallow depth. Among the soil conditions, the Case III with shallow depth of clay (20 ft) appear to produce maximum active earth pressure in Figure 3.10 (a). Thus, the case has been considered further for analysis of earth retention system, as discussed in Chapter 5. The worst and the best soil conditions (Case V and Case VI) encountered are also considered. Detail design of the earth retention systems for construction of the metro tunnel in Dhaka is discussed in Chapter 5.



(a) Clay depth less than excavation depth



(b) Clay depth greater than excavation depth

Figure 3.10: Lateral earth pressure on a retention system

CHAPTER 4

PROPOSED METRO RAIL SYSTEM

4.1 Introduction

As discussed earlier in Chapter 1, the feasibility of a metro rail system along the existing railway track from Tongi to Kamalapur has been investigated in this research. Type of tunnel structure and possible Metro Station has been selected based on typical structure used for metro system. The size of tunnel structure usually depend on number of track to be used and the clearance requirements. Double box structure that would accommodate double track has been considered here.

4.2 Clearances for Railroad Tunnels

Although clearances may vary for individual railroads to suit the dimensions of their equipment, the minimum clearances for tunnels are identical to those recommended by the American Railway Engineering Association as shown in Figure 4.1 (Bickel et al, 1997). On curved Track, the lateral clearances should be increased for the mid-ordinate and overhang of a car 88 ft long and 62 ft between the centers of tacks, equivalent to 1 in per degree of curvature.

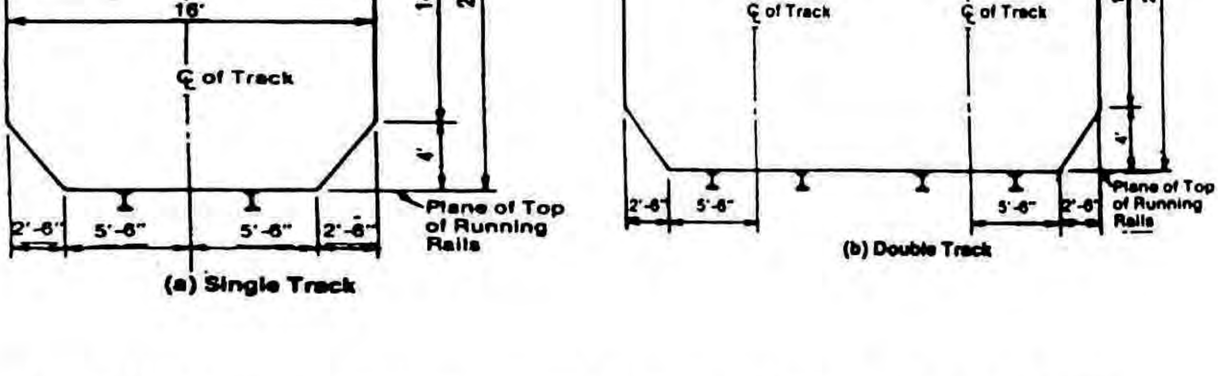
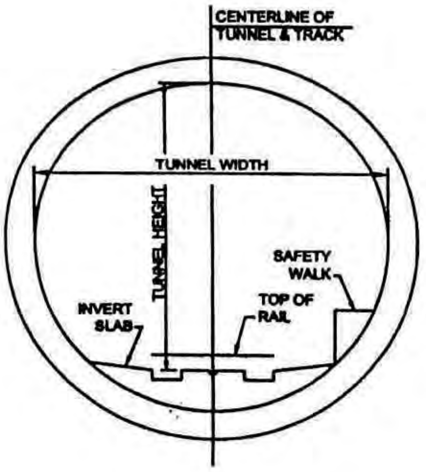


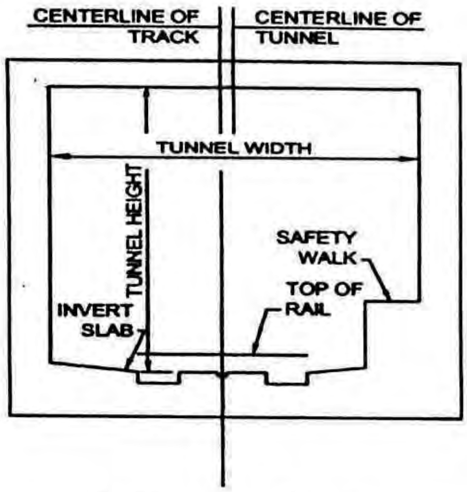
Figure 4.1 : Minimum clearances recommended for railroad tunnels (Bickel et al, 1997)

4.3 Typical Tunnel Sections

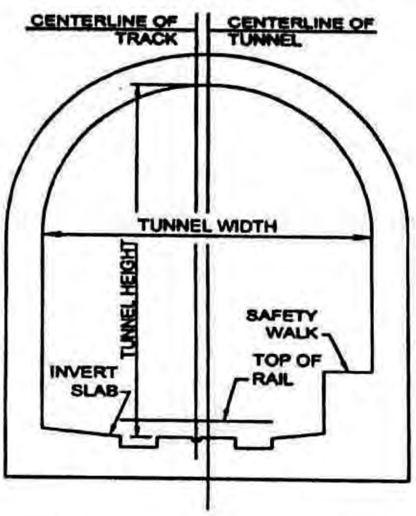
For a railroad, transit, or vehicular tunnel, selection of cross-section is dependent on vertical and horizontal clearances, number of lanes or tracks, type of ventilation system, and the method of construction. Typical cross-section configurations for cut and cover construction method are circular, octagonal, arch and rectangular. The configuration for a two lane single bore is best suited to a circular or octagonal shape tunnel, which structurally is the most efficient. The flat-bottomed arch shape is suited to single bore tunnels with a semi-transverse supply ventilation system. Rectangular shape for multilane single or multiple roadways have center or side duct locations, which reduce the depth of the structure and dredging. Figure 4.2 shows the typical shapes for rail transit tunnels. These shape typically relates to the method of construction and ground conditions in which they are constructed. The shape of rail transit tunnels often varies along a given rail line. These shapes typically change at the transition between the station structure and the typical tunnel cross-section. However, the change in shape may also occur between stations due to variations in ground conditions. The configuration of tunnel structure can depart from the typical section to accommodate a typical track alignment or grade. When system standards mandate that stations be designed with a center platform, the track centers will need to be widened through an appropriate transition length upon approaching the station. Occasionally, it will be advantageous to gradually change the alignment and grade to the "over and under" positions in which one track lies above and in line with the other. In general, the configuration of the tunnel structure must be subordinate to system requirements for track alignment and grade. However, when the stations be designed with side platform, the track centers will be same upon approaching the station. Thus, the alignment and grade for the double box construction can be the same at joint section between tunnel structure and the station building. As a result, no extra volume of materials will be required for this arrangement of double box section. For a cut-and-cover double box subway line structure, the station is usually of the side platform type (Bickel et al., 1997). A standard double box section for a double lane metro is shown in Figure 4.3. A box type structure is generally recommended in areas where the cost of concrete is less relative to the cost of sheet (Bickel et al, 1997). Workmanship requirement for box type structure is not high compared to the oval or circular structure. A box type structure is therefore recommended for Dhaka metro tunnel.



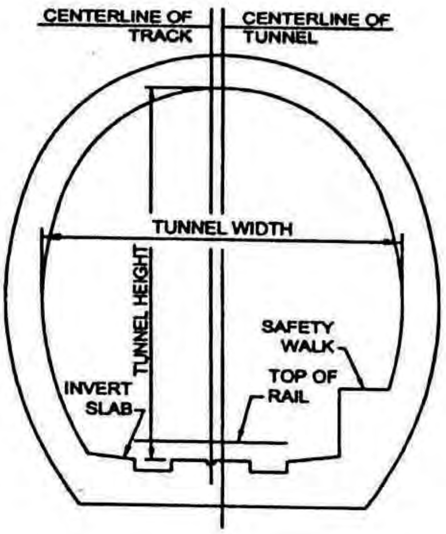
(a) Circular Tunnel



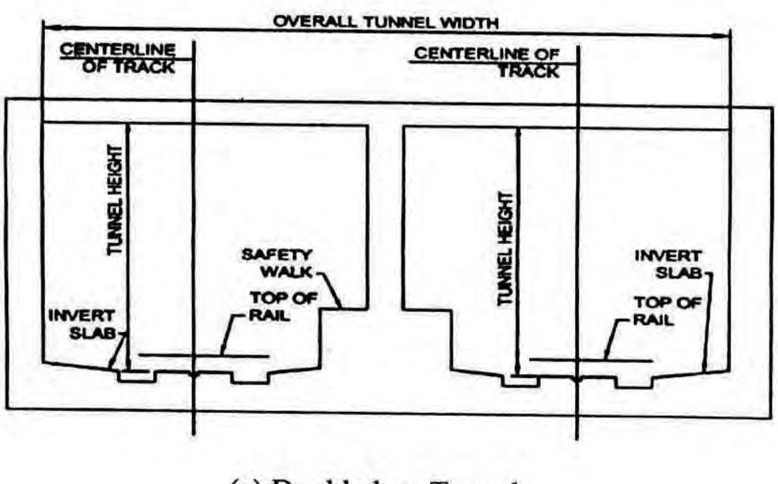
(b) Single box Tunnel



(c) Horseshoe Tunnel



(d) Oval Tunnel



(e) Double box Tunnel

Figure 4.2 Circular, Single box, Horseshoe, Oval and Double box tunnel (FHWA, 2005)

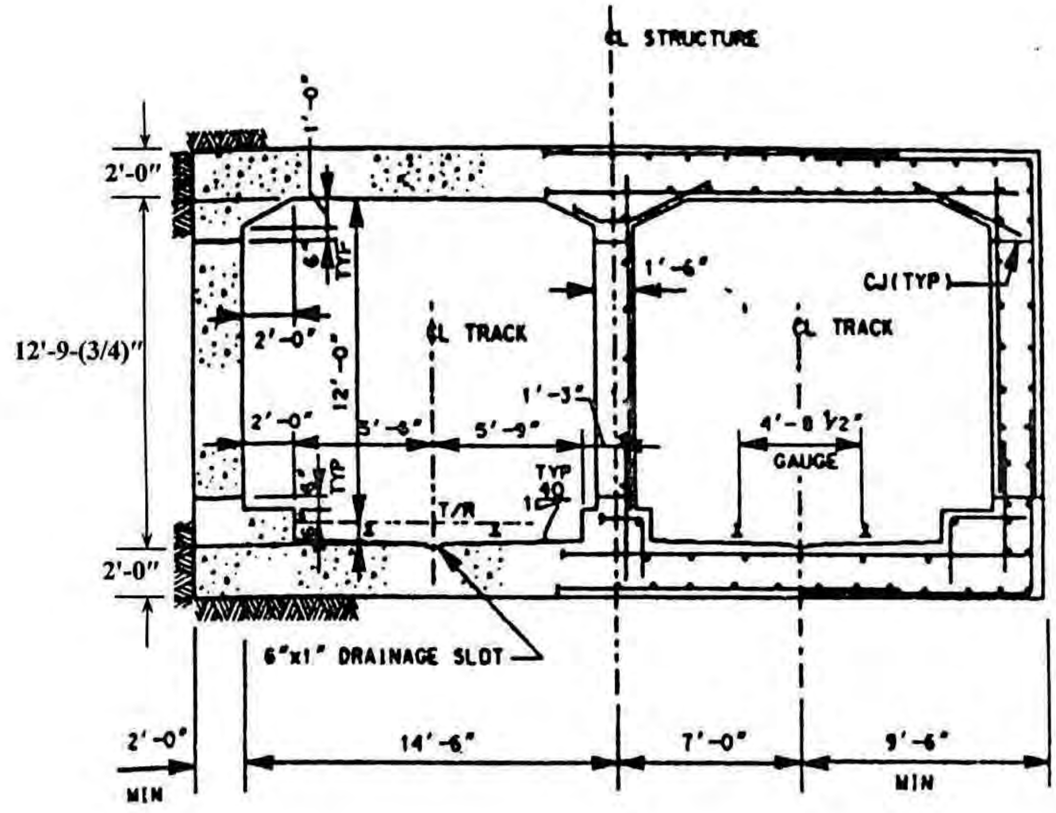


Figure 4.3 : Standard double box section. (Bickel et al, 1997)

4.4 Metro Railway stations

The usual perception of a cut-and-cover subway station is that of a two or three story reinforced concrete structure constructed in a rectangular excavation 50-65 ft wide, 500-800 ft long, and 50-65 ft deep (Bickel et al., 1997). Typically the station is a two story structure with two tracks, as well as the center or side platforms, supported on the invert. A mezzanine floor typically lies between the roof slab and the invert or platform level. Figure 4.4 shows a station box section with central platforms for the Silicon Valley Rapid Transit Project (Burns, 2007). This station box is 60 ft wide, 40 ft high which top is approximately 20 ft underground and 1000 ft long. The complete subway station will be much more complex than is indicated in Figure 4.4. Internal configuration will be significantly affected by the need to provide escalators, stairs, ventilation requirements, rooms for mechanical and electrical equipment and other maintenance, and safety and service facilities. Architectural treatment of the station will also effect internal configuration and may have an important effect on the design of the basic structure as well. The external configuration of the station will ordinarily be irregular at and above the mezzanine level, where station entrance must be provided. Although the two story station may be considered conventional, more complex station structures are not unusual, particularly in and near the center of urban areas. At these locations the subway stations may be constructed at the intersections of principal system train-way lines, for example. In such cases boarding and off-boarding platforms in two directions are normally required, so that in plan view the station structure is constructed in the shape of a cross, with, usually, more than two stories. Whenever there is a requirement that the subway station accommodate more than two tracks, the station structure will ordinarily be significantly more complex than the conventional subway station.

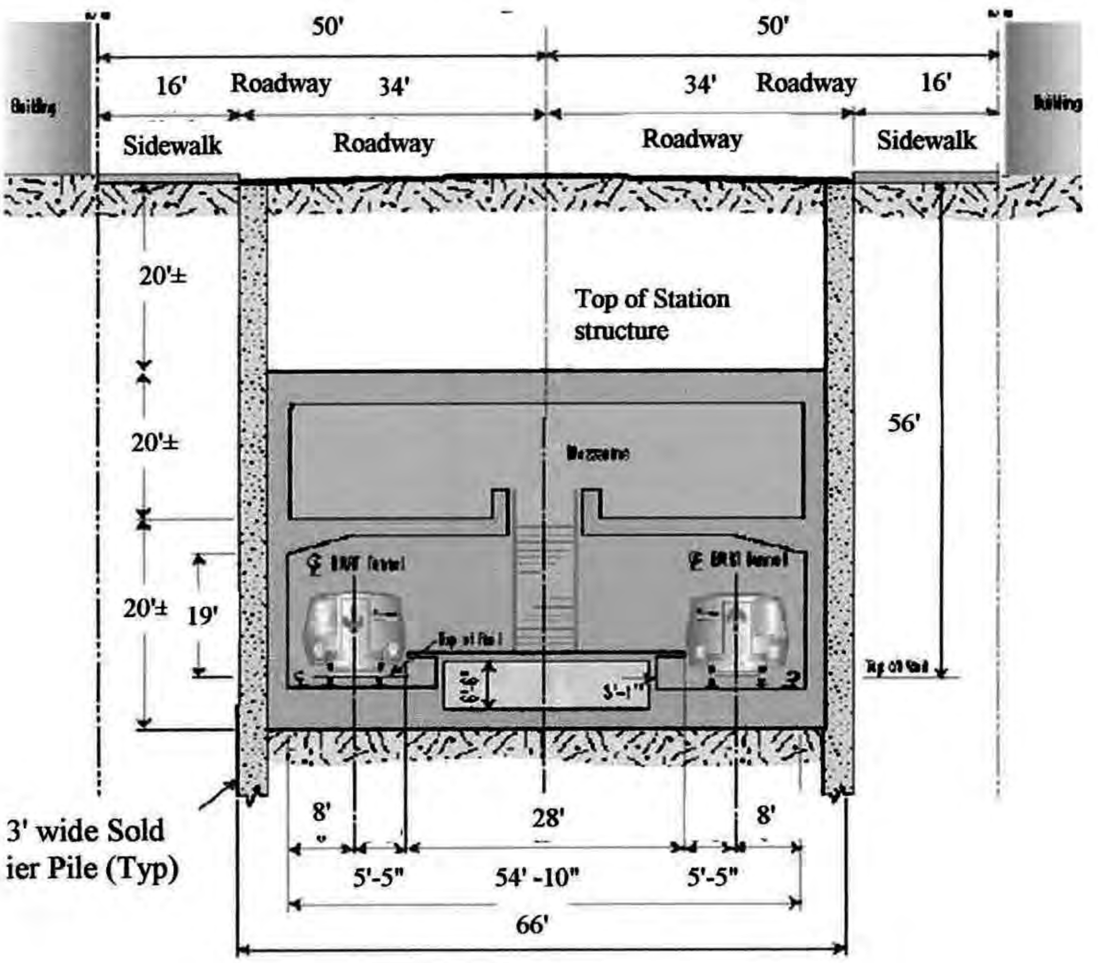


Figure 4.4 : Station Box with central platform (Burns, 2007)

Figure 4.5 shows a side platform based subway station at De Ruijterkade, Amsterdam, The Netherlands. Escalators bring passengers there from the platform level to a concourse level, from there escalators go sideways in two directions to ground level.

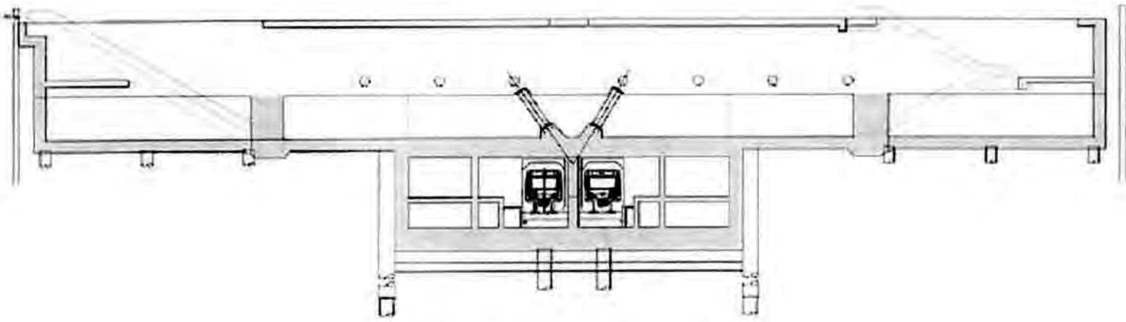


Figure 4.5 : Station Box with side platform at De Ruijterkade, Amsterdam, The Netherlands (Ploeg et al., 2006)

4.5 Proposed System for Dhaka

4.5.1 Subway Line

Typical subway line structure for different metro rail has been discussed in the earlier sections. It appeared that the structure type and the sizes usually depend on track width of line, construction type, and the clearance requirement. Existing rail line of Bangladesh Railway between the Kamalapur Railway Station and Tongi Bridge, proposed for the tunnel, uses Meter Gauge with 1000 mm of width (Information Book of Bangladesh Railway, 2005). Two lines currently exist between the stations, covering a total rail road width of about 30 ft. Considering minimum clearances recommended for railroad tunnels by American Railway Engineering Association for double track, the width of a double track tunnel is 29 ft as shown in Figure 4.1. A tunnel box width of 30 ft is chosen in this study for the proposed double track metro tunnel in Dhaka. Vertical clearance within the box was used as 13 ft based on the standard box section shown in Figure 4.3. Figure 4.3 showed the standard double box section recommended by Washington Metropolitan Area Transit Authority. A similar section with dimensions shown in Figure 4.6 is proposed as the tunnel section for Metro tunnel for Dhaka City.

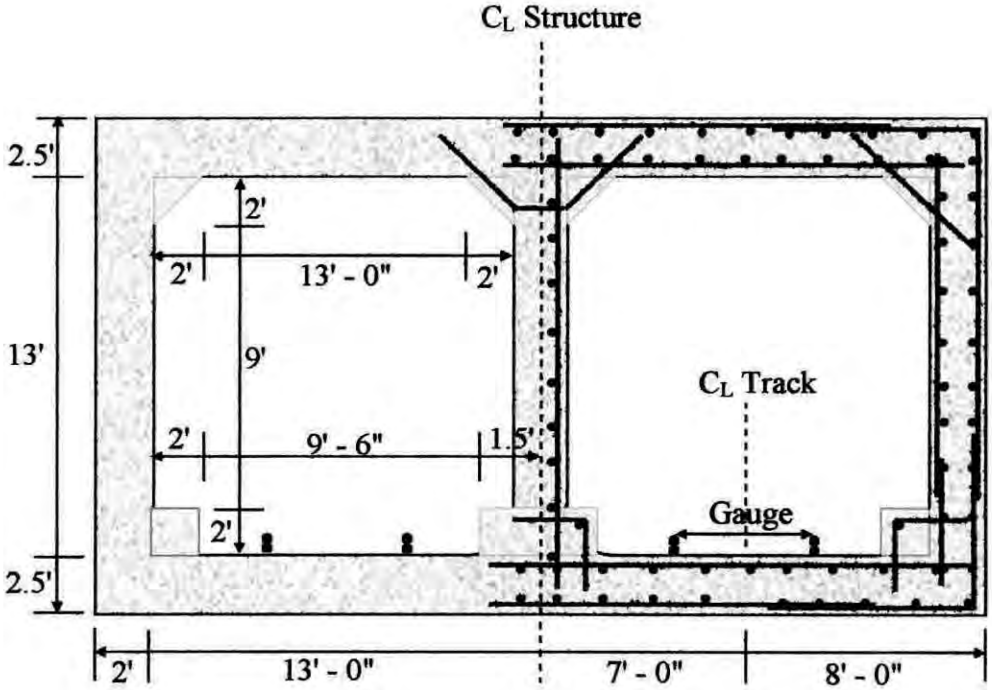


Figure 4.6 : Proposed double box section. (Not to scale)

4.5.2 Subway Station

Subway line with both center platform and side platform are possible for metro rail systems. As discussed earlier, central platform require changes of alignment of track. The track centers need to be widened through an appropriate transition length upon approaching the station. However, for side platform, the same alignment can be used. Separation of the passengers for Metro in two alternate direction are also possible. It is recommended to use the side platform type metro station for a cut-and-cover double box subway (Bickel et al, 1997). A side platform type station is therefore chosen for the Metro line in Dhaka City. For easy entrance into the Metro station a three storied building is considered. The width and height of Metro railway station are the same as those for typical stations. Figure 4.7 shows a plan of the proposed station. Similar station was used for Daikai station in Japan (Umehara et al. 1998). Two sections of Daikai Station, one across the station building and the other beyond the building as shown in Figure 4.8, and Figure 4.9. A similar station with adjustment of the dimensions has been chosen for the proposed Metro system for Dhaka. Height of the platform from the rail was taken as the typical height used for train such as 800 mm to 1200 mm (Wikipedia). The height of a railway platform usually varies between railway systems. Heathrow Airport Ltd specified their platform height at 1100 mm above rail level for the Heathrow Express rail service to and from Paddington Station (Wikipedia). Figure 4.10 shows the cross section of the proposed subway station. There are six columns inside the stations which have dimensions 36"x 36". Clear spacing between the columns exists 25 ft in longer direction and 20 ft in shorter direction. The platform height above plinth level is 3.5 ft. Total length of the platform is chosen as 500 ft. A provision two staircase and two escalator is included in the station.

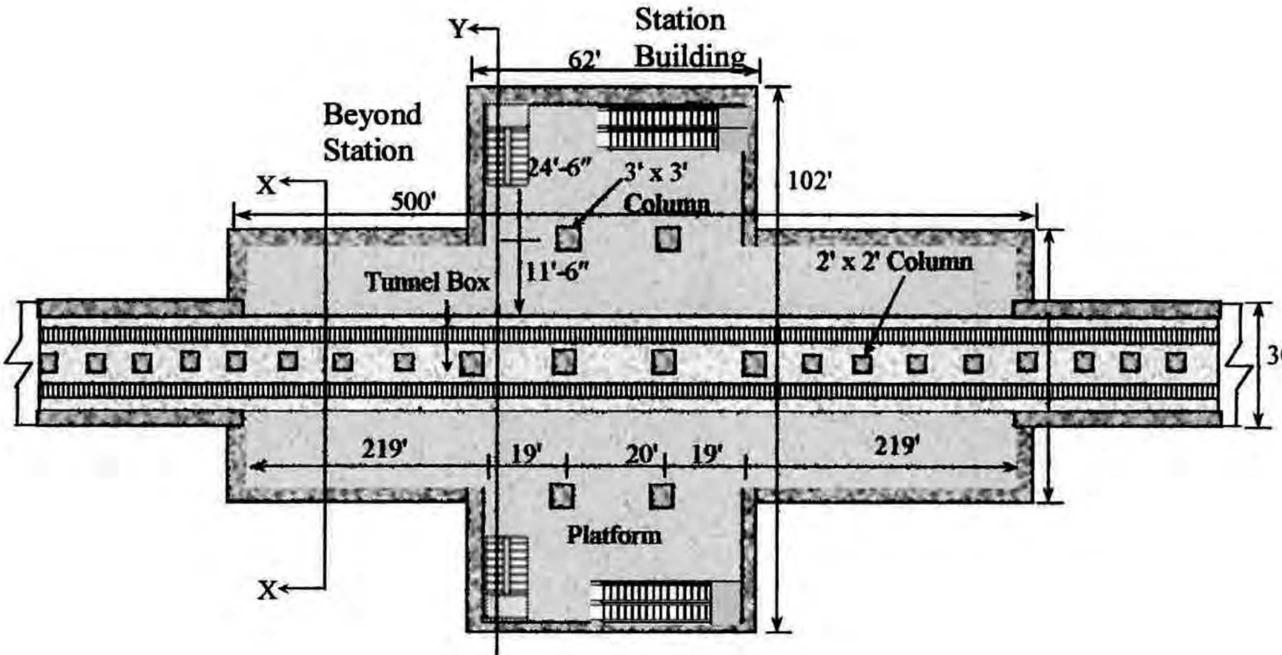
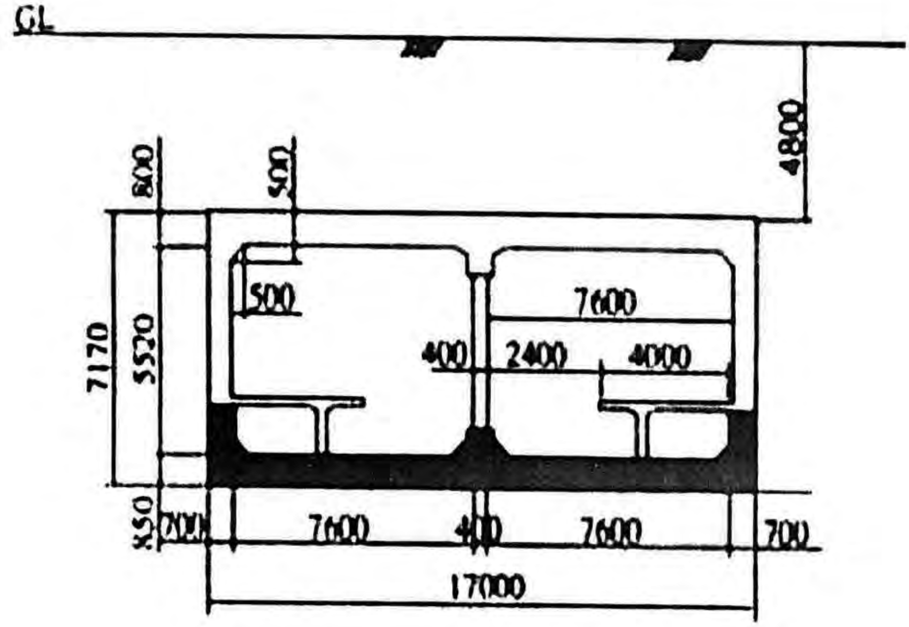
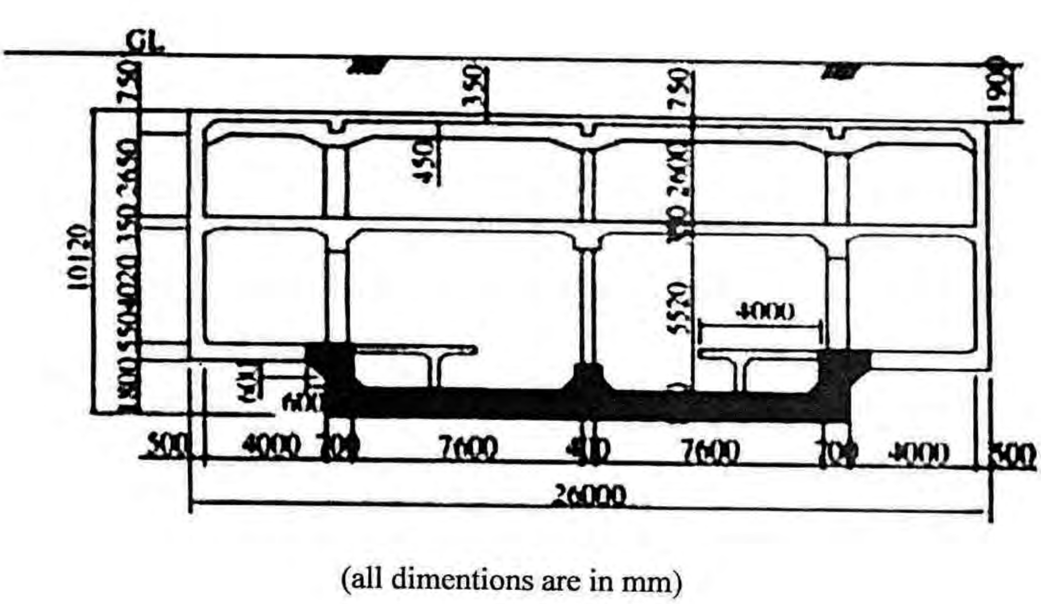


Figure 4.7 : Sectional Ground Floor Plan of a Typical Railway Station.(Not to scale)



(all dimentions are in mm)

Figure 4.8: Cross-section beyond the station building at Daikai Station, Japan (Umehara, 1998)



(all dimentions are in mm)

Figure 4.9: Cross-section across the station building at Daikai Station, Japan (Umehara, 1998)

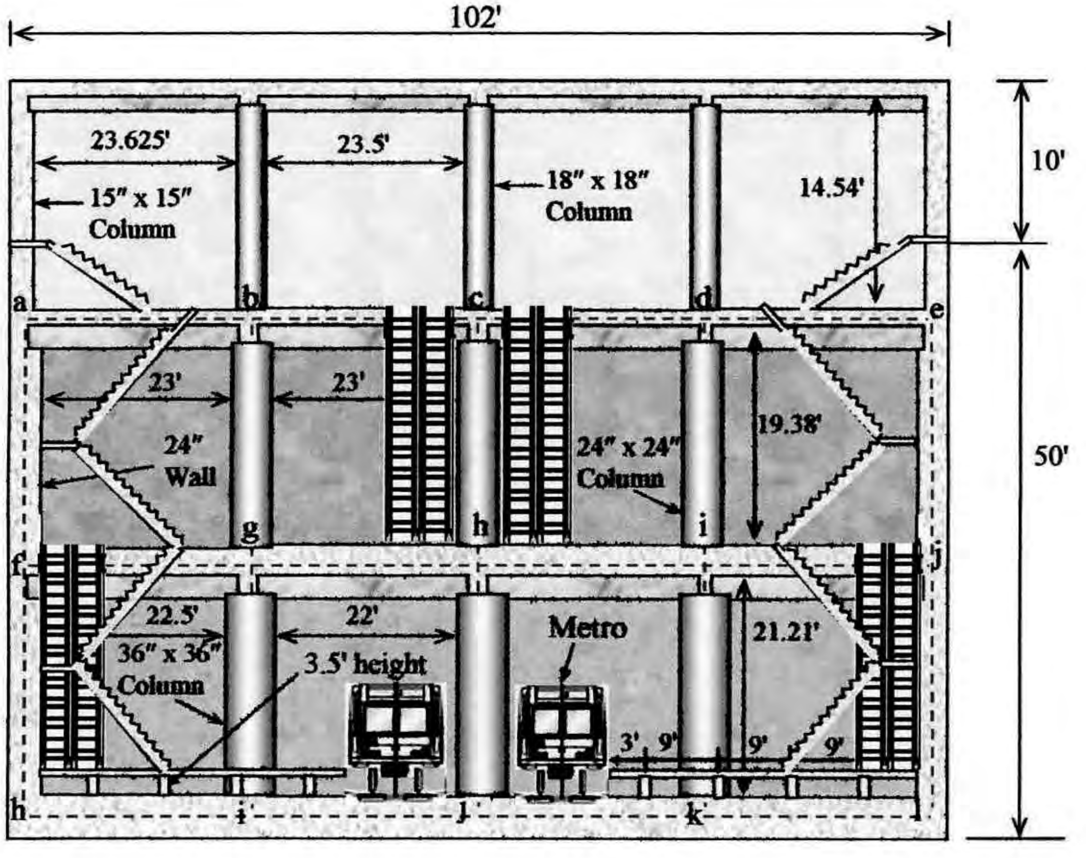


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Figure 4.10 : Cross section of Station Building in Y-Y direction. (Not to scale)

CHAPTER 5

DESIGN OF THE TUNNEL SYSTEM

5.1 Introduction

The Cut and Cover method of tunneling is relatively cheap for shallow tunnels and requires simple technology. In this method, the ground is excavated along a planned route using temporary or permanent retaining structures to retain soil up to the depth of excavation. Tunnel is then built and covered by soil. Several methods can generally be used for earth retention during excavation in crowded city like Dhaka. These are: (1) Cantilever Sheet pile, (2) Sheet pile with bracings, (3) Diaphragm wall with bracings, (4) Bored pile with bracings etc. Suitability of each of the method depends on the type of soil and on the availability of the technology. Chapter 3 of this thesis describes the soil profile along the proposed tunnel alignment. Earth retention system is designed to cover all the ranges of soil types encountered at the site. As seen in chapter 3, the upper soil of the site is generally clay that is underlain by sand. Thickness of clay layer varied from 20 ft up to over 100 ft. Soft clay was encountered at some places. Analysis was also performed to investigate the effect of nearby structure on the earth retention system. Types of structure and their distance from the existing rail track is discussed in Chapter 1. Depth of cut has been taken as 50 ft as discussed in Chapter 4.

5.2 Design Considerations

5.2.1 Factors Influencing Earth Retention System

Design of earth retention system during deep excavation depends on many factors. These are:

(1) Water Table: There are mainly two types of water that are considered during design, i.e. (a) Ground Water (b) Surface Water (Trapped water).

(a) Ground Water: For Dhaka City ground water table is much lower than the depth of cut (i.e, 50 ft). Even during a rainy season this water table is not expected to affect the construction. Thus ground water is not a concern for excavation in the city.

(b) Surface Water Table: During the rainy season, when surface water table rise inside ponds or lakes, this may influence the stability of earth retention system. Seepage of water through the soil and change of consistency of the clayey soil may destabilize the cut. Increase of water content of the soil raise the unit weight of the soil. Saturated unit weight of the soil is therefore considered to account for this effect.

(2) Soil Ingredients: The soil constituents along the tunnel locations are typically formed by Clay, Silt and Sand. Upper clay layer with soft to very stiff consistency is underlain by a sandy layer. The worst soil condition is formed when the surface water saturates the clay and the soil looses its cohesive property.

(3) Structural Load: Different types of Structural load such as Building structure, Electric pole, Boundary wall etc. can mostly influence during the construction of earth retention system. Effects of a building structure which are situated close to the typical route are to be considered in the design of earth retention system to evaluate the effect.

(4) Types of earth retention method: Different types of earth retention methods discussed in Chapter 2 broadly divided mainly into two groups. (1) Temporary (2) Permanent. Cantilever Sheet Pile and Braced Cuts are generally used. Earth pressure on the retaining structures depends on the types of the structure used.

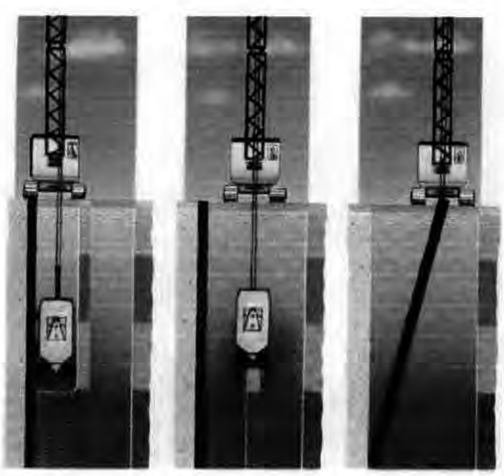
(5) Load from existing Rail: Vibration of the train movement may affect the earth retention system. However, if the existing rail is for away from the cut the effect may be minimized. The effect of the vibration due to train movement is considered in the analysis of earth retention method.

5.2.2 Construction sequence:

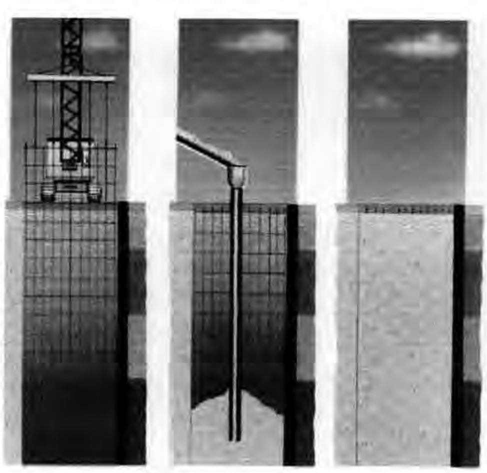
Construction methods for a cut-and-cover tunnel is mainly classified into two types namely "bottom-up" and "top-down" based on the progresses of the excavation. In the 'bottom-up' excavation full-face excavation is first constructed followed by the construction of tunnel from the invert to the crown. In 'top-down' construction, on the other hand, a staged excavation is used supporting the excavation as its progress. Construction using sheet piles involve 'bottom-up' method, while both 'top-down' and 'bottom-up' methods are possible for construction using diaphragm wall. The construction method for braced-cut excavation essentially consists of putting two rows of vertical walls in the soil and suitably propping them against each other by steel struts as the excavation is made. Underground utilities that conflict with the construction are temporarily relocated.

In a top-down construction, when the final excavation level is reached, the subway box is cast and backfilling done to restore the original ground surface as struts are progressively removed. The walls are usually 30 ft apart in the running sections for the proposed tunnel and wider at station sections where the cut is made to accommodate the platform and other services. The excavation depth is near about 50 ft as required for the tunnel. The description of the sequences is given below:

(1) Diaphragm Wall: The diaphragm wall is built in panels of 10ft normally with the full depth of excavation (50 ft). The sides of the diaphragm wall trench are stabilized by bentonite slurry of 1.07 to 1.10 specific gravity. The excavation time for each panel can be 6 to 8 hours using. After the trench excavation the reinforcement cage of the diaphragm wall, is lowered by a crane and concreting is done by tremie method to replace the bentonite slurry which is pump back to vats on the ground for recycling and reuse. The concreting is done continuously and may take 3-8 hours for each panel. The cylindrical form tubes at the end of each panel ended in semi-circular concave shape and the space between two such panels is filled with panels of equal length having semi-circular convex ends.



(1) Diaphragm wall excavation



(2) Dia. wall reinforcement & concreteing

(3) Reinforcement

(4) The finished wall after excavation

Figure 5.1: Basement top-down construction using diaphragm walls

Thus, alternate panels of concave and convex ends form an interlocked system of diaphragm wall. Figure 5.1 shows the Basement top-down construction using diaphragm walls.

(2) Struts: During excavation the diaphragm walls are hold in position by multiple steel struts as per design. In most case three or four struts are found adequate for the 50 ft deep cuts. The struts are placed generally 10 ft center to center intervals along the length.

In the running stretches, the width of the cut is 30 ft and the struts are designed as horizontal compression members resting on brackets at two ends. For wider cuts an intermediate support is provided with a central vertical joist driven into the ground to reduce the effective length of struts. The struts are preloaded by hydraulic jacks immediately after installation so that they are held tightly in position against the diaphragm walls. Excavation can then be done manually and using mechanical grabs. However, under the busy road, manual excavation may not be suitable. Figure 5.2 and 5.3 shows using of struts for construction of tunnel structures.

(1) Steel pipes used as struts
(Twine D, 2004)

(2) I-sections used as struts
(Paya Lebar expressway)

Figure 5.2 : Using struts for construction of Tunnel Box

Figure 5.3: Construction of station building (Heathrow ART)

(3) <u>Box construction and Backfilling</u>: After the diaphragm walls are constructed on both sides of the subway structure, one rail line from the surface and first meter of

earth are removed. While this is underway, train is moved to the other rail line. Temporary arrangements are made during this stage for traffic movement at crossing points. Beams, called "Street Beams" and timber decking are placed across the excavation between the diaphragm walls. To provide accessibility to the nearby areas during construction. Timber decking is placed when excavation progresses to the point where machinery can operate below the street beams. As the excavation proceeds, struts are placed between the diaphragm walls to hold back the earth. Once the bottom of the excavation is reached, a concrete floor is poured. This creates a flat, hard surface to begin the construction of the subway box. Reinforcing steel is then shaped and wired together. The reinforcing steel for the walls sticks out of the floor allowing to tie them together as one box with the bottom of the subway box. The walls are poured, using formwork on the inside and lagging boards on the outside. Formwork is installed to hold up the roof while the steel is placed and concrete is poured. Once the concrete is hard enough, the formwork will slide along so that work on the next roof segment can begin. After all of the subway box is complete, the excavation is filled with compacted earth and the street beams are removed. The utilities are placed in their final position and the road, landscaping, curbs and sidewalks are restored. After the road is fully restored, the subway tunnel is ready for the installation of railway ties, track, signals, electrical equipment and fire protection systems. Figure 5.4 shows the construction procedure of Tunnel Box.

(1) First metre of earth are removed

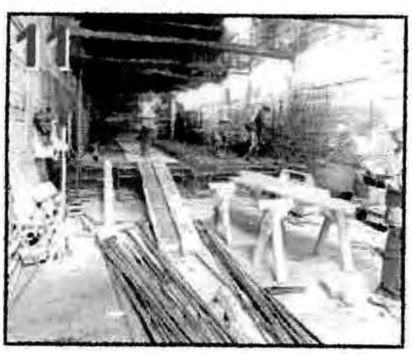
(2) Struts are placed across the excavation

(3) Decking is placed on the street

(4) Timber lagging boards are placed

(5) Struts to hold back the soil

(6) Concrete floor is poured



(7) Reinforcing steel is shaped and wired

(8) Steel for the walls tie with bottom

(9) Walls are poured using formwork

(10) Excavation is filled with earth

(11) Subway tunnel is ready for the installation

Figure 5.4 : Construction sequences of Tunnel Box using Cut and cover method. (Sheppard Subway Project, TTC)

5.3 Design of Earth Retention Methods

For design and construction of underground tunnel by cut and cover method, four conditions have been selected for design of earth retention system. These are arranged in a tabular form given in Table 5.1, based on the six conditions discussed in Chapter 3 (Table 3.12). Case-1 corresponds to 20 ft of clay overlain by sand. This represents a typical condition expected along the site. Case-2 represents an excavation in stiff clay situation. Case-3 correspond to worst soil condition expected (BH 12) where the excavation is in soft clay. Case-4 represents a complex soil condition that may be subjected to a surcharge load from the nearby building structure. Soil retention systems for each of the cases are discussed below. Anchoring of the sheet pile was not considered in the design, since the required space for anchoring may not be available. It would also be difficult to find workers skilled on anchoring in Bangladesh.

Table 5.1: Different cases based on soil type for design of earth retention system.

Case	Soil class	Layer depth (ft)	C_u (psf) (Typical)	φ (degree)	Referance
Case - 1	Clay, C_4	20	2400	–	BH : 10
	Sand, S_4	30	–	35	
Case - 2	Clay, C_5	17	3000	–	BH : 25 and BH : 26
	Clay, C_4	60	2400	–	
	Sand, S_3	23	–	30	
Case - 3	Clay, C_2	65	900	–	BH : 12
	Sand, S_5	35	–	40	
Case - 4	Clay, C_2	14	900	–	BH : 8
	Clay, C_4	16	2400	–	
	Sand, S_3	30	–	30	
	Sand, S_4	20	–	35	
	Sand, S_5	20	Nil	40	

* 1 psf = 0.048 kpa

5.3.1 Case -1

Case 1 corresponds to 20' depth of upper Clay layer, which is the minimum clay layer depth encountered along the tunnel alignment. It was found between Kawran Bazar and Tejgaon industrial area. The clay layer is underlain by a sand layer. The clay layer is C4 class with cohesion varing from 2230 psf to 2520 psf. The typical value of 2400 psf has been used in the design. The underlying sand layer is of S4 class. Designs of each of the retention systems for the soil condition at this site are described below.

(1) Cantilever Sheet Pile

Figure 5.5 shows the lateral earth pressure expected on a sheet pile under the soil condition is Case 1. For the average cohesion value the active earth pressure in the

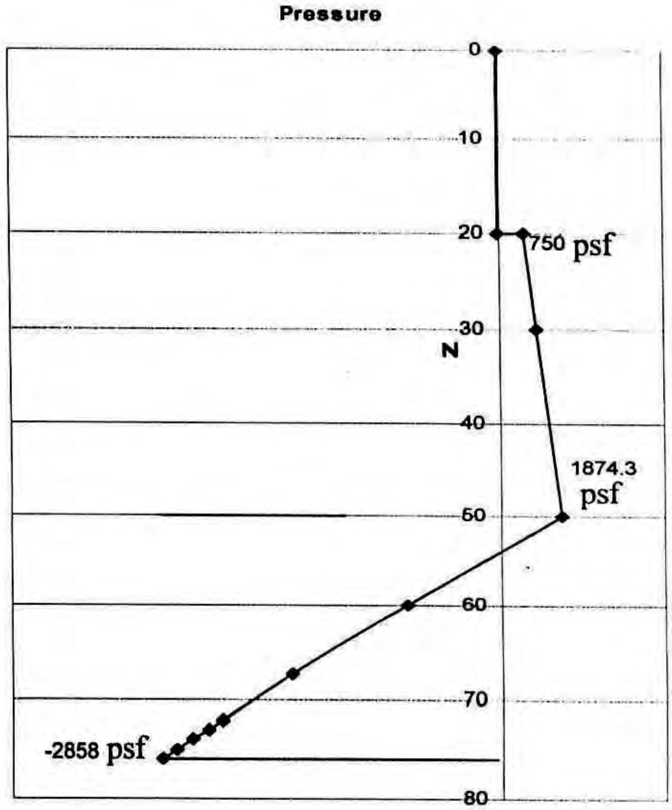


Figure 5.5 : Pressure distribution diagram in Cantilever Sheet Pile (Case 1)

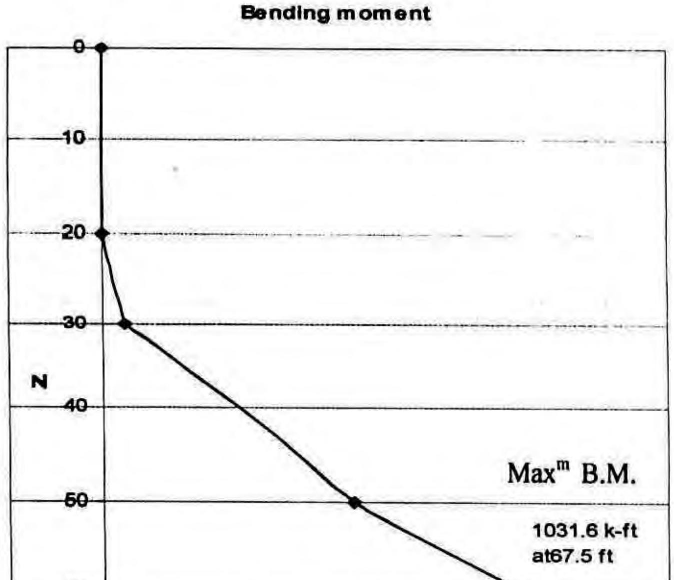


Figure 5.6 : Bending moment diagram in Cantilever Sheet Pile (Case 1)

clay layer zone becomes negative. The negative active earth pressure is ommited to be on the conservative side. The maximum active earth pressure in the sand layer zone is 1874.3 psf at the depth of cut. Maximum bending moment for this active earth pressure occured at 67.5 ft from the top of sheet pile, which is 1031.6 kip-ft. Figure 5.6 shows the bending moment diagram. Total depth of sheet pile need for resisting this moment is 78 ft. For such a high bending section for sheet pile was not available. Thus, the sheet pile is not suitable for earth retention under this soil condition for the proposed tunnel.

(2) Sheet pile with Bracings

In construction of Braced-Cut, struts should have a minimum vertical spacing of about 9 ft or more (Bickle et al. 1997). For braced-cut in clay soils, the depth of the first strut below the ground surface should be less than the depth of tensile crack, Z_c (i.e, Z_c = $2C_u/\gamma$). Initial strut was chosen at a depth of 12.5 ft and all other strut were spaced at a distance of 10 ft for the analysis. The last strut is thus placed at 7.5 ft above the bottom of cut. Peck (1969) developed a earth pressure against braced-cut for different type of soil.

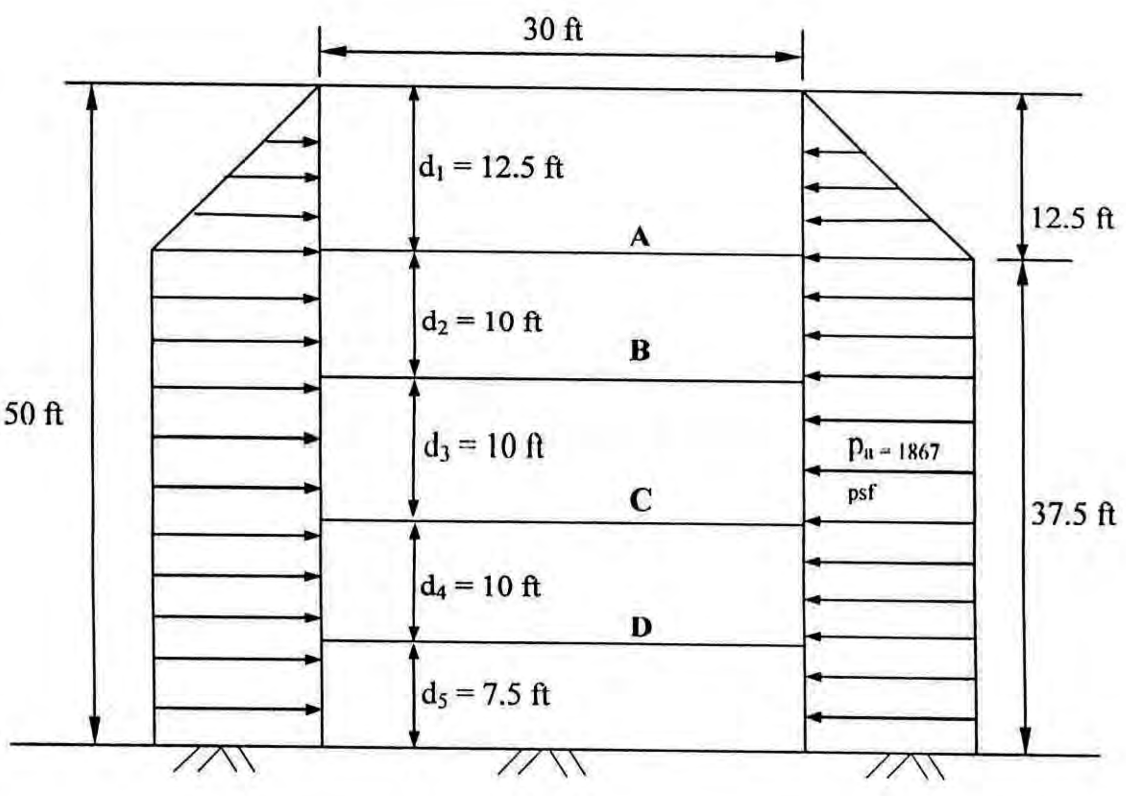


Figure 5.7 : Pressure distribution diagram in Braced-cut (Case 1)

The earth pressure for braced-cut is different from that of retaining wall due to the deformation shape of the braced-cut wall. As discussed in chapter-2, very little wall yielding occurs at the top of the cut, which gradually increases with depth of the excavation. Thus lateral earth pressure is substantially lower toward the bottom of the wall than the Rankin active earth pressure. The lateral earth pressure for medium stiff clay which is similar to the Figure 5.7 when $\gamma H/C_u > 4$. Maximum bending moment has been calculated from the analysis of earth pressure as shown in Figure 5.8. Detail about the calculation of the moments and strut loads are given in Appendix-II. Maximum bending moments for the minimum, average and the maximum value of soil cohesion were calculated. In each case equivalent cohesion for variability soil type was estimated as recommended in Peck (1969) as :

$$C_{ave} = \frac{\gamma_s K_s H_s^2 \tan\varphi_s + (H - H_s)n' q_u}{2H} \tag{5.1}$$

Here, H is the total height of cut, γ_s is the unit weight of sand, H_s is the height of the sand layer, k_s is the a lateral earth pressure coefficient for the sand, (≈ 1), φ_s is the angle of friction of sand, q_u is the unconfined compression strength of clay, n' is a coefficient of progressive failure, (ranging from 0.5 to 1.0; average value 0.75), the average unit weight of γ_a of the layers may be expressed as :

$$\gamma_a = \frac{\gamma_s H_s + (H - H_s)}{H} \tag{5.2}$$

where γ_c = saturated unit weight of clay layer. When several clay layers are encountered in the cut, the average undrained cohesion becomes,

$$C_{ave} = \frac{C_1 H_1 C_2 H_2 + \dots\dots + C_n H_n}{H} \tag{5.3}$$

where $C_1, C_2, \dots\dots C_n$ are undrained cohesion in layers 1,2,.......n. and $H_1, H_2, \dots\dots\dots\dots H_n$ are thickness of layers 1,2,............n. The average unit weight, γ_a is given by :

$$\gamma_a = \frac{\gamma_1 H_1 + \gamma_2 H_2 + ...\gamma_n H_n}{H} \qquad (5.4)$$

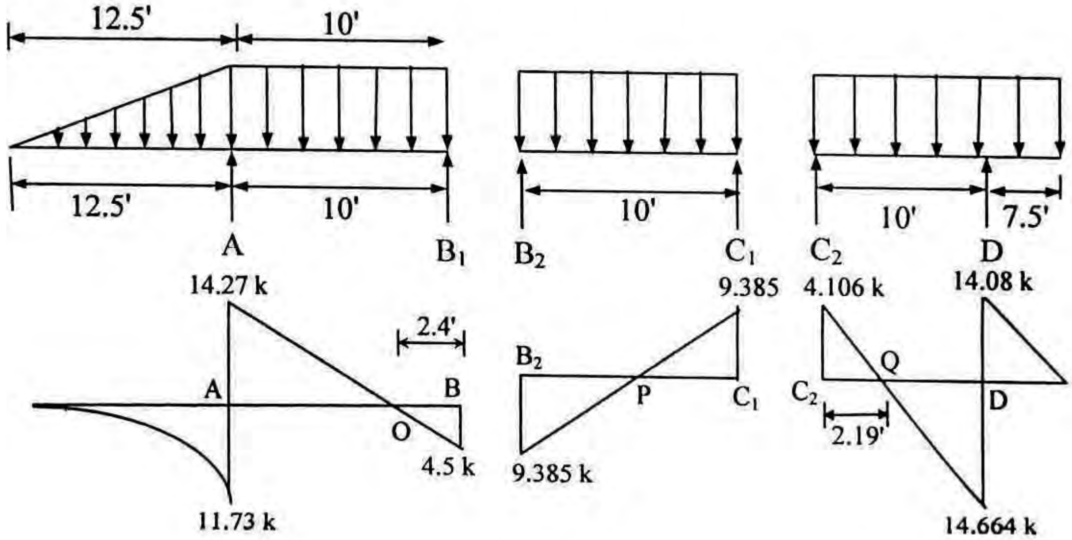


Figure 5.8 : Pressure distribution and Shear force diagram in Braced-Cut (Case 1)

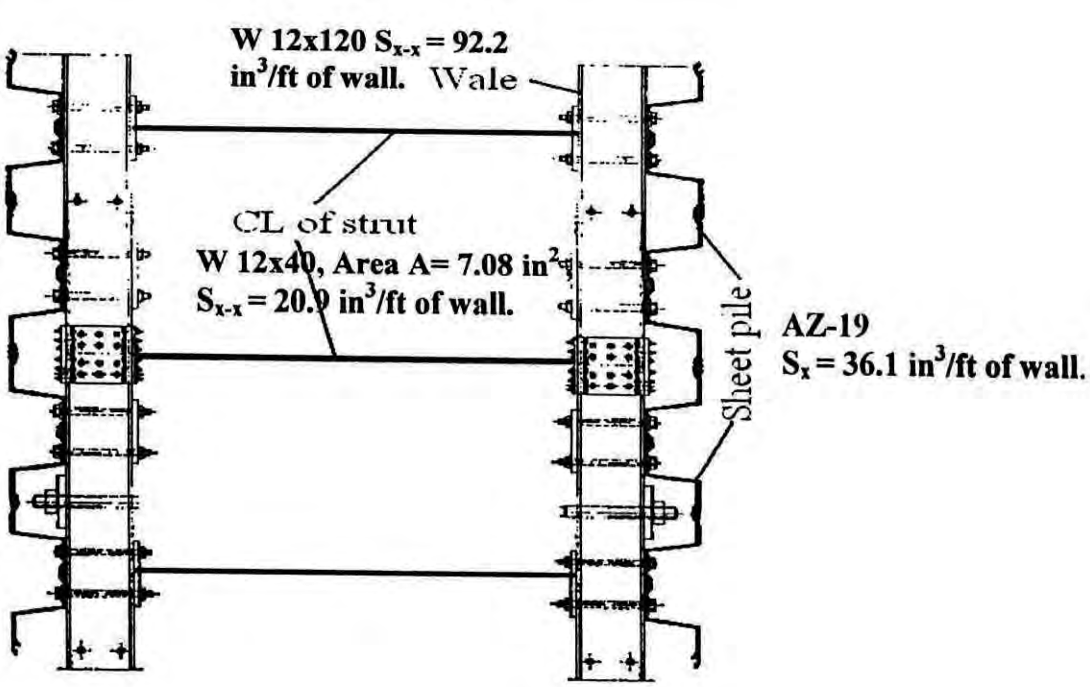


Figure 5.9 : Section Plan of Braced-Cut with Sheet pile (Case 1)

Figure 5.9 shows the sectional plan of Braced-cut for equivalent value of $C_{eq} = 1060$ psf which is found for average undrained cohesion. The maximum moments for the minimum, the average and the maximum values of cohesion were obtained as 73.35 kip-ft, 70.12 kip-ft and 66.89 kip-ft respectively. The corresponding sectional modulus of sheet pile are 36.67 in^3/ft, 35.06 in^3/ft and 33.45 in^3/ft, respectively. Details calculation for moment and section modulus is given in Appendix-II. Thus, for average value of C_u, AZ-19 steel hot rolled sheet pile would be required, which have a sectional modulus S_x = 36.1 in^3/ft of wall. In this case, the placement of strut at levels A, B, C and D are 12.5 ft, 22.5 ft, 32.5 ft and 42.5 ft from top. Wales at these level are W 12x120, W12x65, W 10x77 and W 18x97 and struts at these level are W 12x40, W 8x35, W 8x35 and W 12x45.

(3) Drilled Pile with Bracings

In Drilled or Bored Pile, strut is used as a compression member which is similar to the concept of Braced-Cut. Earth pressure and resulting moment would be the same for drilled pile if the wales and struts with the same spacing are used. Similar spacing are assumed for the bored piles to be used as the earth retention system. Thus, the maximum moment is taken as same as Braced- cut as discussed earlier. The diameter of Bored Pile for the maximum moment was obtained as 16 inch. As main Bar 6 Nos # 8 bar and as stirrup #3 bar @ 1.5 inch c/c would be required. Wale and strut design are similar to Braced Cut. Details calculation for the design are given in Appendix-II. Figure 5.10 shows the Sectional plan and Cross section of Bored Pile.

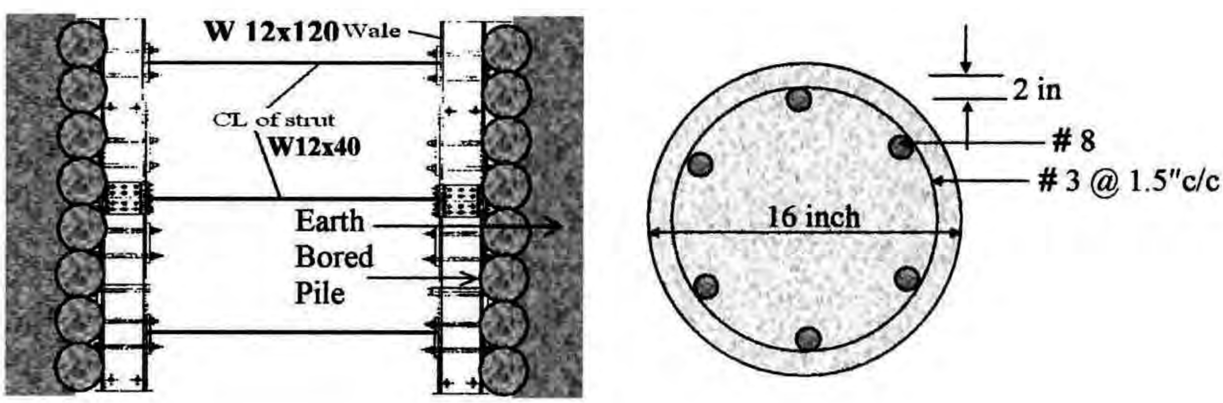


Figure 5.10 : Section Plan and Cross section of Bored Pile with bracing (Case 1)

(4) Diaphragm Wall with Bracings

Diaphragm Wall also uses compression members similar to the concept of Braced-Cut Thus, the earth pressure as those of Braced-cut could be used in the design based on same spacing of struts and wales. Maximum moment is also the same as those of Braced-Cut. The depth of Diaphragm Wall for maximum moment was taken as 12 inch, while the width was obtained as 10 ft. For Main Bar 19 Nos. # 8 bar @ 6 inch c/c and for Stirrup #3 bar @ 12 inch c/c was required. Wale and Strut design are also similar to Braced Cut, as discussed in detail in Appendix-II. Figure 5.11 shows the Sectional plan and Cross section of Diaphragm Wall.

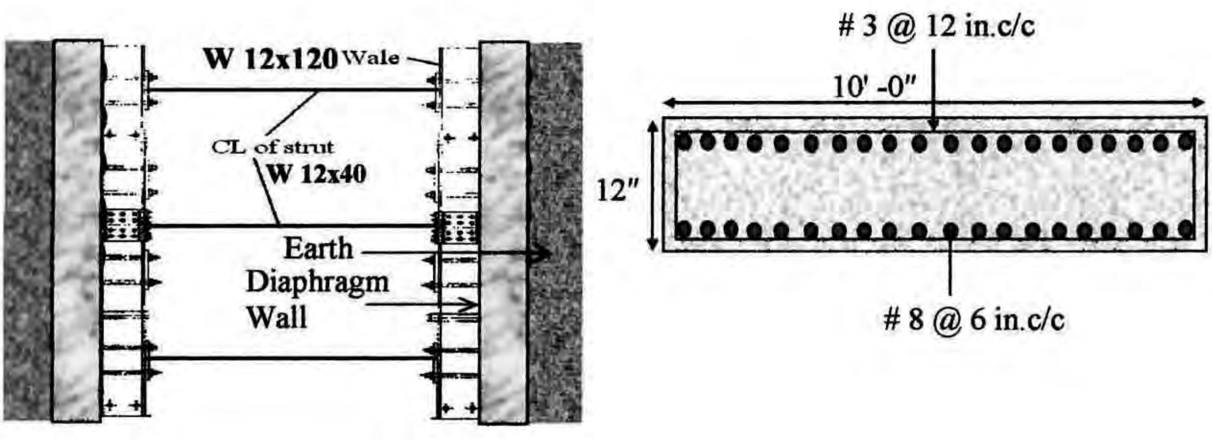


Figure 5.11 : Section Plan and Cross section of Diaphragm Wall with bracing (Case 1)

5.3.2 Case 2

Case 2 correspond to 77' depth of upper clay layer which is one of the maximum clay layer depths expected. The soil condition was encountered at Cantonment Railway Station. Thus the full depth of cut for the tunnel, which is 50 ft, would lie within the clay layer. The clay layer is underlain by a sand layer. Of the clay layer, top 17 ft consists of C5 class clay and the remaining 60 ft is of C4 class clay. The underlying sand layer is S4 class. Designs of each of the retention system for the soil condition at this site are described below.

(1) Cantilever Sheet Pile

For the soil condition described as Case-2 (Table 5.1), earth pressure against a sheet pile is shown in Figure 5.12, The negative earth pressure is again neglected here. Thus, the earth pressure up to a depth of 34 ft is zero. The minimum active earth pressure in the

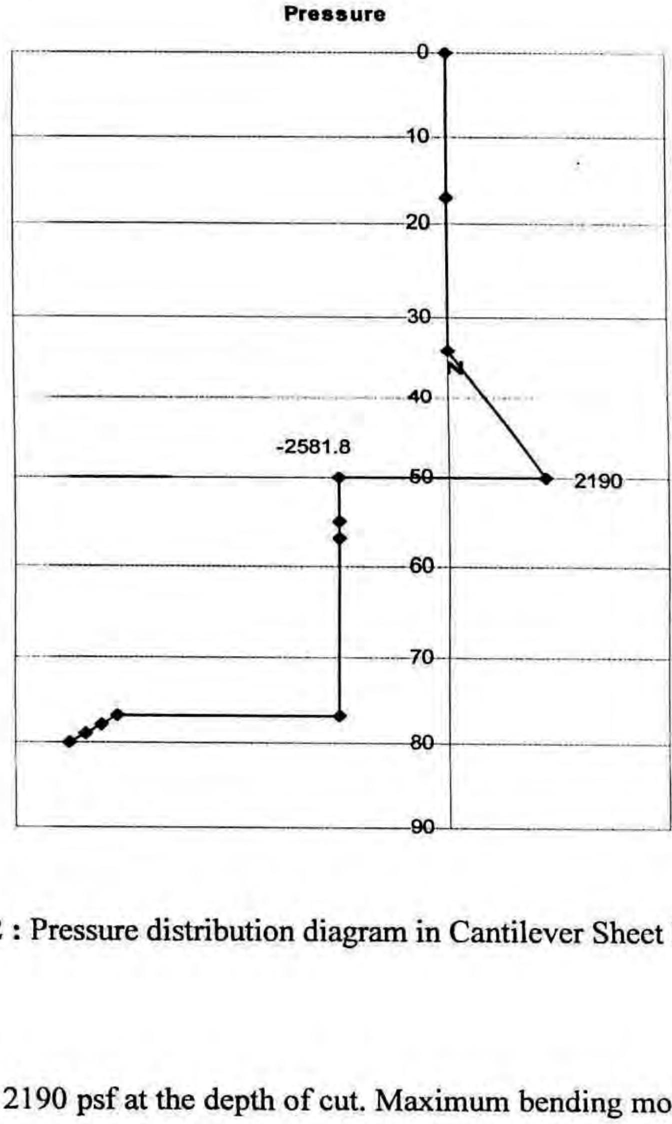


Figure 5.12 : Pressure distribution diagram in Cantilever Sheet Pile (Case 2)

sand layer zone is 2190 psf at the depth of cut. Maximum bending moment for this active earth pressure is occur at 56.8 ft from the top of sheet pile, which is 200 kip-ft. Figure 5.13 shows the bending moment diagram. Total depth of sheet pile need for resisting the earth pressure is 58 ft. If ASTM A-572 GR 50 steel is used, sectional modulus required to resist the maximum moment is 75 in^3/ft. Sheet pile section AZ 46, have sectional modulus of 85.47 in^3/ft which can be used. Figure 5.14 shows the sectional plan of a Sheet Pile that can be use for earth retention using sheet pile under the soil condition of Case-2.

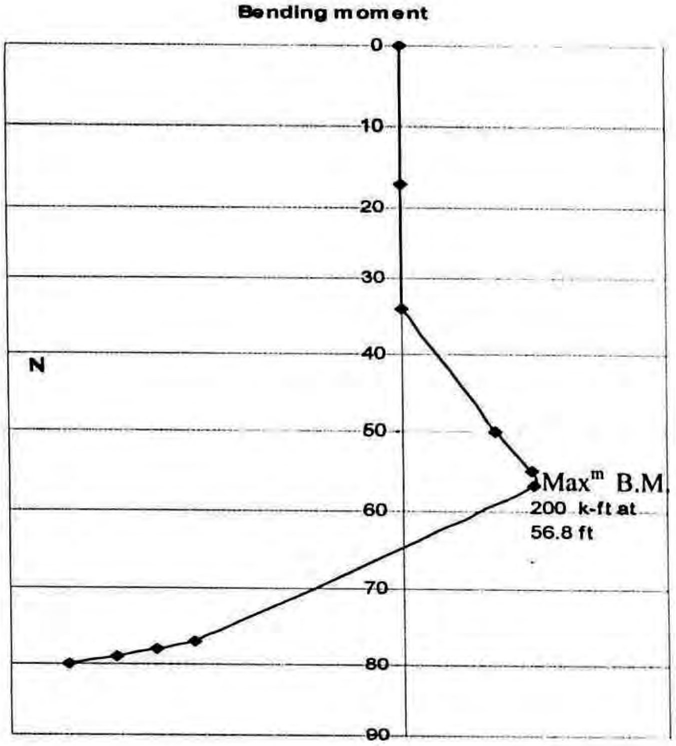


Figure 5.13 : Bending moment diagram in Sheet Pile (Case 2)

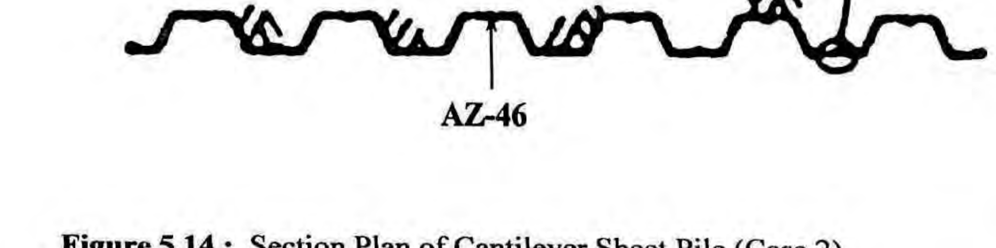


Figure 5.14 : Section Plan of Cantilever Sheet Pile (Case 2)

(2) Sheet pile with Bracings

Same spacing of strut as assumed in Case-1 is also assumed for the bracd-cut in the soil condition of Case-2. Figure 5.15 shows the earth pressure against the braced-cut wall according to Peck (1969). Maximum bending moment has been found from the analysis of pressure diagram given in Figure 5.16. Earth pressure corresponding to $\gamma H/ C_u < 4$

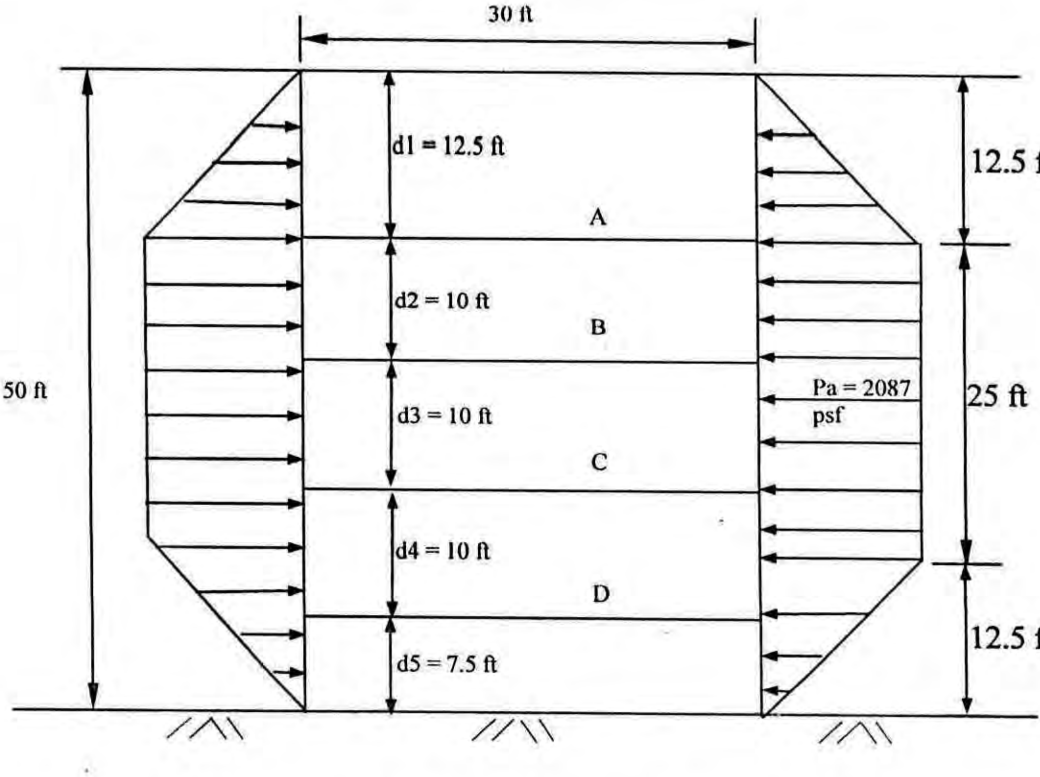


Figure 5.15 : Pressure distribution diagram in Braced Cut (Case 2)

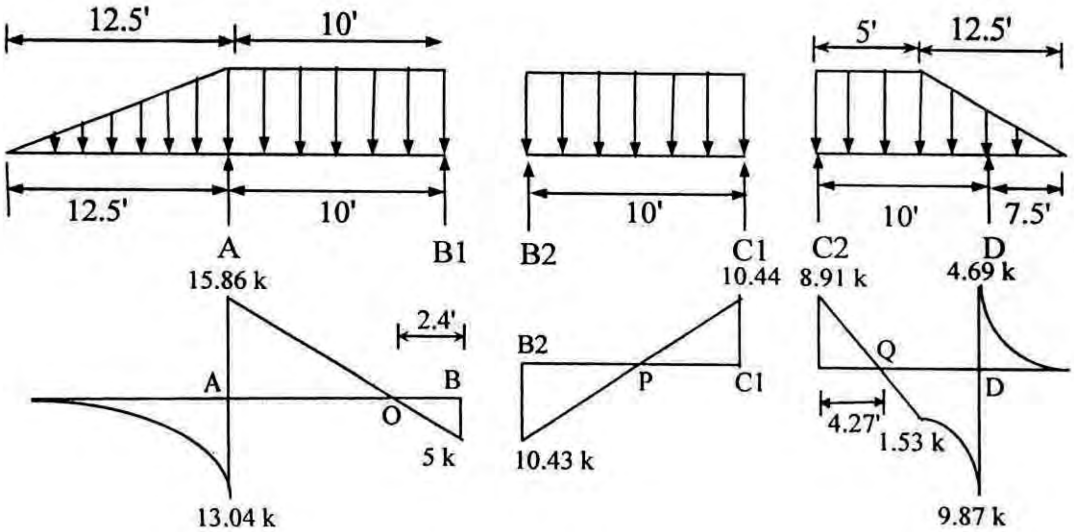


Figure 5.16 : Pressure distribution and Shear force diagram in Braced Cut (Case 2)

condition is used. For equivalent cohesion calculated according to Peck (1969), $\gamma H/ C_u$ = 2.62, which is less than 4. Earth pressure for the minimum, average and the maximum values of C_u expected for this soil condition was examined, which is similar for this three values of C_u. For the average value of the cohesion maximum bending moment was calculated to be 54.34 kip-ft, which occurred between strut at A and B. The required sectional modulus is 27.17 in^3/ft. Thus AZ-25 can be used that gives S_x = 45.7 in^3/ft of wall. Wales at those levels of strut in both sides can be used of sizes W 12x136, W 12x72, W 14x82 and W 16x57. Maximum moment hold by the wale is 361.37 kip-ft/ft of wall at level A. Sectional modulus require for this moment is S_{x-x} = 180.7 in^3/ft of wall. Sectional modulus, S_{x-x} = 186 in^3/ft of wall was provided based on the wale sections. Struts of sizes W 10x49, W 10x30, W 10x39 and W 10x26 are recommended for bracing. Maximum force on by the strut was estimated to be 289.1 kip. Area require for this force is A= 14.17 in^2 , which is obtained using the above sections that provided an area of A = 14.31 in^2. Figure 5.17 shows the designed section of the braced-cut.

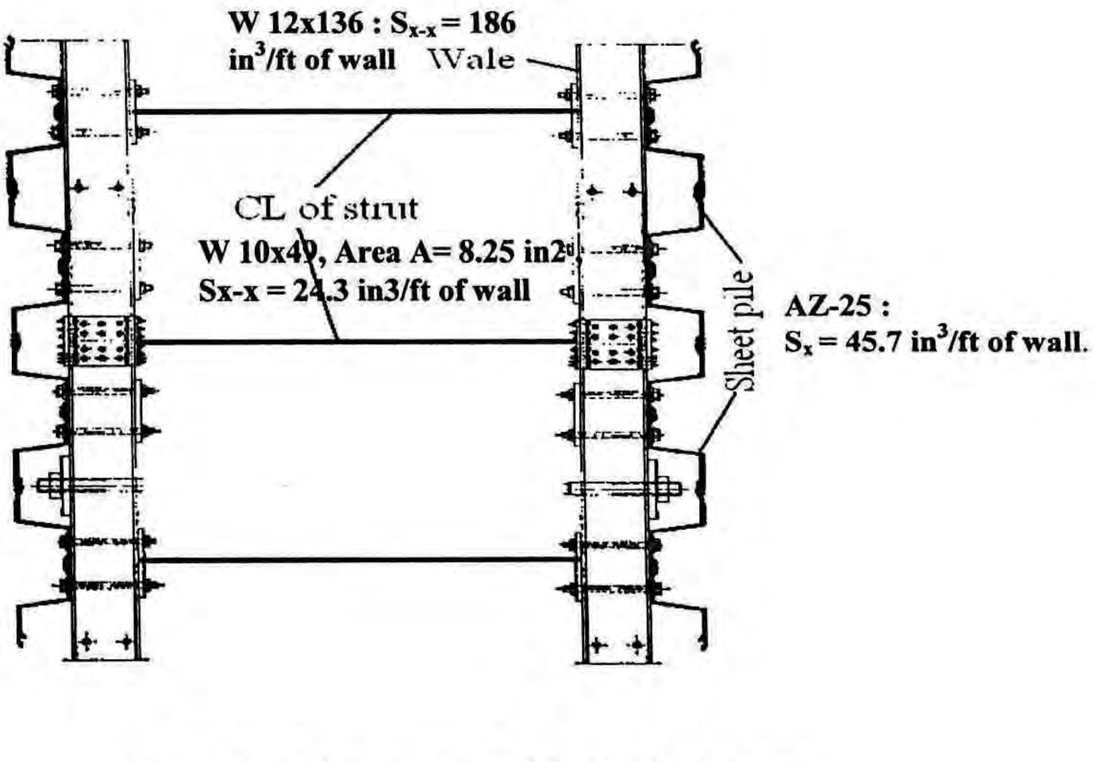


Figure 5.17 : Section Plan of Braced-Cut. (Case 2)

(3) Drilled Pile with Bracings

As discussed earlier Drilled pile or Bored pile, uses compression member as in Braced-cut and have same pressure distribution. The maximum moment is also same as that for Braced-cut. The diameter of Bored Pile for maximum moment was obtained as 16 inch. Main reinforcement of 6 Nos # 8 bar and Stirrup of #3 bar @ 1.5 inch c/c was obtained from design calculation for the Bored pile. Wale and Strut design are similar to Braced Cut. Same section of wales and strut as those obtained for Braced-cut is recommended for bored pile. Section Plan and Cross section of Bored Pile is shown in Figure 5.18.

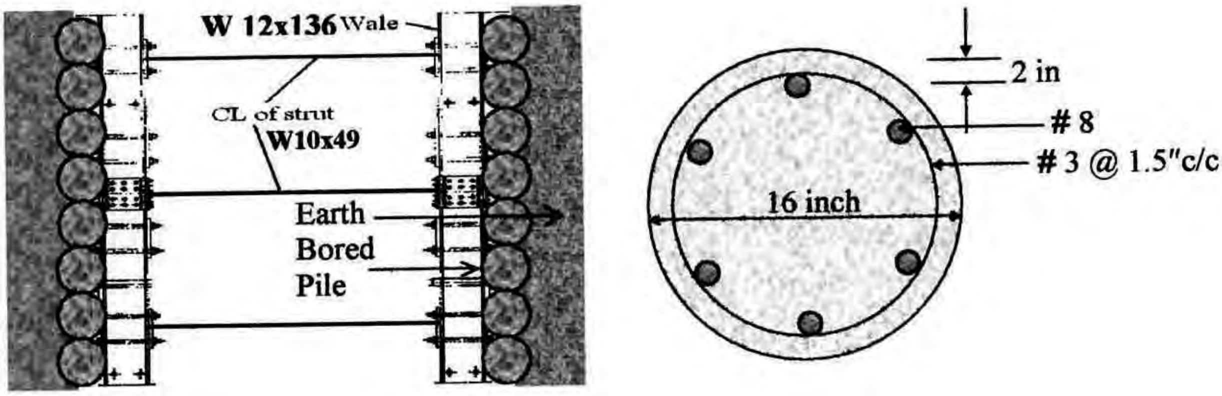


Figure 5.18 : Section Plan and Cross section of Bored Pile (Case 2)

(4) Diaphragm Wall with Bracings

For the Diaphragm wall, the depth of Diaphragm wall for maximum moment is calculated to be 12 inch and while width is assumed to be 10 ft. Main Bar of 19 Nos. # 8 bar @ 6 inch c/c and stirrup of #3 bar @ 12 inch c/c were obtained as the reinforcements. Wale and Strut design are similar to those used for Braced Cut. Sectional plan and Cross section of Diaphragm wall is shown in Figure 5.19 for this case of ground condition.

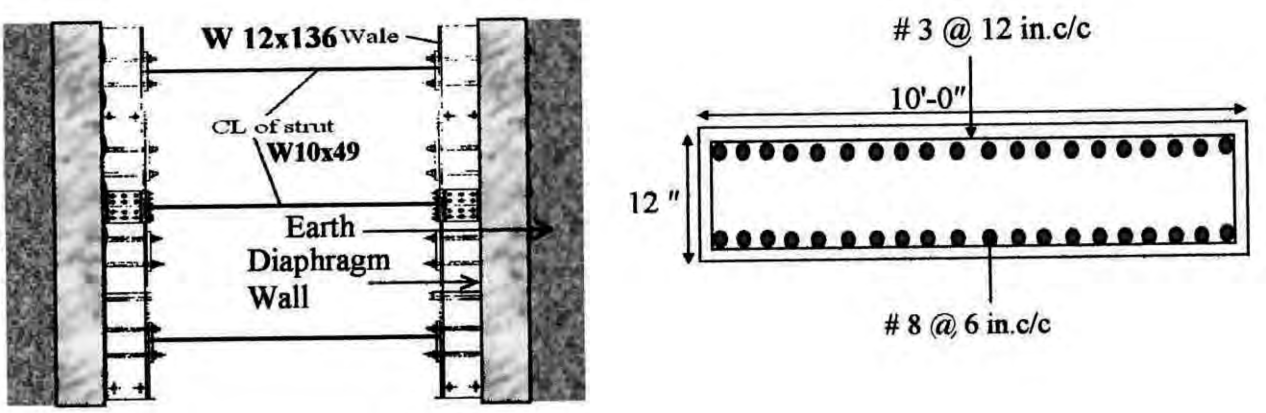


Figure 5.19 : Sectional Plan and Cross section of Diaphragm Wall (Case 2)

5.3.3 Case -3

Case 3 correspond to the worst case with 65' depth of soft upper clay layer.This soil was encountered at Tejgaon. Thus the full depth of cut for the tunnel, which is 50 ft, would lie within the soft clay layer. There are 65 ft layer is C2 class clay, underlain by S5 class sand. Designs of each of the retention system for the soil condition at this site are described below.

(1) Cantilever Sheet Pile

Cohesion of the clay for this site varies from $C_u = 1008$ psf to $C_u = 1512$ psf, with the average being 1260 psf. Figure 5.20 shows the earth pressure distribution on a retaining wall for $C_u = 1260$ psf. As shown in the Figure, the positive active earth pressure is shown omitting negative active earth pressures. Total penetration depth is required below the Depth of Cut is 30 ft to resist the earth pressure. Maximum moment which is generated for the average value of C_u is 2368 ft-kips at 74 ft below the ground level as

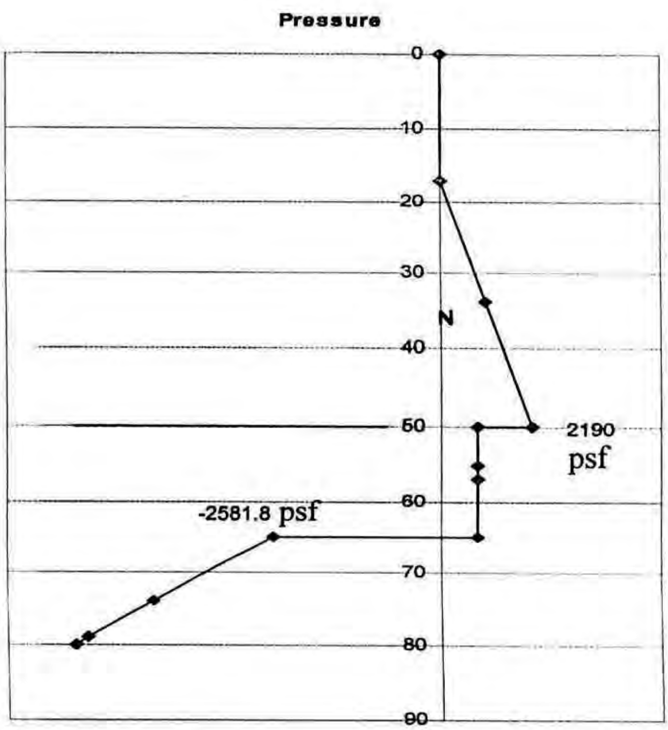


Figure 5.20 : Pressure distribution diagram in Sheet Pile (Case 3)

shown in Figure 5.21. This required sectional modulus of S = 28416/32 = 888 in^3/ft, which is not obtainable using available sheet pile sections. Sheet Pile method thus will not apply to this soil condition.

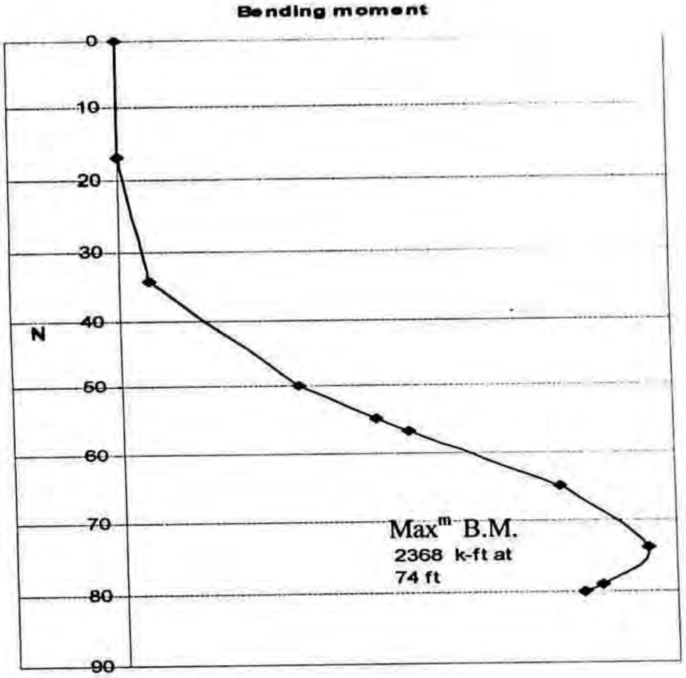


Figure 5.21 : Bending moment diagram in Sheet Pile (Case 3)

(2) Sheet pile with Bracings

Peck(1969) developed earth pressure for Braced-Cut analysis, in medium stiff clay soil which is similar to the Figure 5.22 (when $\gamma H/ C_u > 4$). Maximum bending moment has been found from the analysis of pressure diagram given in Figure 5.23. For the minimum, the average and the maximum value of C_u maximum moments were 56.1 kip-ft. The maximum moment occurred at strut level D. The required sectional modulus for the maximum moment is 28.05 in^3/ft,which obtained using Use AZ-17 section. The section provided a sectional modulus of $S_x = 31.0$ in^3/ft of wall. Maximum moment hold by the Wale is 381.8 kip-ft/ft of wall at level D. Section W 21x93 with $S_{x-x} = 192$ in^3/ft of wall was used to resist the moment. Maximum load to be carried by the Strut is 305.44 kip.

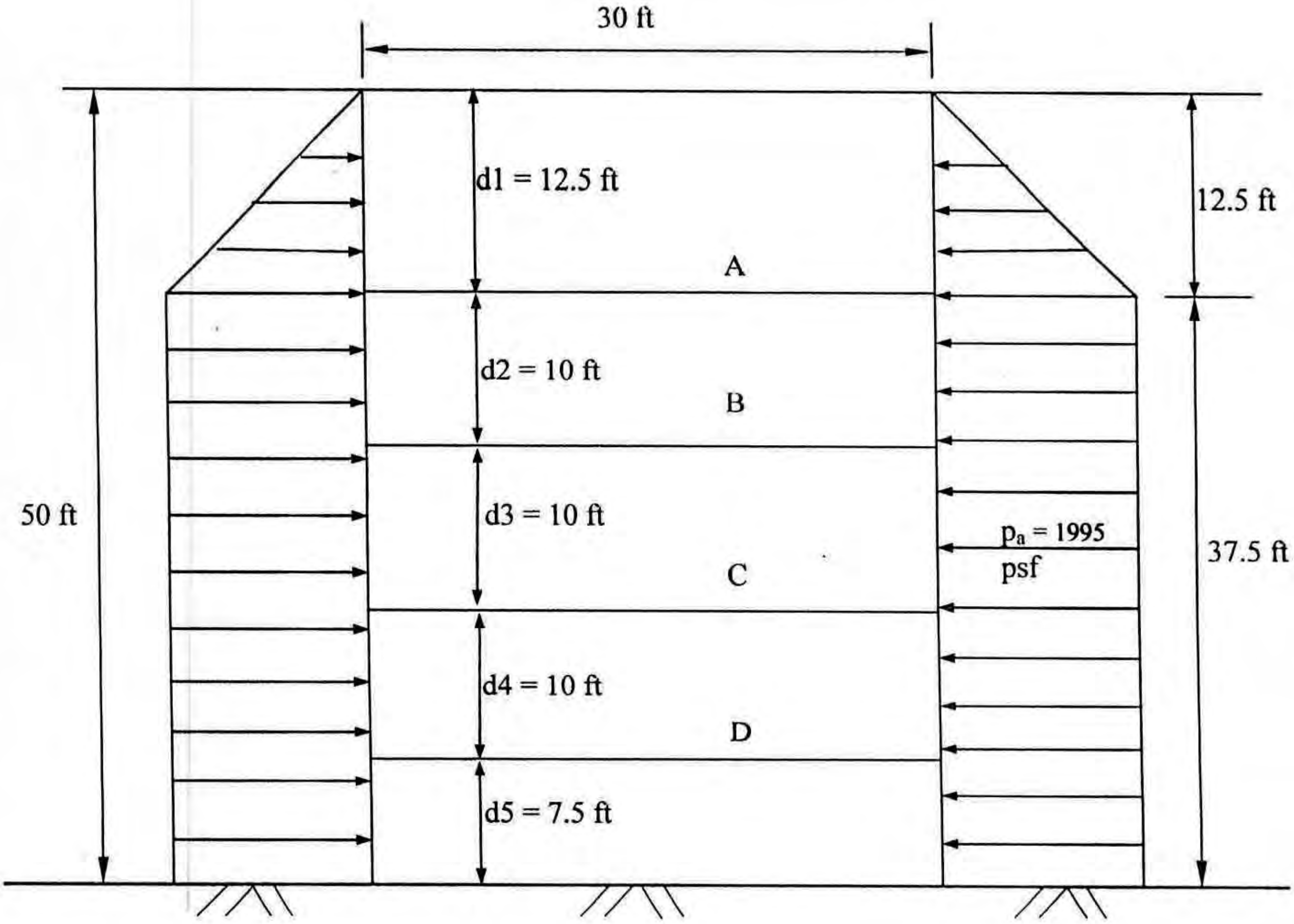


Figure 5.22 : Pressure distribution diagram in Braced Cut (Case 3)

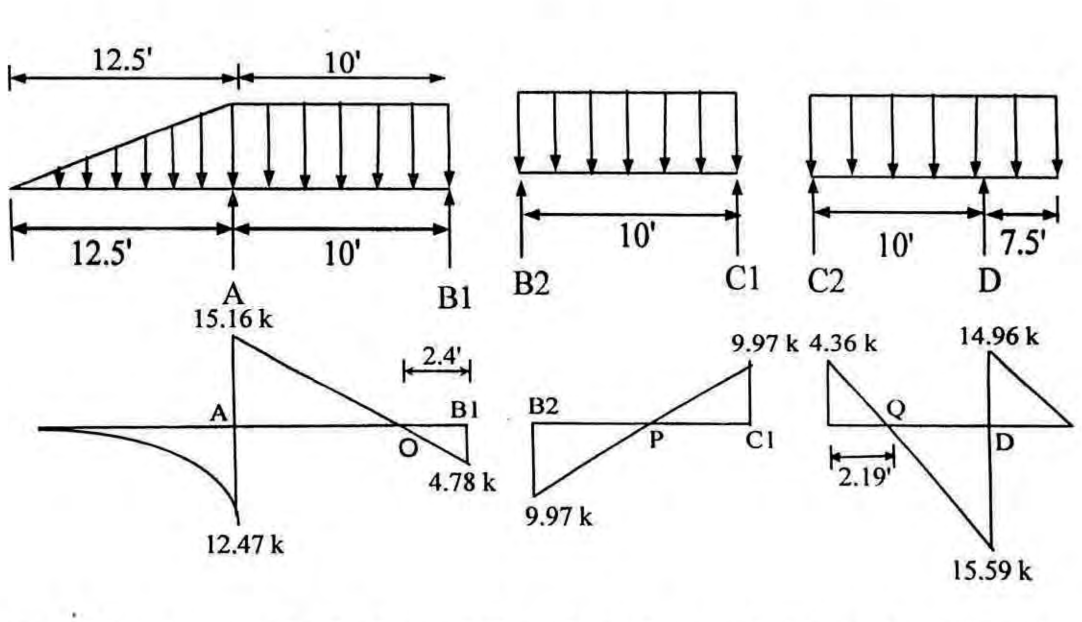


Figure 5.23 : Pressure distribution and Shear force diagram in Braced Cut (Case 3)

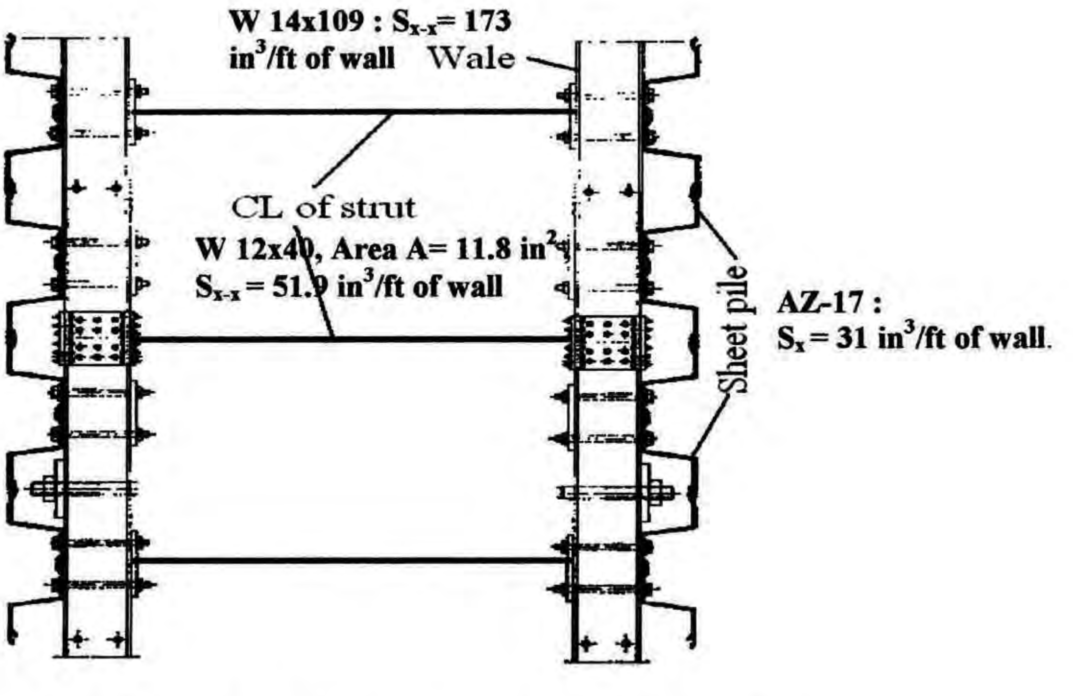


Figure 5.24 : Section Plan of Braced-Cut (Case 3)

A strut section of W 12x45, with area A= 13.2 in^2, and section modulus S_{x-x} = 58.1 in^3/ft of wall was used to carry the load. Figure 5.24 shows the Braced-Cut sectional plan recommended for the site.

(3) Bored Pile with Bracings

The diameter of Bored Pile for maximum moment in case of average value of C_u is obtained as 16 inch for this type of soil. Main bar of 6 Nos # 8 bar and stirrup of #3 bar

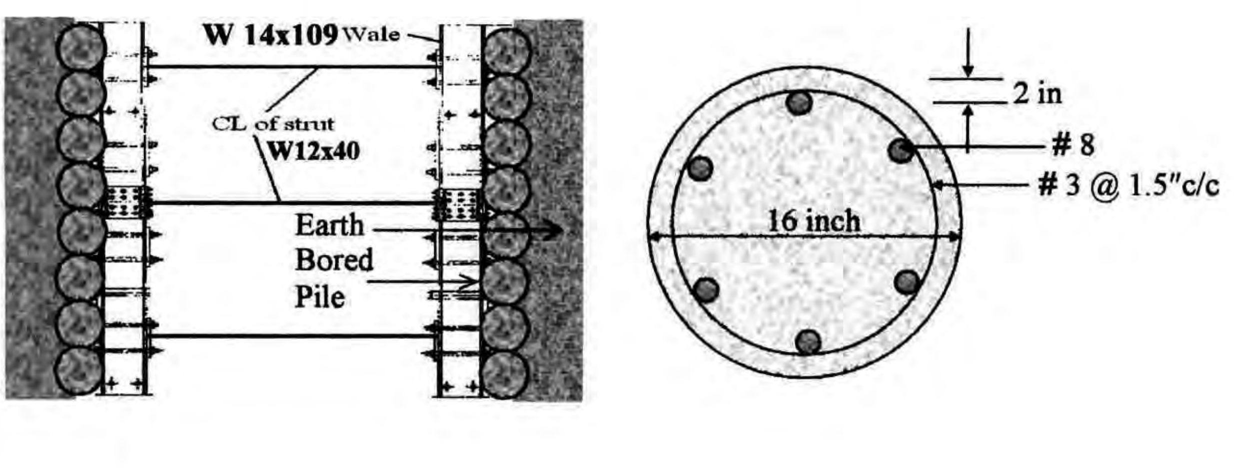
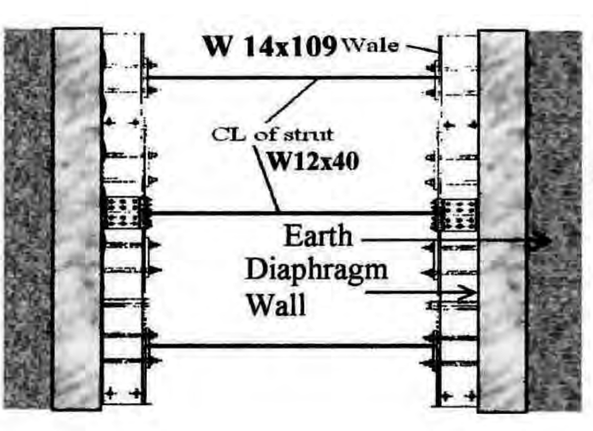


Figure 5.25 : Section Plan and Cross section of Bored Pile (Case 3)

@ 1.5 inch c/c was obtained as the reinforcement. Wale and Strut design are the same as those used for Braced Cut. Figure 5.25 shows Sectional plan and Cross section of Bored Pile in this case of soil.

(4) Diaphragm Wall with Bracings

The depth of Diaphragm Wall for maximum moment has been obtained as 12 inch in case 3 type of soil condition. The width of the wall was used as 10 ft. Main Bar of 19 Nos. # 8 bar @ 6.0 inch c/c and Stirrup of #3 bar @ 12 inch c/c would be required as the reinforcement. Detail calculation for the design are shown in Appendix II. Wale and Strut design are the same as Braced Cut. Figure 5.26 shows sectional plan and cross section of Diaphragm wall to be used.



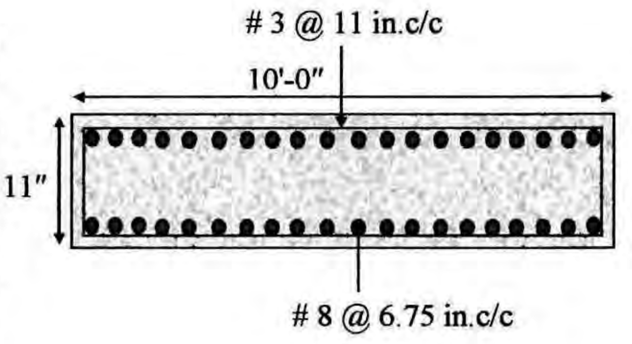


Figure 5.26 : Sectional Plan and Cross section of Diaphragm Wall (Case 3)

5.3.4 Case -4

Case 4 include a case where a nearby Building Structures beside tunnel route is to be encountered. Chapter 1 of this thesis discussed the nearby structure along the assumed tunnel alignment. One of the worst case of nearby structure has been analyzed to investigate the effect of the structural surcharge on earth retaining system. As discussed earlier, Malibag and Kawran bazaar area is very crowded and possesses building structures nearby to the rail line. However, major structures are located beyond the right of way of the rail line, resulting in a distance of structure over 10 to 20 ft away from the rail line. Chapter 1 revealed that some structures are very close (as close as 2 ft) to rail line, which is expected to be within the right of way. These structures should be removed and therefore not considered here as a challenge. A five storied building located at a distance of 8 ft from the proposed cut was considered to study the effect of building load on the earth retention system. Boussinesq solution was used to calculate the earth pressure to be exerted on a retaining structure from the building weight. A building of 66 ft x 44 ft size was assumed with load per floor of 300 psf resulting in a total building pressure of 1500 psf. The front face of the building was assumed at a distance of 8 ft. It was revealed that lateral pressure exerted on the retaining due to the building is very less. The maximum pressure from building was obtained as 40 psf. This also indicates that from nearby train line, if exists at a distance of 8 ft or so, may not affect the performance of the earth retention system. Further of the lateral pressure and the design of earth retention systems discussed in the following sections. Soil conditions for the analysis was taken as those encountered in the Kawran bazar area. For this site top 16 ft layer is C2 class clay, then 14 ft is C4 class clay and then the soil is S3 class sand, representing a complex soil condition.

(1) Sheet Pile with Bracings

Figure 5.27 shows the earth pressure distribution on a retaining wall for average value of C_u including the effect of building surcharge. It is revealed that there is no effect on the lateral earth pressure due to surcharge. As discussed above the maximum lateral pressure due to surcharge was 40 psf, which is much less than the active earth pressure. Thus the effect is not revealed in Figure 5.27 due to the scale effect. The maximum bending moment on the sheet pile under this condition was obtained as 1108.9 ft-kips at 76 ft depth which is shown in Figure 5.28. Detail calculation for the design of the sheet pile is given in Appendix-II. Using ASTM A-572 GR 50 steel with F_b = 32 ksi, required sectional modulus of sheet pile section is calculated to be 415.84 in^3/ft. However, a sheet pile with such a high section modulus is not generally available. Thus, a sheet pile wall is not possible for this condition.

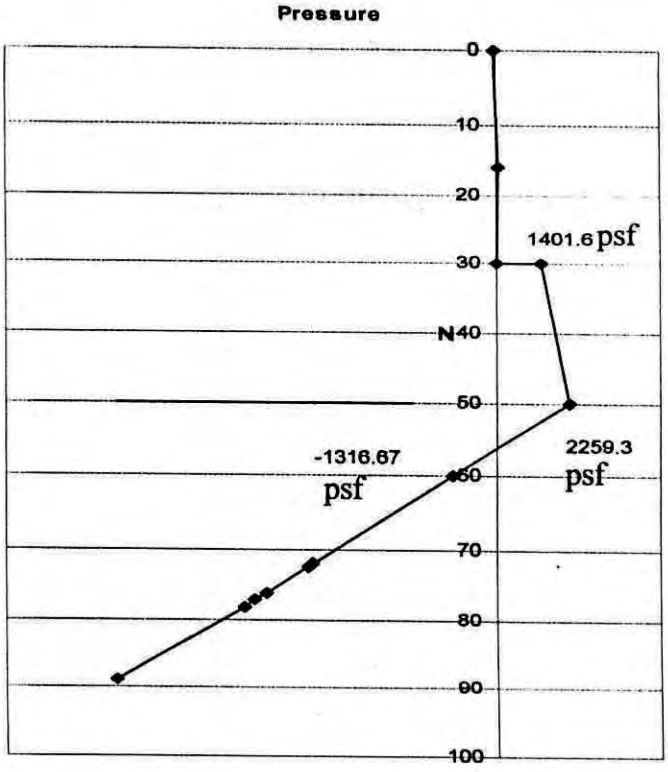


Figure 5.27 : Pressure distribution diagram in Sheet Pile (Case 4)

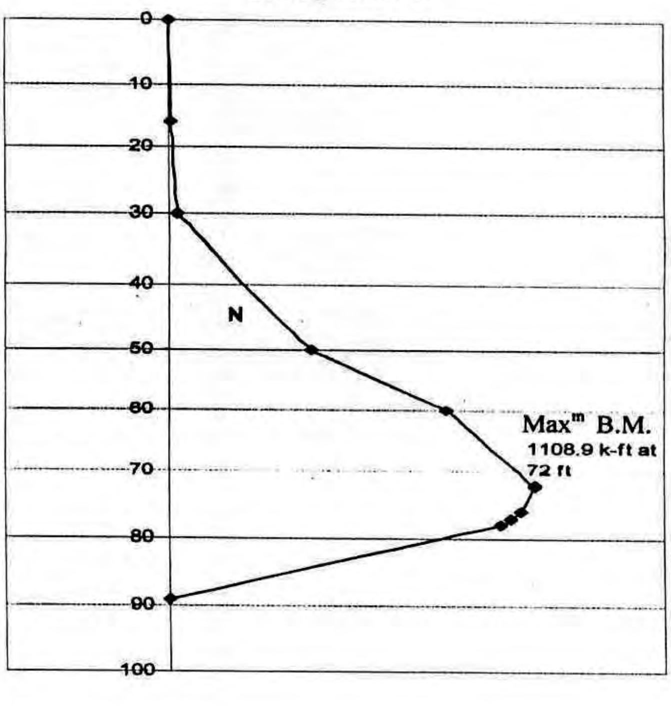


Figure 5.28 : Bending moment diagram in Sheet Pile (Case 4)

(2) Sheet pile with Bracings

Figure 5.29 shows the pressure distribution on a braced-cut wall according to Peck (1969). The surcharge effect due to the building was added as an additional lateral pressure. However, the lateral pressure due to the surcharge was very less compared to the earth pressure. The maximum moment, for the braced cut wall was obtained as 101.83 kip-ft which occurred at level D in Figure 5.30. Thus, sectional modulus requirement of sheet pile is S_x = 50.92 in^3/ft which could be obtained. Using AZ-28 section that provided the sectional modulus as 51.24 in^3/ft of wall. Maximum moment in the wale is

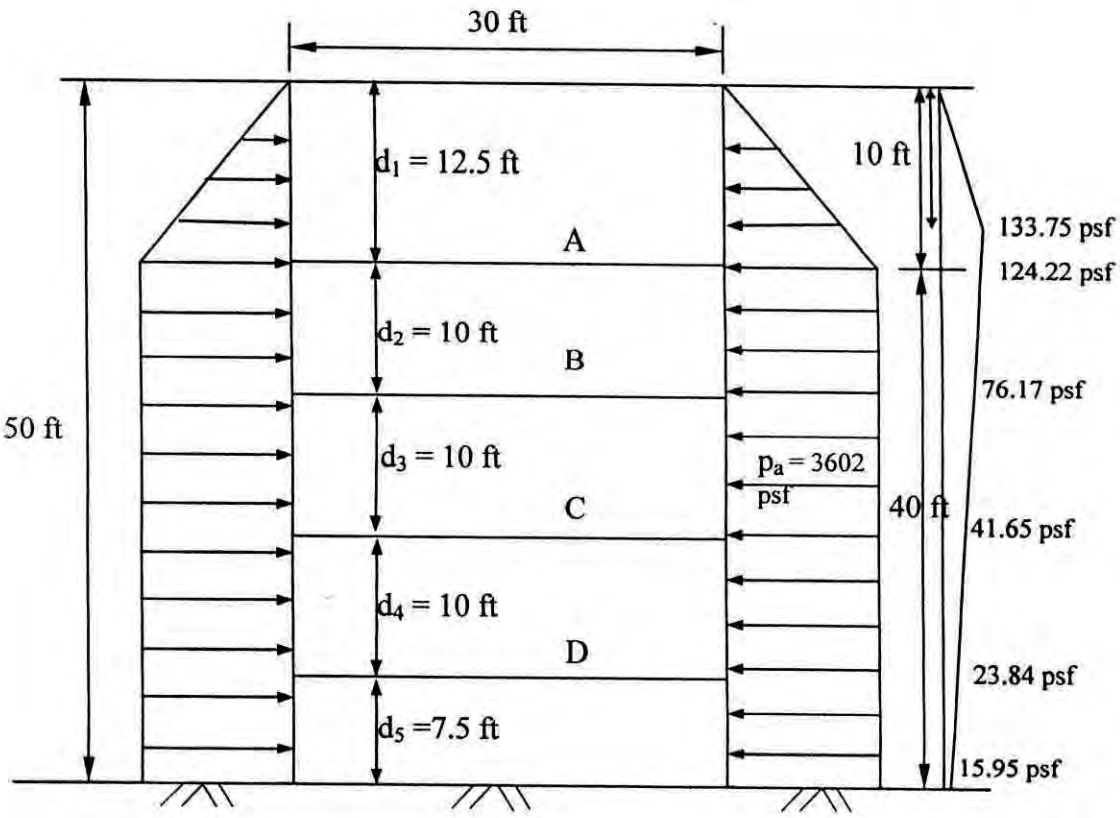


Figure 5.29 : Pressure distribution diagram in Braced Cut. (Case 4)

693.875 kip-ft/ft of wall at level D. Section W 12x252 with sectional modulus of 353 in^3/ft of wall was used to resist the moment. Maximum force on the strut was 555.1 kip. Section W 12x79 with area A= 23.2 in^2 and section modulus of 107 in^3/ft of wall could be used as the strut. Braced-Cut sectional plan for average value of C_u is given in Figure 5.31.

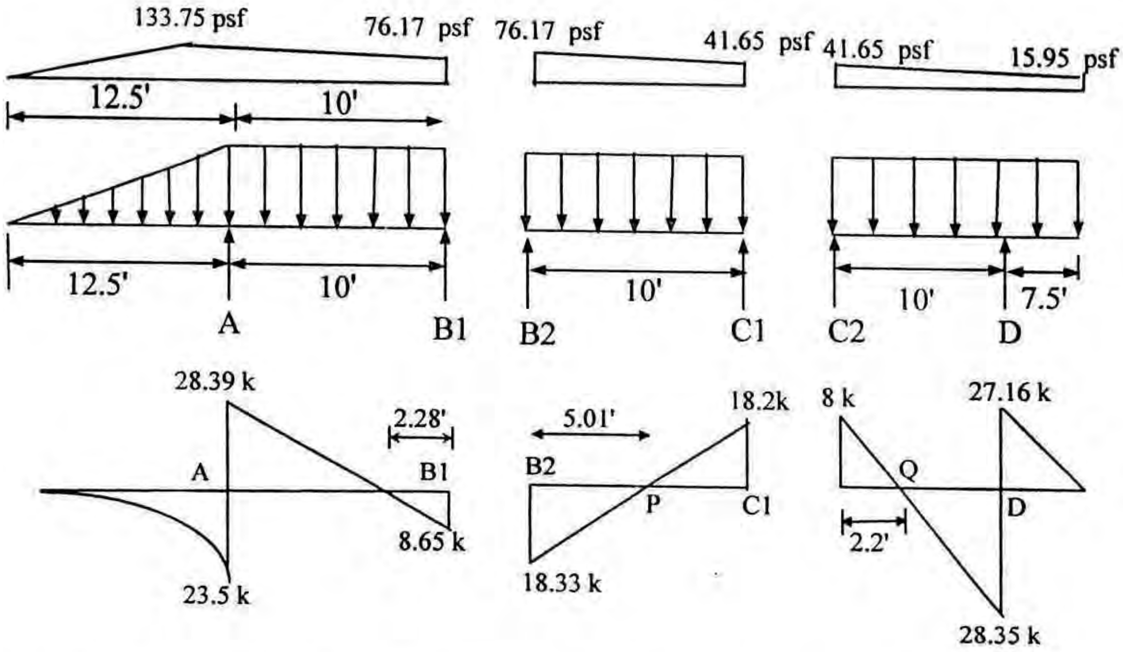


Figure 5.30 : Pressure distribution and a Shear force diagram in Braced Cut . (Case 4)

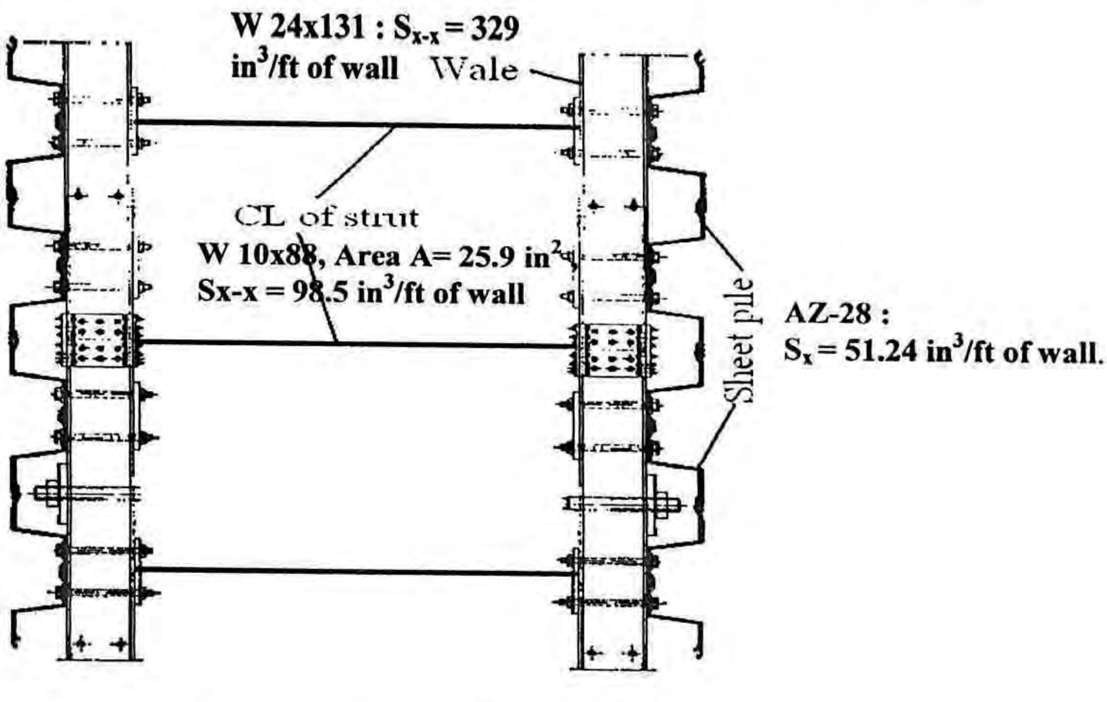


Figure 5.31 : Section Plan of Braced-Cut. (Case 4)

(3) Drilled Pile with Bracings

Diameter of Bored pile for maximum moment discuss with reference to braced-cut was obtained as 22 inch. Design reinforcement are main bar of 10 Nos # 8 bar and stirrup of

#4 bar @ 2 inch c/c. Wale and Strut design are again similar as those for Braced-cut. Figure 5.32 shows designed sectional plan and cross section of Bored pile for Case- 4.

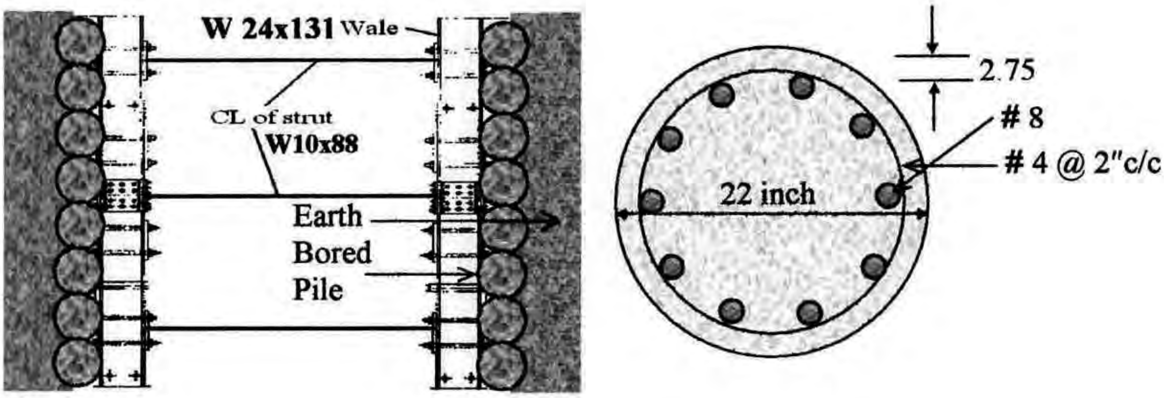


Figure 5.32 : Section Plan and Cross section of Bored Pile . (Case 4)

(4) Diaphragm Wall with Bracings

For the same maximum moment discussed above, depth and width of diaphragm wall was obtained as 16 inch. The width was agained assumed as 10 ft, respectively. Main bar reinforcement was obtained as 25 Nos. # 8 bar @ 4.5 inch c/c and stirrup was obtained as #3 bar @ 16 inch c/c. Wale and strut design are similar as those for Braced-cut. Figure 5.33 shows sectional plan and cross section of Diaphragm wall.

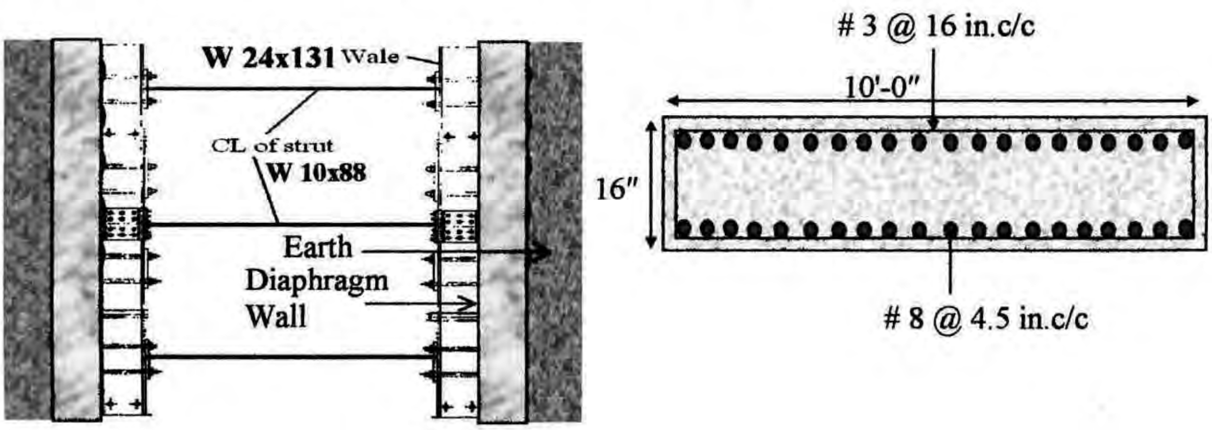


Figure 5.33 : Sectional Plan and Cross section of Diaphragm Wall . (Case 4)

If the effect of surcharge due to nearby structure is neglected, the maximum moment would be 101.32 k-ft. which almost the same as those with the surcharge. This implies the nearby structure has no influence on the design of retaining system.

5.4 Design considering Construction sequence

The construction sequence sometimes influences the design of earth retention system. As discussed earlier, the support provided to a braced cut are different for 'top-down' construction and bottom-up construction For a bottom-up construction excavation is completed from top to bottom while bracings are provided during removal of earth. The construction sequence for a retaining structure at four point bracing can be divided into three stages. In stage 1 excavation completed prior to installation of first bracing. Thus, sheeting acts as cantilever wall. The active earth pressure up to the depth of first bracing was found negative. Thus, the maximum wall moment for stage 1 is only for water pressure. For stage 2, first bracing was installed and excavation is completed up to the level of second bracing are wall moment is gradually calculated assuming a hinge at the level of end bracing (NAVFAC). For the final stage, the analysis is performed based on the earth pressure propose in Peck (1969). Here, hinge point is considered at the every strut level except the first strut (NAVFAC). For Case 1 of soil condition, the pressure diagram corresponding to stage 1, 2 and 3 are given in Figure 5.34. The summary for

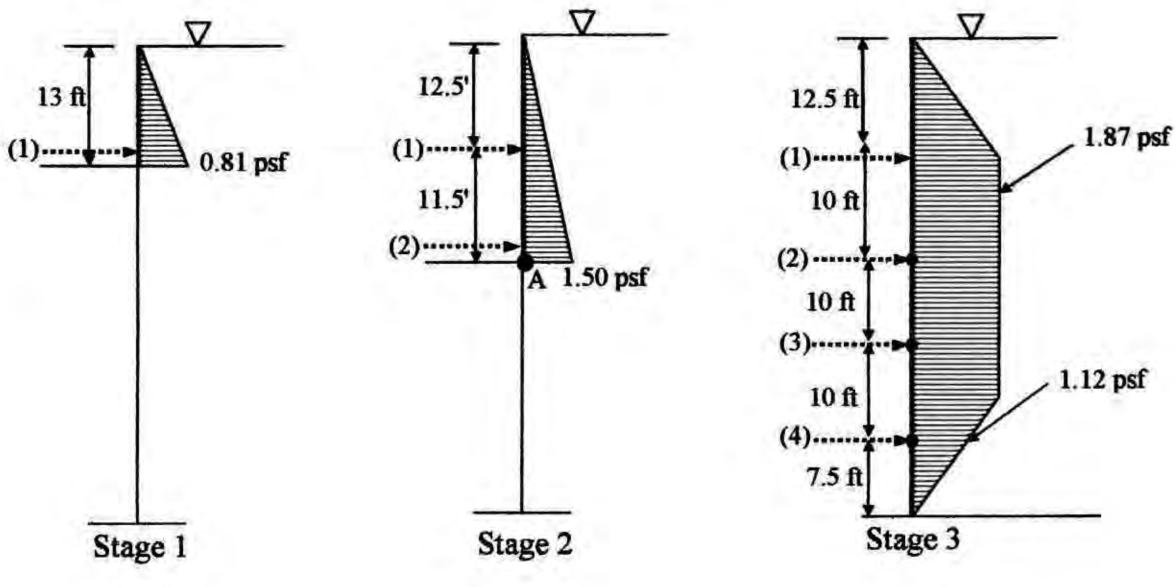


Figure 5.34 : Pressure distribution diagram for Case 1

strut loads and moments at different construction stages are given in Table 5.2. Details calculations of the strut load and wall moment are given in Appendix-II.

Table 5.2 : Summary for strut loads and moments at different construction stages.

Construction Stage	Strut loads (kip)		Moments (ft-kips)
I	-		46
II	R(1) = 14.4		10.57 between (1) and (4)
Final	R(1) = 26	R(2) = 13.89	52.79 between (3) and (4)
	R(3) = 13.49	R(4) = 28.74	

For Case 3 the pressure diagram corresponding to stage 1, 2 and 3 are given in Figure 5.35

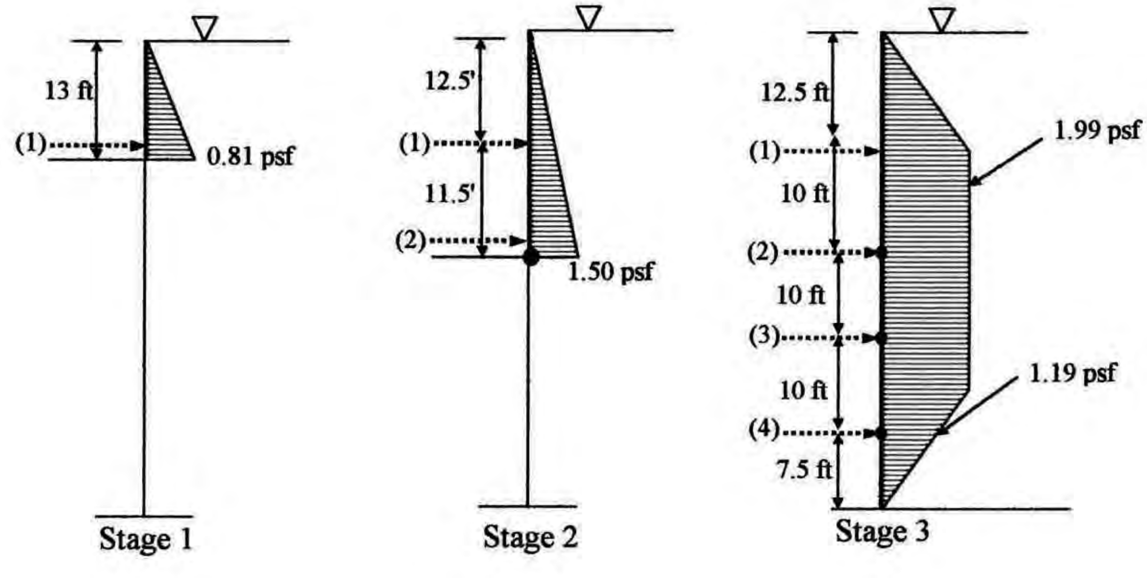


Figure 5.35 : Pressure distribution diagram for Case 3

Table 5.3 : Summary for strut loads and moments at different construction stages.

Construction Stage	Strut loads (kip)		Moments (ft-kips)
I	-		46
II	R(1) = 14.4		10.57 between (1) and (4)
Final	R(1) = 27.63	R(2) = 14.75	56.11 between (3) and (4)
	R(3) = 14.34	R(4) = 30.55	

It revealed that final stage of construction usually governs the strut load and wall moment. Strut and wall design in section 5.3 were based on final stage of construction.

5.5 Design of Tunnel Structure

Chapter 4 discusses the type of tunnel structure assumed for Dhaka city. Design of the structure is presented here considering the worst soil condition. Load on the structure was used as those recommended in Bickel et al (1997) Details of Tunnel Structure design are given in Appendix –II. Three conditions are generally considered during design of tunnel structure as discussed below in.

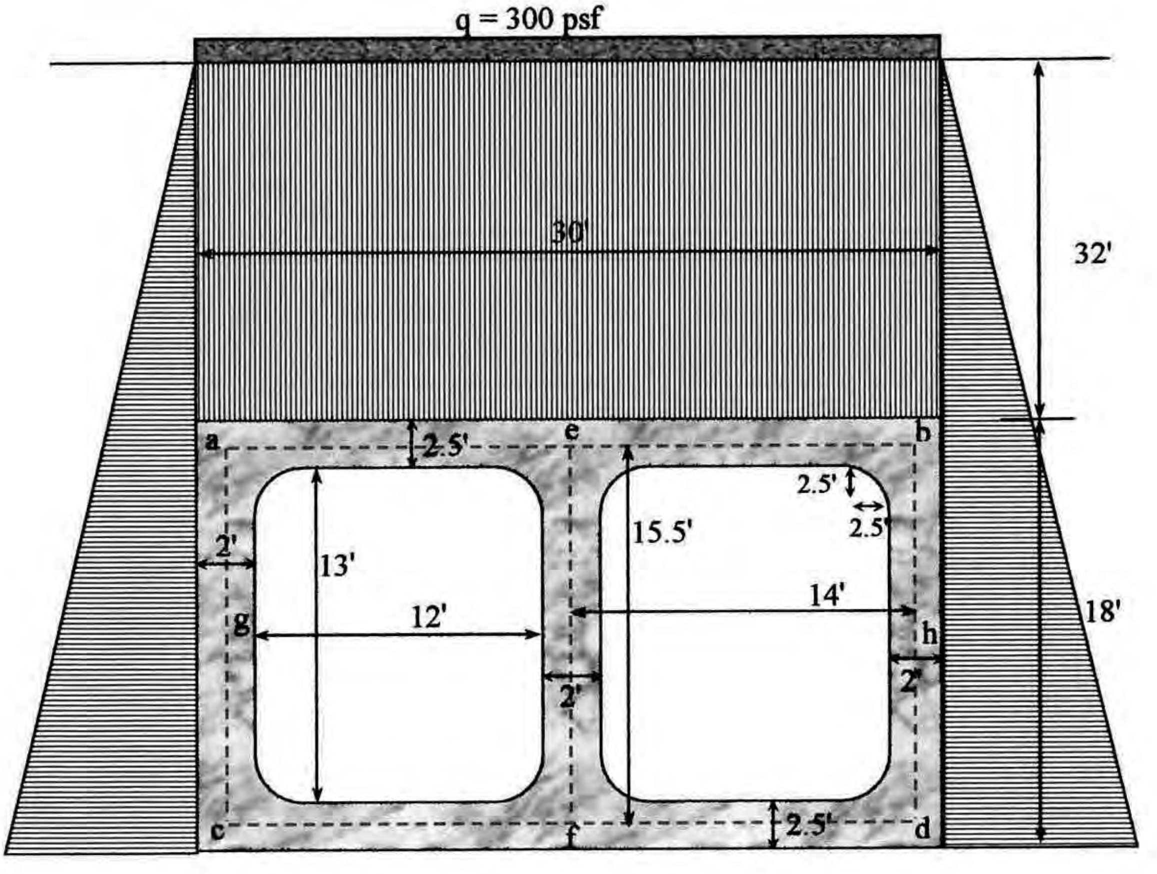


Figure 5.36 : Vertical & Horizontal pressure effect on Tunnel Structure.

5.5.1 Long-Term & Balanced Condition

Incase of long-term and balanced condition, full vertical roof load including earth cover load, roof slab load, surcharge and vertical invert reaction (which include total roof load and weight of walls) is considered as vertical stresses. The maximum long-term horizontal pressure is considered as horizontal stresses in soil. Figure 5.36 shows the Vertical & Horizontal pressure on Tunnel Structure. The earth stress around the tunnel structure for Long-Term and Balanced Condition are given in Figure 5.37 respectively.

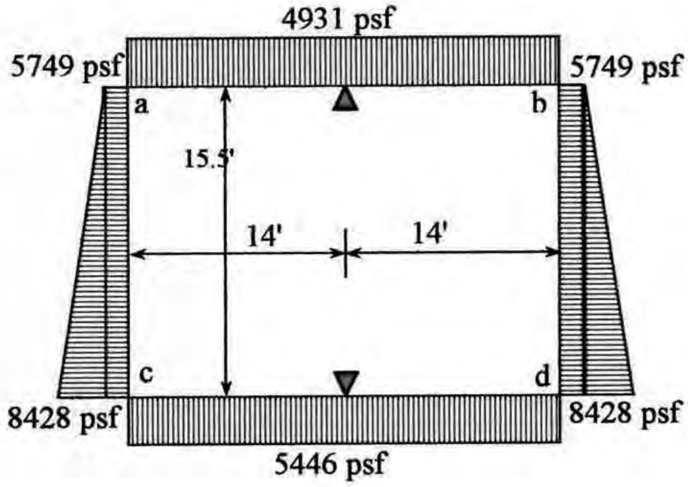


Figure : 5.37 Pressure distribution for Long-Term and Balanced Condition

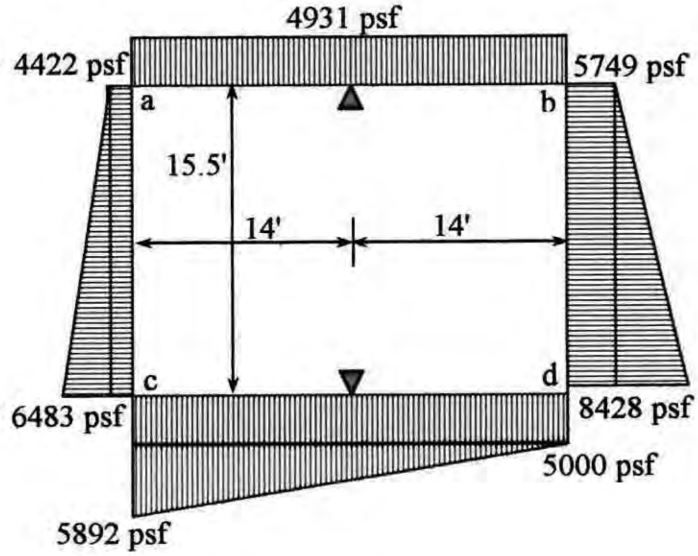


Figure : 5.38 Pressure distribution for Long and Short Term and Un-Balanced Condition

5.5.2 Long and Short Term and Un-Balanced Condition

Incase of long and short term and un-balanced condition full vertical roof load and modified invert reaction as required to account for unbalanced horizontal loading is considered as vertical stresses and the maximum long and short term horizontal pressure is considered as the horizontal stresses. Pressure distribution diagram for Long and Short Term and Un-Balanced Condition is given by Figure 5.38.

5.5.3 De-Watered and Balanced Condition

In De-Watered and Balanced Condition full vertical roof load and vertical invert reaction is considered as vertical stresses and De-Watered active earth pressure is considered as horizontal stresses. Pressure distribution diagram for De-Watered and Balanced Condition is given in Figure 5.39.

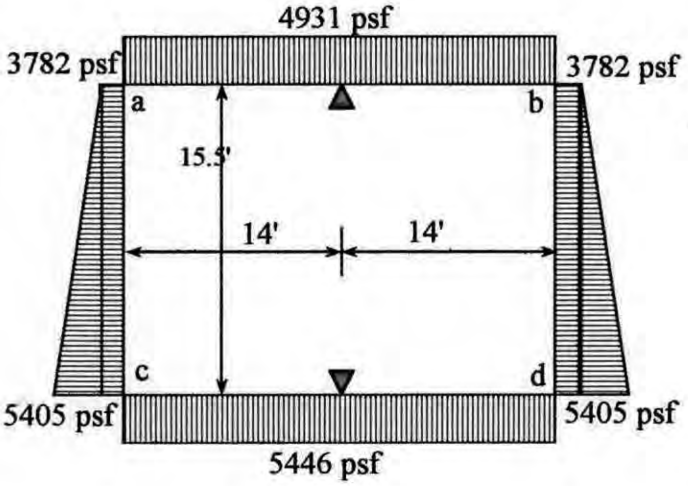


Figure : 5.39 Pressure distribution for De-Watered and Balanced Condition

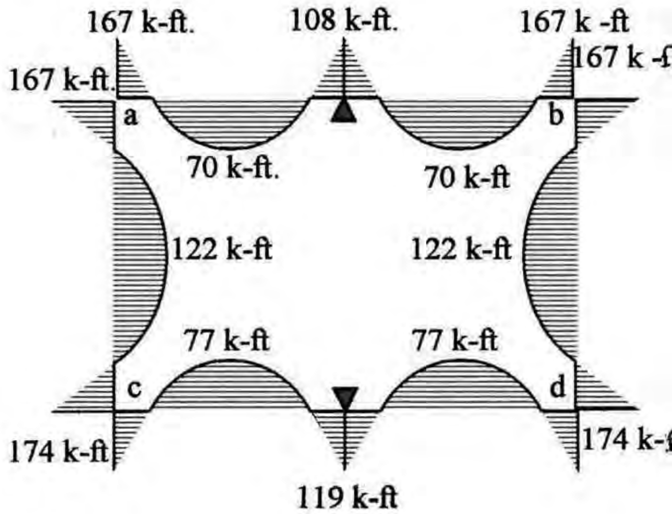


Figure: 5.40 Moment distribution diagram for Long-Term and Balanced Condition

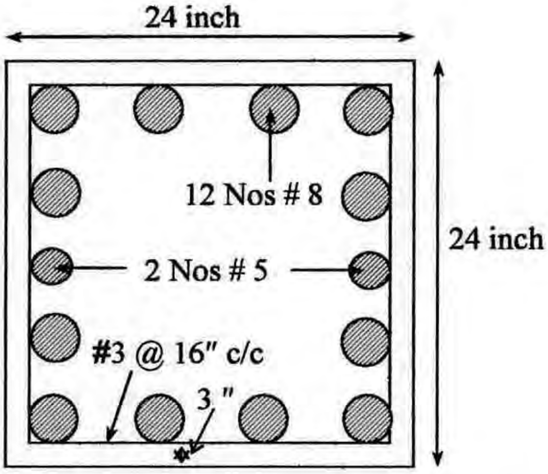


Figure 5.41 : Cross section of Middle Tied Col^{m}

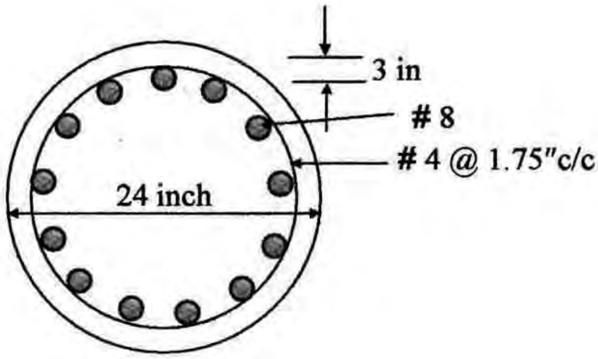


Figure 5.42 : Cross section of Middle Spiral Colm.

On the basis of the pressures acting on Tunnel Structure, it is considered that the Long-term effect will govern the design. Figure 5.40 shows moment distribution around a

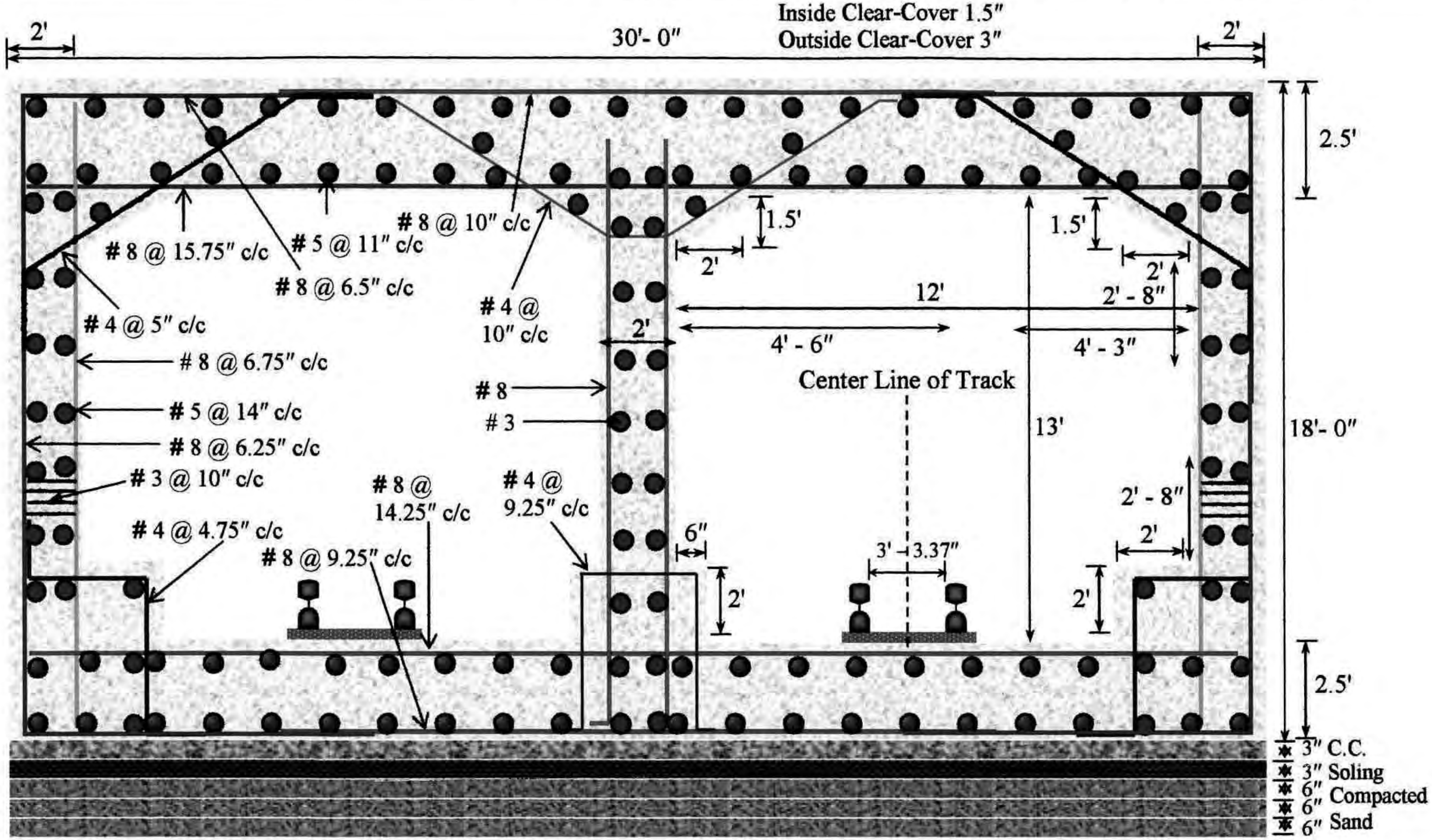


Figure 5.43 : Cross-Section of Standard Double Box Section. (Not to scale)

tunnel structure under long term and balanced condition calculate using the approximate methods. The maximum moment is obtained at joints c and d as 174 ft-kips. For this moment a section with 14 inch effective depth is required. However, a 27 inch effective depth was chosen in order to reduce the reinforcement requirement. If convert the middle wall as a tied column or spiral column then the size of column is given by Figure 5.41 and Figure 5.42. Details design of Tunnel Structure is given by Figure 5.43

5.6 Tunnel Lining

Tunnel lining refers to systems installed to provide initial support for stabilization during excavation and to provide permanent support as durable and maintainable long term finishes. Types of linings to be used depend on the ground conditions and on the end use of the tunnel. Stabilization and lining are provided in two separate operations in a system called "two-pass" system, while a "one-pass" system will combine the functions of stabilization and lining (Bickel et al., 1997). A one-pass system would be suitable for tunneling using cut-and-cover method, where the tunnel structure would work as the lining.

Waterproofing liners are sometime used to protect the tunnel over its working life. Historically, water has been one of the most serious problems encountered in underground works and it requires extremely expensive repair and renovation works. Underground structures should be kept dry, both for safety reasons and on functionality and maintenance grounds. Synthetic liners are sometime used in the construction of the underground railway lines (i.e. railway line in Milan and Rome in the mid-60) (http://www.flaguk.co.uk/underground.htm).

5.7 Design of subway station

The construction method of a subway station is similar to the box construction of tunnel structure Construction of station building may perform without the application of earth retention system where open spaces are available and SPT-N value of clay lies between 8 to above 30 (Bickel et al, 1997) then If the SPT-N value of clay lies between 0 to 4 and where limitation of space exists use of shore pile wall will be required for construction of metro railway stations in Dhaka City. Figure 5.44 shows a plan of a metro rail station

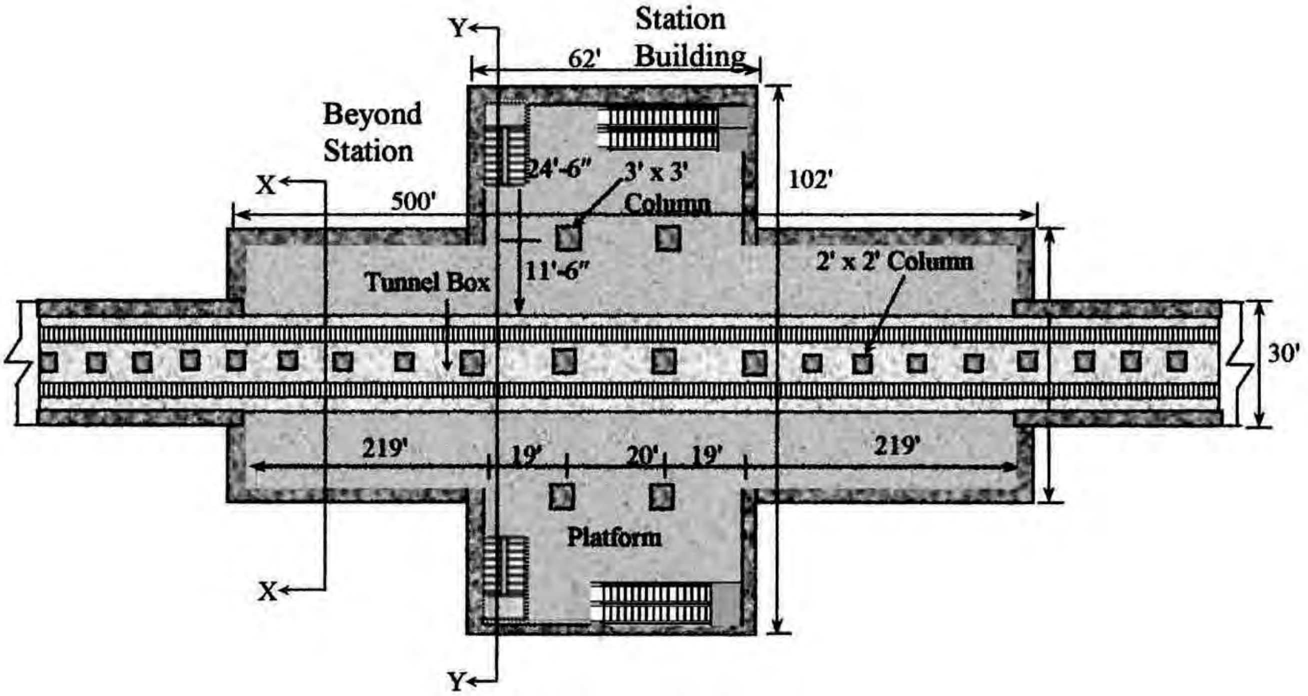


Figure 5.44 : Sectional Ground Floor Plan of a Typical Railway Station.(Not to scale)

assumed for Dhaka city. After the tunnel box section of 30 ft width, the box is widened to 50 ft at the station to facilitate the passengers to off load. The structure widened further to 102 ft to accommodate the platform and a station building for services. The width of the station building was assumed as 62 ft. A three storied building is considered with the

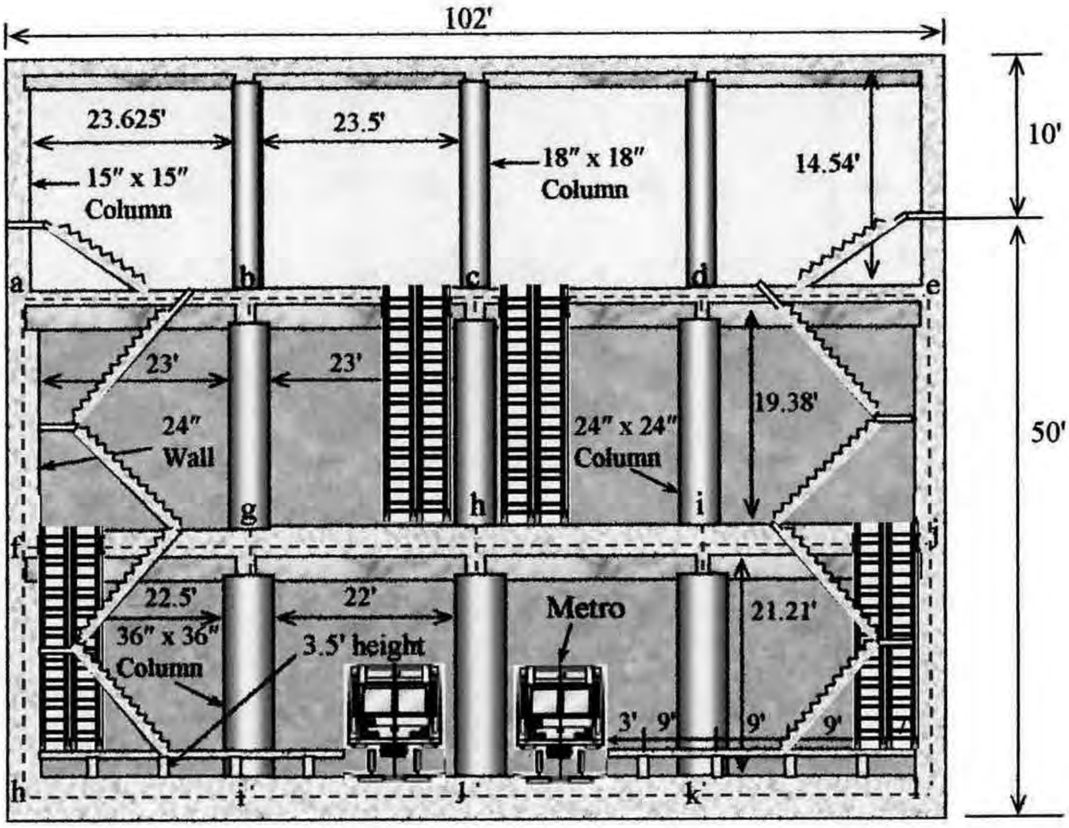


Figure 5.45 : Cross section of Station Building in Y-Y direction. (Not to scale)

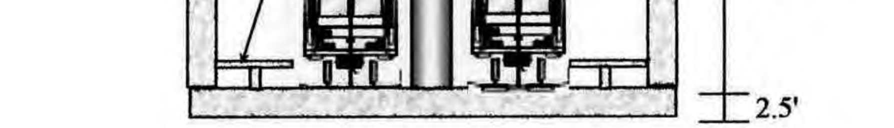


Figure 5.46 : Cross section of Station Building in X-X direction. (Not to scale)

total height of the building as 60 ft from the depth of cut. Thus top 10 ft is situated on the ground surface. Figure 5.45 and 5.46 shows cross sections of the building indicating different facilities. The building has been designed for the dead load, live load and the

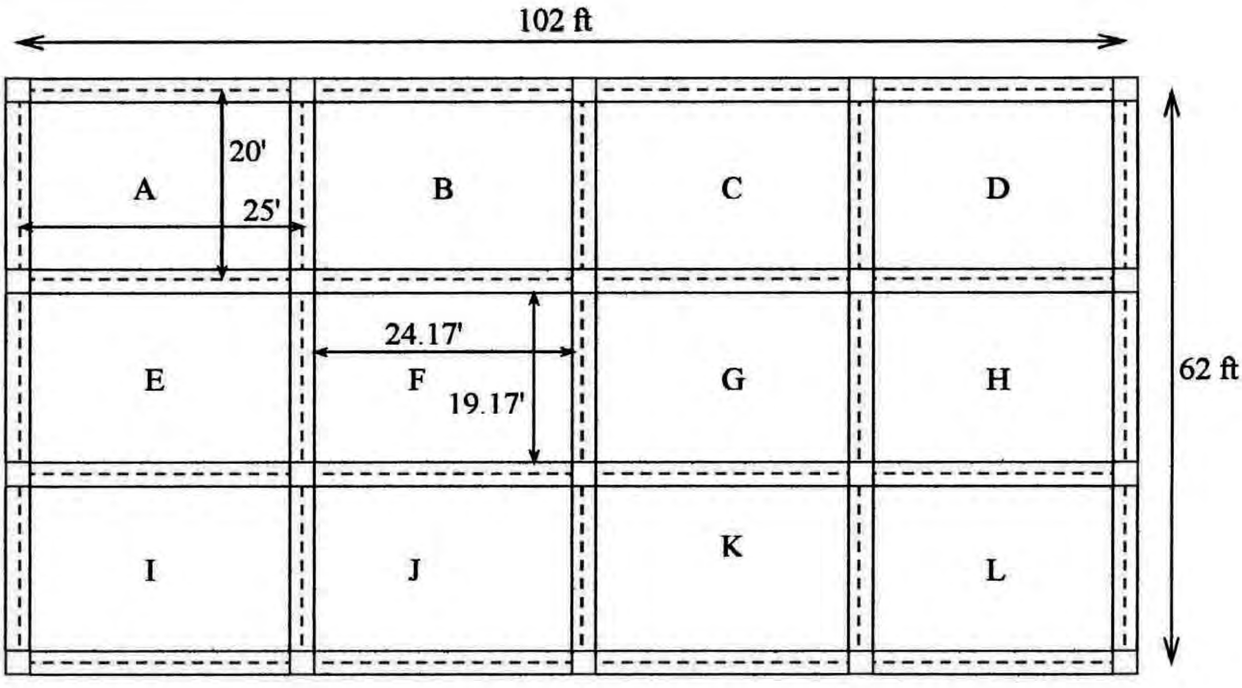


Figure 5.47 : Component of Slab in station building. (Not to scale)

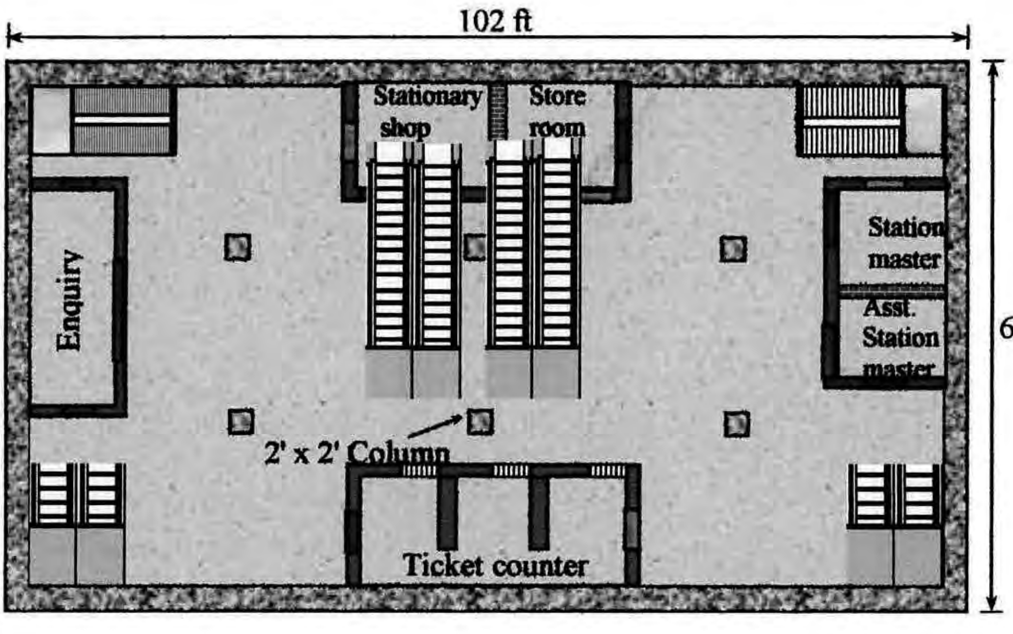


Figure 5.48 : Sectional plan of First floor Slab. (Not to scale)

earth pressure. The Slab of the Station building is divided into twelve parts. Three sides of the Station building are covered by the walls and one side of the building is attached with the Tunnel Box at Ground floor. Components of the in slab of the station building is shown in Figure 5.47. For saving the cost by decrease the thickness of slab and increase the rigidity of Station building against earth pressure at different sides Beam-Column slab was chosen. The sectional plan of first floor and second floor are shown in Figure 5.48 and Figure 5.49. Detail calculations of station building are given in Appendix II.

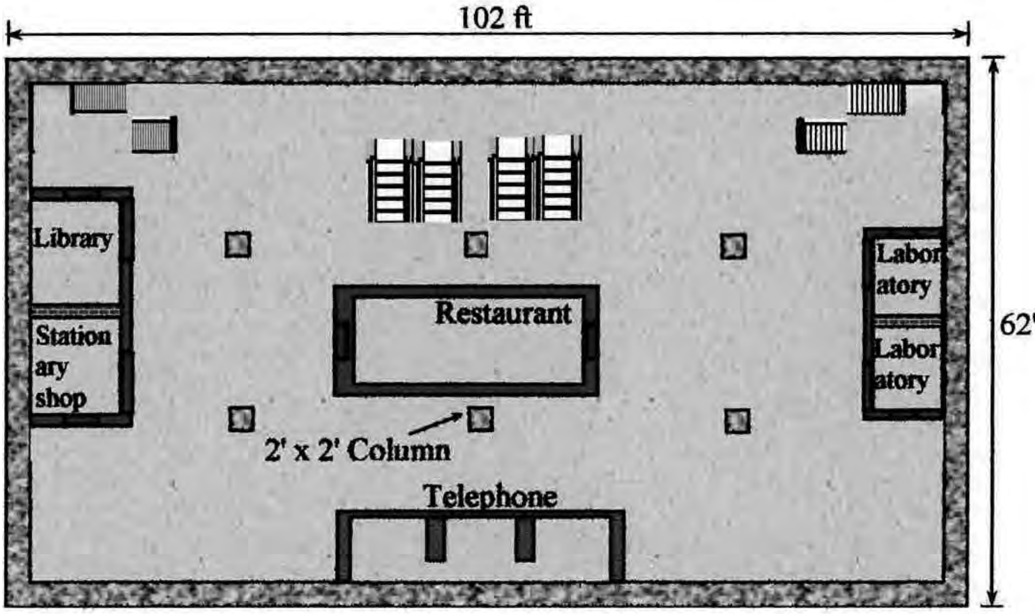


Figure 5.49 : Sectional plan of Second floor slab. (Not to scale)

Designing the Ground floor and Surrounding Wall of the Station building, Earth pressure and Vertical Load (Dead Load and Live Load) are considered. For the factor of safety, horizontal earth pressure is considering for Long-term condition. The Internal sanitary and water supply system is arrange at ground surface and Internal Electrification and Ventilation System are accommodate inside the building. Details structural arrangement of the Station building is given by Figure 5.50.

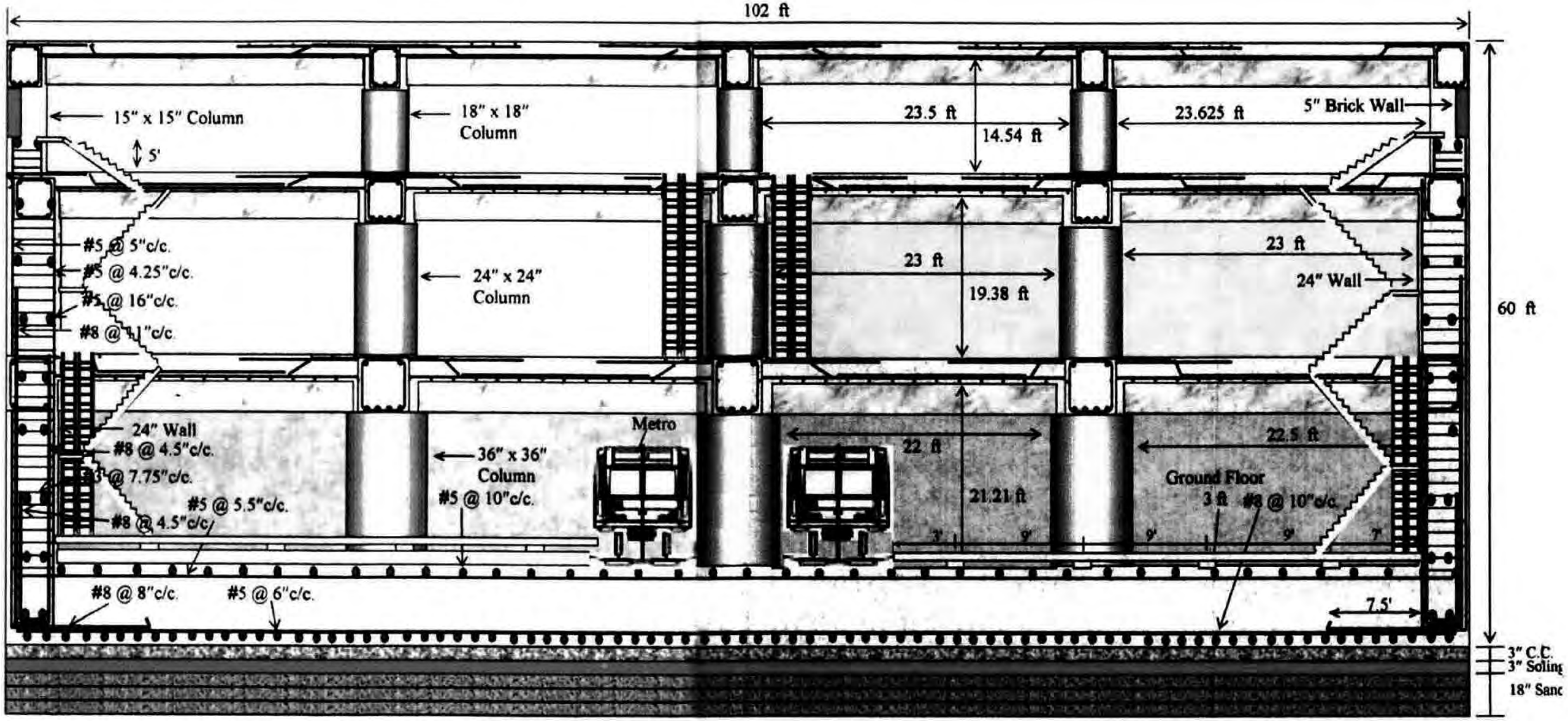


Figure 5.50 : Longitudinal Cross Section of Metro-Railway Station at Interior Face (Middle Section) (Not to scale)

CHAPTER 6

COST ANALYSIS

6.1 Introduction

Construction cost for a tunnel project along the proposed route has been estimated considering different earth retention systems. An estimation of the cost for the materials and equipments would be required to determine the suitability of tunneling for Dhaka city. The bill of quantities and the cost on the basis of "Schedule of Rates for Civil Works (PWD 2006), was determined. This chapter is divided into three parts : (1) Cost for earth retention system. (2) Cost for the tunnel structure including subway stations and (3) Comparison of cost for underground tunnel with existing flyover structure.

6.2 Cost for Earth Retention System

6.2.1 Costs for different site conditions

To estimate and compare costs for different earth retention systems it is necessary to know the amount of materials and equipments required for construction of tunnel by each earth retention method. Then the price of materials and equipments is figured out based on Public Works Department schedule of rates. Table 6.1 shows sample calculation of the cost for a drilled pile system. Details of calculation for all other systems are presented in Appendix-III. The total cost and cost per km for different earth retention methods at different soil conditions are arrange in a tabular form in Table 6.2. For upper clay depths of 20 ft, 65 ft soft upper clay layer and for the site with Building structure corresponding to the Case 1, Case 3 and Case 4 respectively discussed in Chapter 5, cantilever sheet piles were found infeasible as active earth pressure is high and passive resistance is less. Cost of cantilever sheet pile for these condition is therefore not included in the Table. For clay depth of 77 ft corresponding to the Case 2, the cost of per kilometer earth retention using cantilever sheet pile is Taka 595 million. It is revealed from Table 6.2 the cost of diaphragm wall is generally less than the other methods of earth retention among the braced system. The bored or drilled pile appeared as one of the costliest method. It also depend on the soil condition of the sites. Figure 6.1 compares from Tk.355 million to

Table 6.1 : Sample cost estimation for earth retention system (for Drilled pile)

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (million Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Boring for cast-in-situ pile	Rm	513	1333364	684.01
3	Cast-in-situ pile with R.C.C. works	Cum	7668	173052	1326.96
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	320666	1731.59
5	Fabrication of 60 grade 10 mm deformed bar	Quintal	5400	228686	1234.90
6	Providing and making Point Welding	Point	2.38	210452820	500.87
7	Providing and making Welded Splice	Inch	3.57	9446760	33.72
8	Supplying, fabrication & fixing of Wales	Quintal	7359	213610	1571.95
9	Supplying, fabrication & fixing of Struts	Quintal	7359	138362	1018.20
Total Cost for 20 ft depth of upper clay layer					8103.52

Table 6.2 : Variation of Cost (million taka) in different soil conditions by different Methods

Soil Conditions	Anchored S. P.		Braced-Cut		Bored Pile		Diaphragm Wall	
	Total cost (million Tk.)	Cost per km (millon Tk.)	Total cost (million Tk.)	Cost per km (million Tk.)	Total cost (million Tk.)	Cost per km(mill. Tk.)	Total cost (million Tk.)	Cost per km (millon Tk.)
Assuming 20' depth of upper Clay layer	Not Applicable	Not Applicable	7460.54	373.03	8103.52	405.18	6566.91	328.35
Assuming 77' depth of upper Clay layer	11891.9	594.60	7335.74	366.79	7978.72	398.94	6442.10	322.10
Assuming 65' depth of upper Soft Clay layer	Not Applicable	Not Applicable	7101.46	355.07	7744.44	387.22	5924.84	296.24
Building Structure beside the route	Not Applicable	Not Applicable	11686.07	584.30	10816.89	540.84	9206.07	460.30

* Tk. 1 Crore = Tk. 10 million

Tk.373 million for Case-1 to Case 3. Cost of the braced-cut construction increased for the site with a building nearby to the structure due to stiffer soil condition at the site. It is to be noted that the nearby structure located beyond 8 ft from has no influence on the design of retaining system. As discussed earlier, buildings structure are present closer to the railway track near Kawran Bazar and Malibag area. Soil conditions of this site was used to estimate the cost of earth retention system. Cost of the braced-cut wall for the site is Tk.584 million.

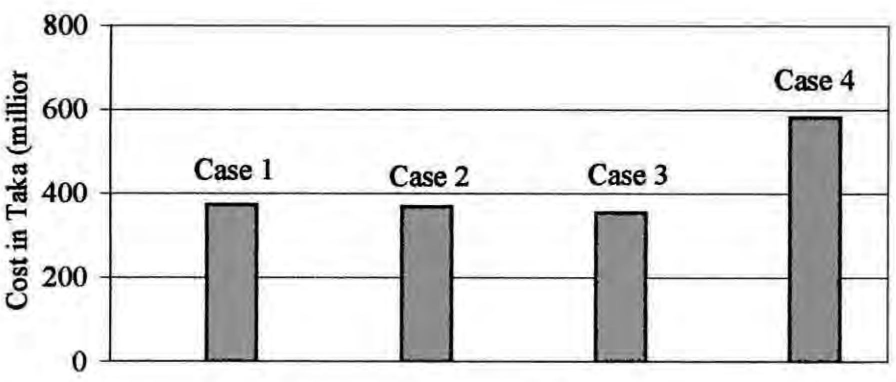


Figure 6.1 : Varition of Cost per km of Braced-cut in different Soil Condition

The cost for drilled pile is greater than the braced-cut by 8.62% to 9% for Case-1 to Case 3. The cost for Diaphragm wall are less than those for Braced-cut and Drilled pile. However, the differences varied depending on the ground condition. Figure 6.2 and 6.3 shows the cost for Drilled pile and Diaphragm wall, respectively. For each of the three earth retention system, the construction in the soft ground appeared to be less. The reason lies on the passive earth pressure behind braced wall which is greater for stiffer soil. Unit weight of the soil is also less for soft soil. The cohesion of the stiffer soil was conservatively assumed in the analysis.

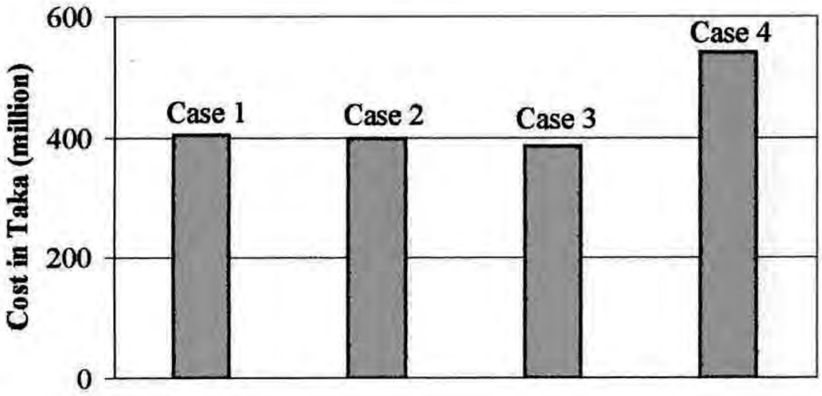


Figure 6.2 : Varition of Cost per km of Drilled pile in different Soil Conditions

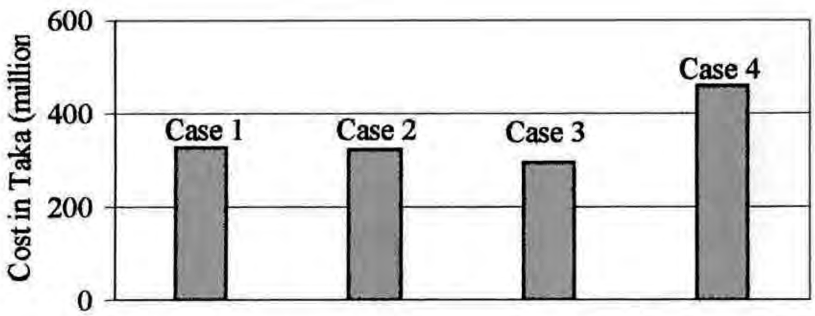


Figure 6.3 : Varition of Cost per km of Diaphragm Wall in different Soil Conditions

6.2.2 Comparison among the retention systems

Figure 6.4 to 6.7 show a comparison of the cost of different earth retention system for Case 1 to Case 4 of the site conditions (The cases were discussed in Chapter 5). Figure 6.4 reveals that for the Case 1 with 20 ft of upper clay layer soil, Diaphragm wall provide the most economic earth retention system, followed by the Braced-cut and Drilled pile. Cost of Diaphragm wall is 19% lower than the Drilled pile and 12% lower than the Braced-cut. Even for the site with Case 2, Diaphragm wall appeared as the most economic system of earth retention (Figure 6.5). Cost for Diaphragm wall is 19% lower than the Drilled pile and 12% lower than the Braced-cut. For the worst soil condition specified in Case 3, diaphragm wall appeared again as the most economic solution of earth retention (Figure 6.6). As discussed earlier, Sheet pile was infeasible for the worst soil condition. The cost for diaphragm wall is 23.5% lower than the drilled pile and 16.6% lower than the braced-cut.

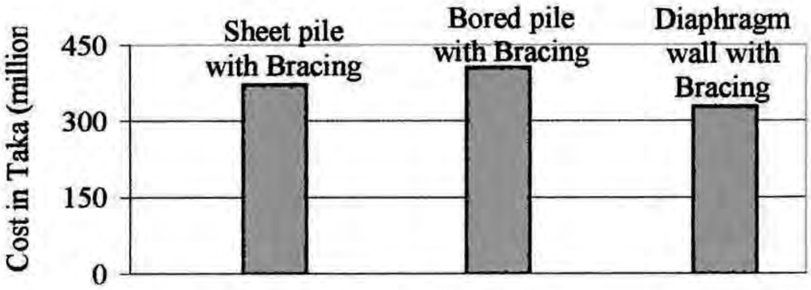


Figure 6.4 : Cost per km in different earth retention method for Case 1

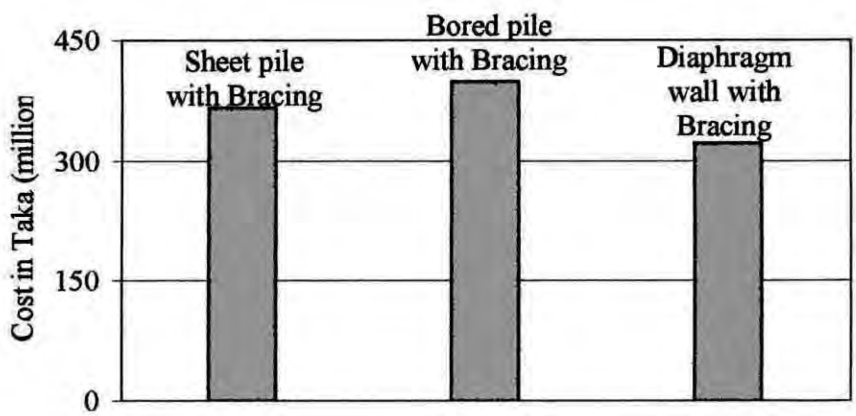


Figure 6.5 : Cost per km in different earth retention method for Case 2

For the case of Complex soil condition (also including building structure beside route) (Case 4), cost per km of different earth retention methods shown in Figure 6.7. Diaphragm wall was again found as the economic earth retention system. Cost of Diaphragm wall is lower than Braced-cut by 21%, and than Drilled pile by 15%. Based on the findings presented above, diaphragm wall system is recommended for earth retention for construction of metro tunnel in Dhaka city.

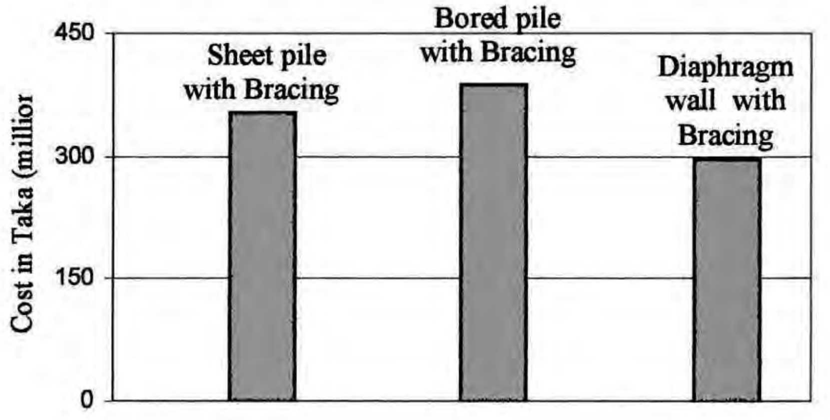


Figure 6.6 : Cost per km in different earth retention method for Case 3

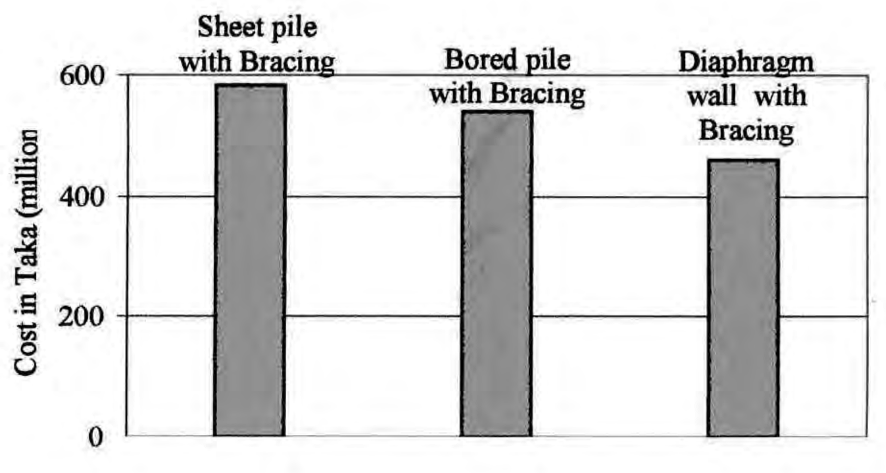


Figure 6.7 : Cost per km in different earth retention method for Case 4

6.3 Cost for Tunnel Structure

Construction of tunnel structure involve earth work and tunnel construction. Both the excavation and the structural cost is included in the cost of tunnel structure. Costs for the earth work and the tunnel structure are discussed below :

6.3.1 Earthwork

After the installation of the earth retention system soil is excavated and the tunnel structure is constructed. Earth cutting and sand filling are included within the cost estimation of earth-work. Cost of dewatering required during excavation is also included according to PWD (2006). PWD (2006) assume an additional cost of excavation for the dewatering. Along the typical route of about 20km long, total volume of earthwork in excavation in foundation trenches upto the maximum depth of 10 meter of tunnel excavation is 2788519 cum. Cost for cutting the volume of earthwork is calculated to be Taka 89.23 million based PWD (2006), with rate applicable for depth upto 1.5 m. However, there would be additional costs for excavation depth beyond 0.5 m, 1.5 m and others. Considering everything total costs for excavation becomes Tk.108 million. Extra rate for earthwork in excavation in foundation trenches, all around protected by palisading and de-watering/bailing-out water upto 1.5 m depth and 60 m lead is 2788519 cum. Cost for this volume of dewatering is 680.4 million. Extra rate for earthwork in excavation in foundation trenches, all around protected by palisading and de-watering/bailing-out water for depth exceeding 1.5 m is 2514129 cum. Cost for this volume of dewatering is 1068.5 million. and Sand filling in foundation trenches and plinth with coarse sand having minimum F.M. 1.2 in 150 mm layers including leveling, watering and compaction to specified percent by ramming each layer up to finish level is 1003867 cum and cost for this volume is Taka 654.52 million. Along the typical route, there is no definite place for storing such a large volume of earth which would be excavated. So, the excavated earth should be replaced after construction to a place, where there are available rooms beside the route for storing this volume of earth and where earth retention system is free from surcharge effect generating by these volume of excavated earth. No specific cost was considered for this item of replacing and storing of earth . Details calculation for earth work and related costs are given in Appendix-III.

6.3.2 Tunnel Structure

Cost of tunnel structure has been estimated starting from brick soling to painting. The area of one layer brick flat soling in floor with first class bricks including of bed and filling the interstices with local sand is 182927 sqm and total cost for this amount is Taka 29.63 million. The area of 75 mm thick damp proof course (1:1.5:3) with Sylhet sand (F.M. 2.2) and stone chips is 182927 sqm and total cost for this amount is Taka 123.66 million. Reinforced Cement Concrete (R.C.C.) works with stone chips in floor slab, roof slab, side wall, middle column and wall and column capitals in nominal mix 1:1.5:3 and minimum f_{cr} = 30 MPa are 414933 cum and total cost for this amount is Tk. 2551.27 million. Shuttering, propping and necessary supports for floor slab, roof slab, side wall, middle column and wall and column capitals are 420440 sqm and total cost for this amount is Tk.165.04 million. Fabrication of 60 grade 25mm, 16mm, 12mm, 10mm deformed bars are of 385149 quintal and total cost for this amount is Tk.2079.80 million. Minimum 6 mm thick cement plaster (1:4) to floor slab, roof slab, side wall, middle column and wall and column capitals is 640927 sqm and total cost for this amount is Tk.63.45 million. Plastic emulsion paint of best quality and approved color to floor slab, roof slab, side wall, middle column and wall and column capitals of two coats over a coat of acrylic sealer including, cleaning, sand papering the surface and necessary scaffolding etc. all complete is 496342 sqm and total cost for this amount is Tk.44.17 million. Including everything, total cost of tunnel structure for the total length of 20 m is Tk. 8262.82 million.

6.3.3 Tunnel lining :

As discussed earlier one pass tunnel lining would be suitable for tunneling using cut and cover method. However, as water proofing liner, polythene sheet of one kilogram per 6.5 m^2 specification can be used. The area to be covered by the polythene sheet is 585366 m^2. According to PWD (2006), the cost of sheet would be Tk. is 12.88 million.

6.4 Cost for Station Building

6.4.1 Earthwork

Layout, shore protection work, earth cutting and sand filling are included within cost estimation for earth work required for the station building. Total area required for layout and marking is 1700 sqm and cost for this area is Tk.0.012 million. Shore protection work per station building is 5300 sqm and total cost for this area is Tk.3.07 million. Total volume of earthwork for each station building, excavation in foundation trenches upto the maximum 10 meter depth is 25650 cum and cost for cutting the volume of earthwork is Tk.0.821 million. Considering the additional cost for excavation beyond 1.5 m depth, total cost of earth cutting becomes Tk.1.0 million. Extra rate for earthwork in excavation in foundation trenches, all around protected by palisading and de-watering/bailing-out water upto 1.5 m depth and 60 m lead is 25650 cum. Cost for this volume of dewatering is 6.26 million. Extra rate for earthwork in excavation in foundation trenches, all around protected by palisading and de-watering/bailing-out water for depth exceeding 1.5 m is 23150 cum. Cost for this volume of dewatering is 9.84 million. Sand filling in foundation trenches and plinth with coarse sand having minimum F.M. 1.2 in 150 mm layers including leveling, watering and compaction to specified percent by ramming each layer up to finish level is 700 cum and cost for this volume is Tk.0.46 million. The cost for storing and removal of excavated material was not included within the estimated cost.

6.4.2 Station Structure

Estimation of the cost of station structure includes from brick soling to roof top parapet cost. The area of one layer brick flat soling in floor with first class bricks including of bed and filling the interstices with local sand is 1530 sqm and total cost for this amount is Tk.0.25 million. The area of 75 mm thick damp proof course (1:1.5:3) with Sylhet sand (F.M. 2.2) and stone chips is 1530 sqm and total cost for this amount is Tk.1.03 million. R.C.C. works with stone chips in ground floor, roof, second and first floor, side wall, column, beam, staircase and lintle in nominal mix 1:1.5:3 and mimimum f_{cr} = 30 MPa are 5161 cum and total cost including extra cost over the rate of concrete (materials, consumables and placing concrete) of R.C.C works where the free height of the structure exceeds 4 meter for this amount is Tk.32 million. Shuttering, proping and necessary supports for ground floor, roof, second and first floor, side wall, column, beam, staircase and lintle are 7122 sqm and total cost including extra cost over the rate of shuttering (steel) of R.C.C works where the free height of the structure exceeds 4 meter for this amount is Tk.2.81 million. Fabrication of 60 grade 25mm, 16mm, 12mm, 10mm deformed bars are of 4119 quintal and total cost for this amount is Tk. 22.24 million. First class bricks works 125 mm thickness with cement sand (F.M. 1.2) mortar (1:4) and making bond with connected walls including necessary scaffolding, raking out joints, cleaning and soaking the bricks etc. is of 13005 sqm and cost for this amount is Tk. 5.5 million. Minimum 6 mm thick cement plaster (1:4) to ground floor, roof, second and first floor, side wall, column, beam, staircase and lintel is 6718 sqm and total cost for this amount is Tk. 0.67 million. Minimum 12 mm thick cement plaster (1:6) to wall both inner and outer face of second floor is 305 sqm and cost for this amount is Tk. 33.24 thousand. Total area of 10 mm thick color situ mosaic (Pakistani origin with glass strip) is 2884 sqm. and total cost for this amount is Tk. 2.66 million. Plastic emulsion paint of best quality and approved color to floor slab, roof slab, side wall, middle column and wall and column capitals of two coats over a coat of acrylic sealer including, cleaning, sand papering the surface and necessary scaffolding etc. all complete is of 5170 sqm and total cost for this amount is Tk. 0.46 million. Total area of cement paint to outer wall of approved quality and color with (Bangladesh made) two coats over a coat of priming including cleaning and sand papering the surface and necessary scaffolding etc. is of 305 sqm and cost for this amount is Tk. 18.91 thousand. Supplying, fitting and fixing total stair railing of standard height of any design and shape with square box (1 No. box in each trade) made by through welding of 2 nos. 25x25x5 mm mild steel angle is 35 sqm and cost for this amount is Tk. 0.13 million. Total cost for supplying, fitting and fixing of 2 Nos. collapsible gate of 13 sqm area is Tk. 37.59 thousand. Total cost for supplying, fitting and fixing window grills of 168 sqm area is Tk. 0.19 million. Total cost for supplying, fitting and fixing Steel glazed window Shutter of 168 sqm area is Tk. 0.55 million. The cost for supplying, fitting and fixing of 65 sqm M.S. Door Shutter is 0.22 million taka. Cost for painting to Door and window Shutter of 110 sqm area is 11.11 thousand taka. Providing drip course of 116 rm length costs 5.45 thousand taka. The cost for roof top parapet of 92 sqm area is Tk. 87.67 thousand. Totaling all components of the cost, structural cost of the tunnel structure is Tk. 101.39 million. Detail calculation of the cost estimation is provided in Appendix III.

6.5 Cost for Utilities

6.5.1 Utilities for Tunnel Box

Electrification cost is considered for non-residential standard condition with an area of 182927 sqm. The cost for this amount is Tk. 176.89 million based on PWD rate schedule. If the ventilation cost is considered as 4 % of total civil cost, the ventilation cost of the tunnel box become Tk. 303.25 million. Thus, the utility cost for a station building is Tk. 480.14 million.

6.5.2 Utilities for Station Building

Cost of Lighting and Escalators per station is assumed to be Tk. 30 million. Air Conditioning and Ventilation Cost is near about Tk. 64.4 million, based on the cost of Kolkata metro tunnel. Thus, the utility cost for a station building is Tk. 94.4 million.

6.6 Comparison of Cost Between Tunnel and Flyover

Total cost of tunnel per unit length has been estimated to compare the cost with existing flyover system in Dhaka. Three types of costs are included within the total cost of Metro Tunnel for Dhaka City. These are : (i) cost of application earth retention system (ii) construction cost of Tunnel Box and (iii) cost of Metro Railway station. Since cost of diaphragm wall construction appeared minimum, this has been recommended as the earth retention system. Cost of diaphragm wall was therefore considered in the estimation of total tunnel box construction. For the cost of typical flyover, the cost of Mohakhali flyover was considered. This flyover is located at Mohakhali, Dhaka which was inaugurated on 4th November, 2004 and laid the foundation by 19th December 2001. Total length of the flyover 1.12 km. It has width of 17.9 m, 4 lane, 18 pillar and 19 span. Total construction cost for the flyover was Tk. 1150 million. The cost was increased by 30% to account for the price escalation over the period. Thus, the estimated cost of the flyover is Tk. 1495 million, resulting in a per km cost of Tk. 1335 million. On the other hand, total cost of 20 km tunnel considering 21 stations is Tk. 19644 million including the cost of earth retention, tunnel structures and station buildings. This results in a per km cost of Tk.

983 million. Table 6.3 and Figure 6.8 compares the cost of tunnel and the flyover for Dhaka city. It appears that cost of tunnel would be less than the cost of flyover by about 26%. However, a metro tunnel would provide a most effective solution of traffic conditions in a crowded city like Dhaka. A construction of metro rail is therefore expected for improvement of the overall traffic condition of Dhaka. Figure 6.8 shows the variation of cost per km between Metro Rail Tunnel and Flyover Structure.

Table 6.3 : Variation of Cost between Metro Tunnel and Flyover Structure.

Types of Struture	Total construction cost	Cost per km
	(million Tk.)	(million Tk.)
(01) Metro Rail Tunnel	19644 (20 km)	983
(02) Mohakhali Flyover	1495 (1.12 km)	1335

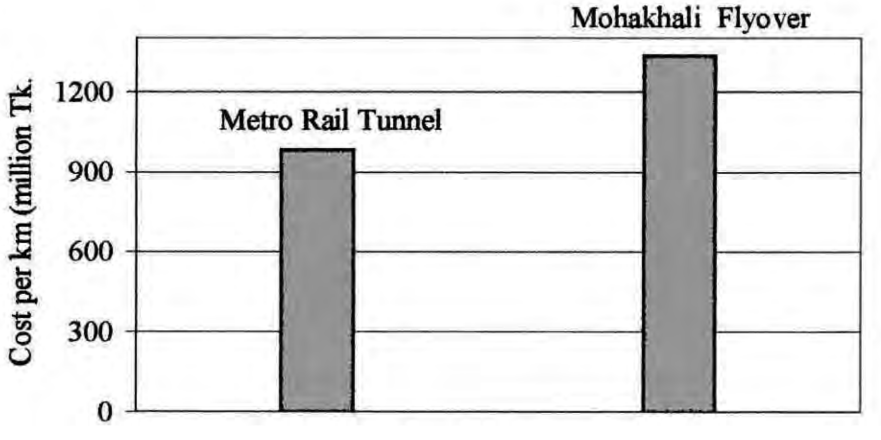


Figure 6.8 : Variation of cost per km bet^n Metro Rail Tunnel and Flyover Structure.

CHAPTER 7

CONCLUSION AND RECOMMENDATION

7.1 Introduction

Underground Metro system is believed to be the most effective transportation means for a crowded city like Dhaka. A metro rail system is desired for Dhaka city for eliminating traffic congestion and related problem in the city. A feasibility study for a metro rail tunnel in Dhaka has been performed in this thesis. A tunnel alignment along the existing surface railroad system from Tongi to Kamalapur has been chosen for the feasibility study. In this typical route there are 18 crossing points between Railway and highway. Average of 30 ft to 35 ft wide right of way exists on each side of two existing parallel railway lines, facilitating construction works for the tunnel.

A study was performed to identify the infrastructures nearby to the existing railway line that may influence the tunnel construction. Excavation nearby to the structures for the tunnel may cause horizontal movement and settlement of the structure that may result in complete collapse or cracking of different components of the structures. Additional load from the structures, on the other hand, may cause additional lateral pressure to the retaining system during excavation. It is revealed that the buildings are located from a distance of 4m up to 40 m from the rail line. Numbers of buildings are the highest in the Basabo, Moghbazar and Mohakhali areas. Minimum distance of the building are the shortest in Moghbazar and Mohakhali area. However, some of the structures that are too close to the track may be unauthorized and therefore can be removed for tunnel construction. The buildings with small areas are likely to be unauthorized.

Existing rail tracks are two meter gauge covering an average width of 10 m. One meter gauge lane can be continued to function for both way traffic, during construction of subway, while other track would be closed . Thus, width of the working area that would be available is about 8 to 10 m. At some places even a larger space is available that would facilitate the tunnel construction.

The displacement occurring in the structures and soil mass adjacent to an excavation will have an effect on the choice of earth retention system required to protect the structures. In softer cohesive soils a stiffer support systems such as concrete diaphragm wall would be required to limit movements. However, the possibility of using different earth retentions systems such as sheet piles, braced-cut, drilled pile and diaphragm wall has been investigation to identify the method that would be suitable for Dhaka. The study revealed that underground metro tunnel system is feasible for Dhaka city and would appear as an effective solution for the current traffic condition. Diaphragm wall system appeared as effective earth retention system during excavation of the tunnel.

7.2 Geotechnical Condition

Geology of Dhaka consist of the Uplifted Pleistocene Madhupur clay with thickness varying from 20 ft to over 100 ft overlying fine to coarse grained sands of the Dupi Tila Formation. The sands are approximately over 300 to 400 ft thick in Dhaka. Geotechnical characterization of subsoil condition along the proposed tunnel alignment revealed variability of the subsoil. The subsoil was characterized dividing the route into 11 zones to cover different soil conditions encountered. Soil parameters for the soil encountered along the profile was established based on test results and available correlations. Considering the soil profile and the soil parameters, four general cases of ground condition has been chosen for analysis of earth retention system for tunnel construction that would cover the best to the worst ground condition expected along the side.

7.3. Selection Tunnel Structure

Considering minimum clearances recommended for railroad tunnels by American Railway Engineering Association for double track, a tunnel box width of 30 ft is chosen in this study for the proposed double track metro tunnel in Dhaka. Vertical clearance within the box was used as 13 ft based on the standard box section.

A side platform type station is chosen for the Metro line. For easy entrance into the Metro station, a three storied building is considered. The width and height of Metro railway station have been chosen as those for typical stations. There are six columns inside the stations with dimensions 36"x 36". Clear spacing between the columns is 25 ft in longer direction and 20 ft in shorter direction. The platform height above the ground level is 3.5 ft. Total length of the platform is chosen as 500 ft. A provision two staircase and two escalator is included in the station.

7.4 Design of Earth Retention System and Tunnel Structures

Design of the earth retention system revealed that sheet pile wall will not be suitable for the ground condition and the depth of excavation required for the tunnel in Dhaka. Earth retention system of braced-cut, drilled pile and diaphragm wall has been designed based on the earth pressure proposed in Peck (1969). The effect of nearby structures on the design of the retention system revealed that if the structure is located at 8 ft or beyond, the effect of the building structure on the earth retention system would be negligible. Tunnel structure and the station building was designed for the dead loads and the live loads expected for typical tunnel structures.

It was revealed that the better the soil condition the higher was the wall moment for the braced retaining wall. This is due to the fact that the earth pressure against braced wall is due to passive pressure which greater fore stiff soil than the softer soil.

7.5 Cost analysis

For the earth retaining system, cost of diaphragm wall was found as the minimum of the three earth retention system such as braced-cut, drilled pile and diaphragm wall. The cost of diaphragm wall for the proposed tunnel ranges from Tk. 296 million (29.6 Crore) to Tk. 460 million (46 Crore). While the cost for braced-cut ranges from Tk. 355 million (35.5 Crore) to Tk. 584 million (58.4 Crore) and for drilled pile ranges from Tk. 387 million (38.7 Crore)to Tk. 541 million(54.1 Crore). Thus the cost of diaphragm wall is less than those of braced-cut by 17% to 21%, and less than those of drilled pile by 23% to 15%. Total cost of 20 km tunnel considering 21 stations is Tk. 19644 million, resulting in a per km cost of Tk. 983 million. When compared to the cost of flyover structure , found for Mohakhali Flyover, the cost of tunnel appear 26% less than the cost of flyover structure.

7.6 Recommendation for Future Study

For future study on the tunneling on Dhaka city map include following aspects :

(1) The analysis of earth retention method has been performed on the basis of field data available from the literature. The borehole locations from the literature was not always located on the tunnel route. In most cases, the boreholes were within 1 km from the existing rail road. Boreholes and soil sampling may be conducted along the tunnel route to obtain site specific sub-soil data to be used for tunnel design.

(2) A detail investigation can be conducted to obtain more accurate soil parameter, rather than relying on estimation based on empirical equations.

(3) Earth pressure against the retaining structure was estimated based on the empirical pressure distribution of Peck (1969). A more sophisticated finite element analysis can be performed to obtain a realistic earth pressure for design of the earth retaining system.

(4) Finite element analysis can be extended to investigate the ground deformation and the effects of tunnel excavation on nearby structures.

(5) A parametric study may be performed to identify the effective tunnel structure based on finite element analysis.

(6) A detail field survey can be conducted to identify authorized and unauthorized structures nearby to the tunnel route.

(7) Feasibility of metro tunnel of any other routes can be investigated.

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Appendix –I

Maps Along Tunnel Alignment

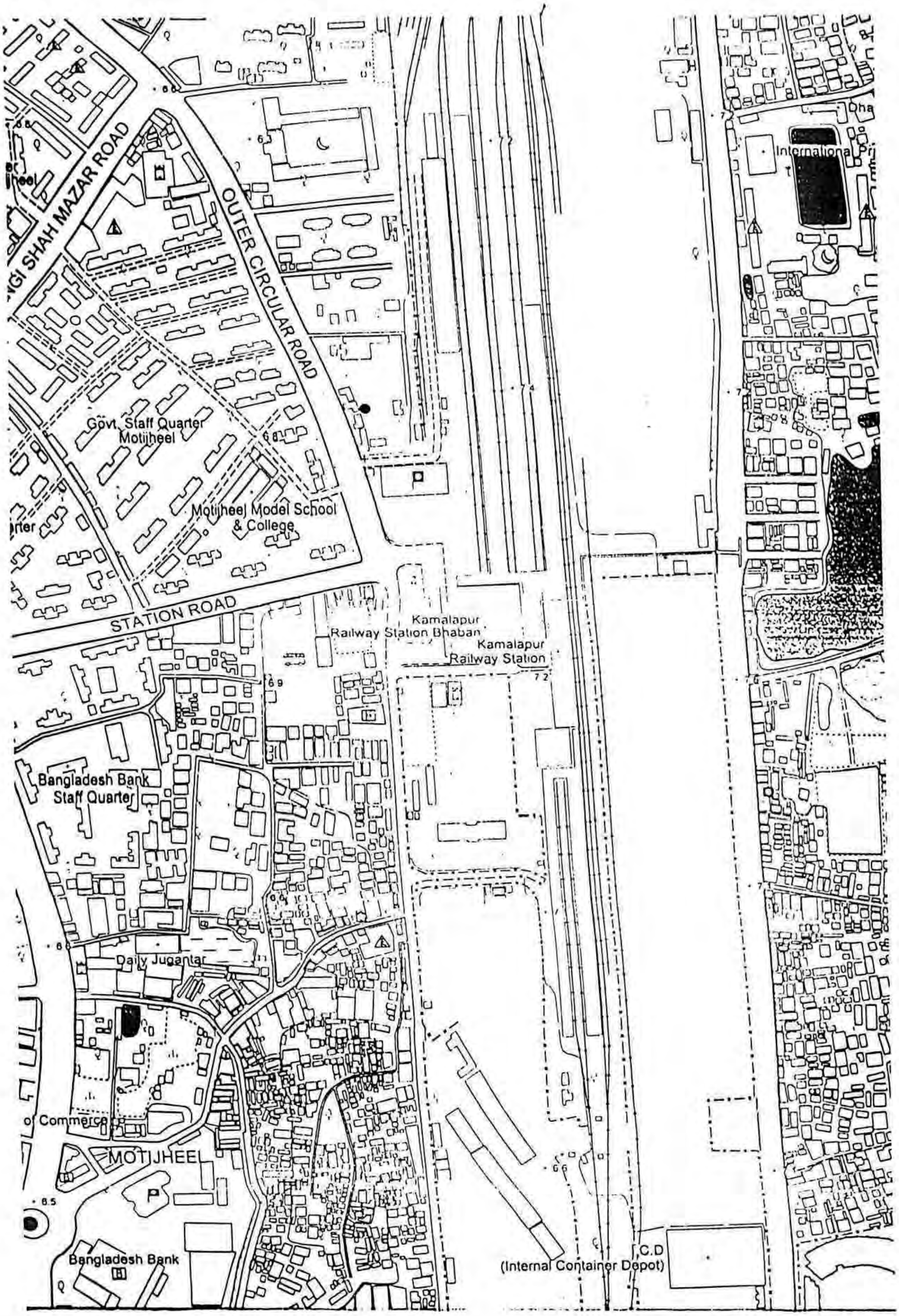


Figure I-1 : Map-1 (Siadabad to Kamalapur)

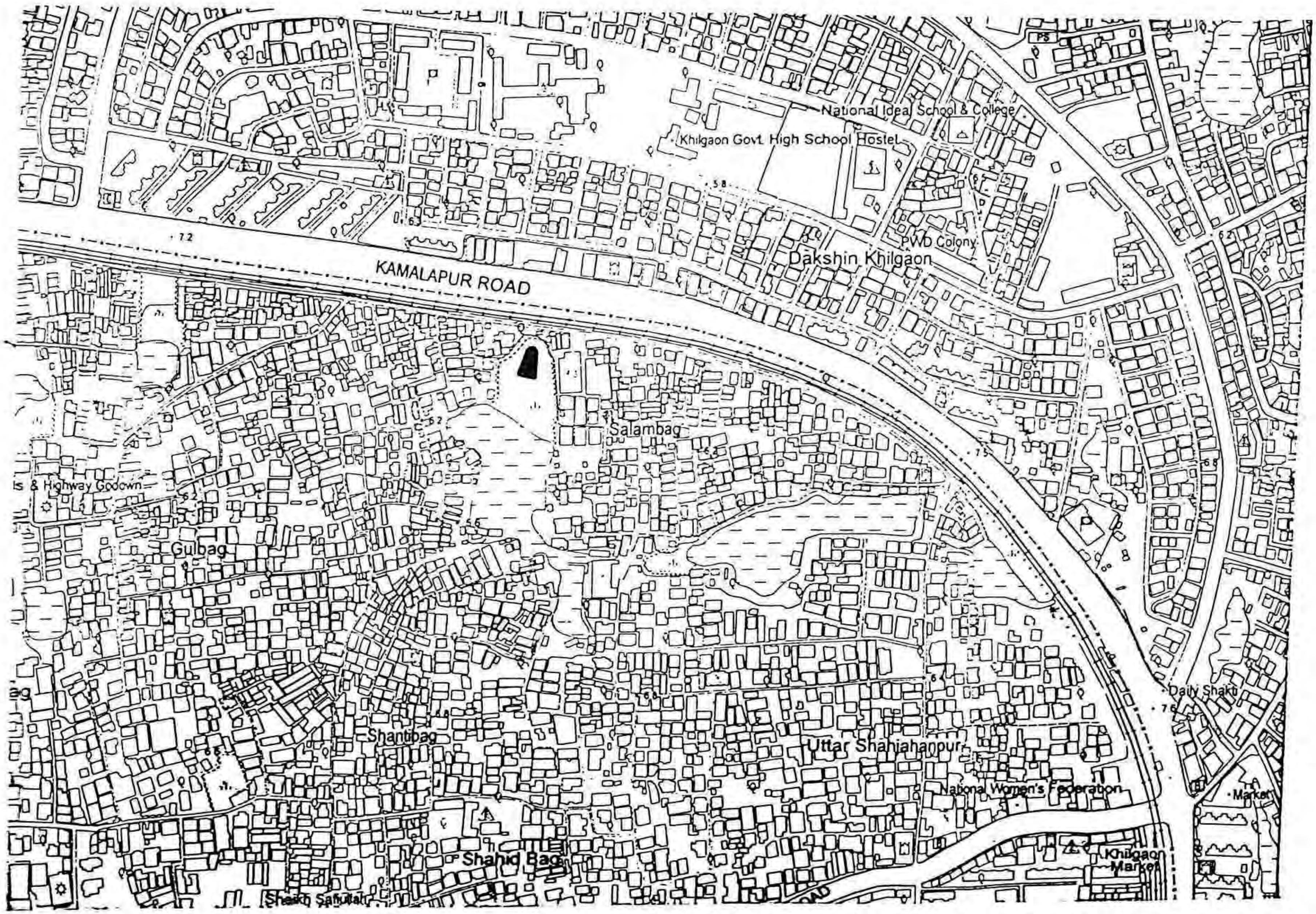


Figure I-2 : Map-2 (Basabo to Meradia)

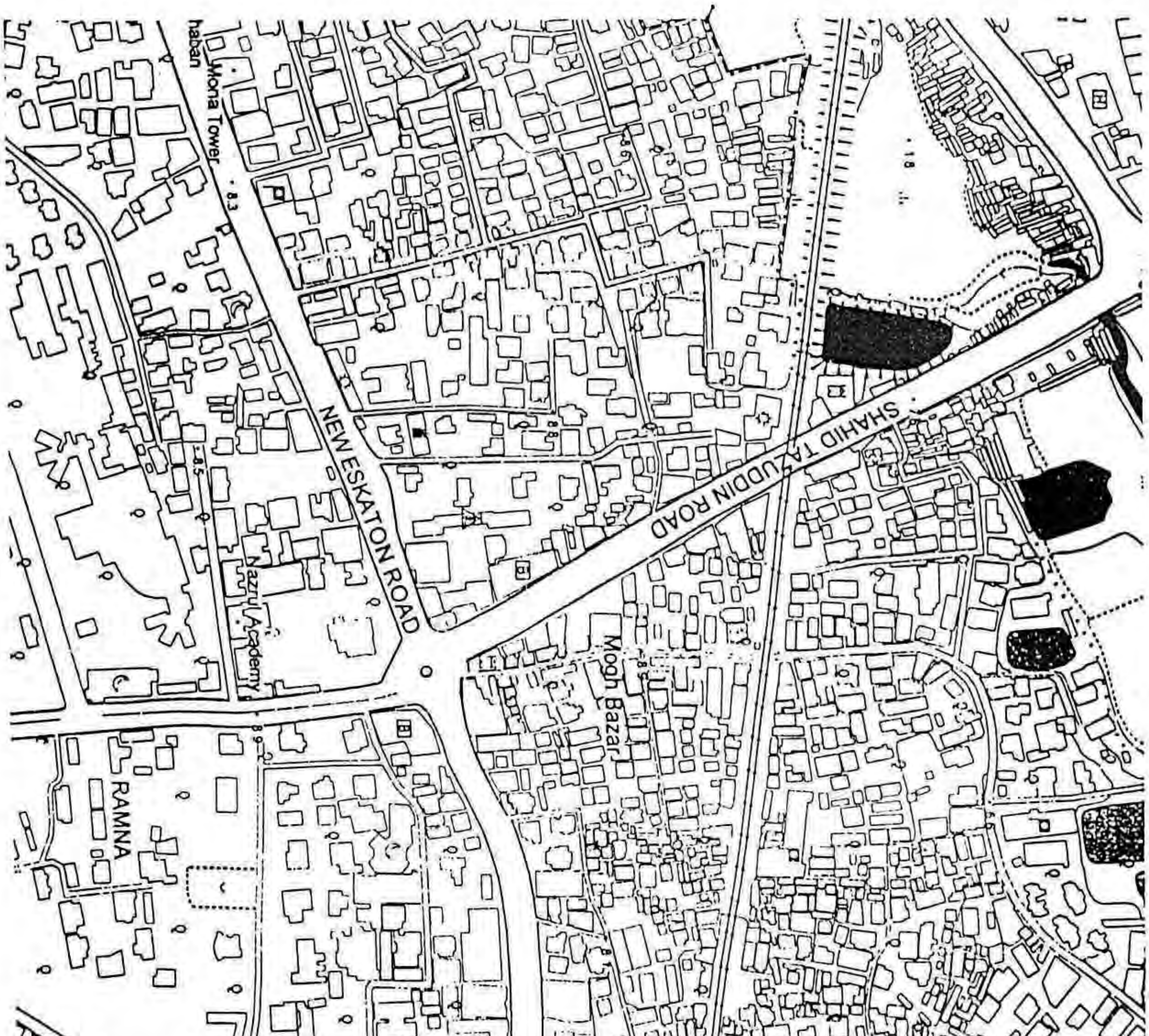


Figure I-3 : Map-3 (Ramna to Moghbazar)

Figure I-4 : Map-4 (Tajgaon R/A to Mohakhali)

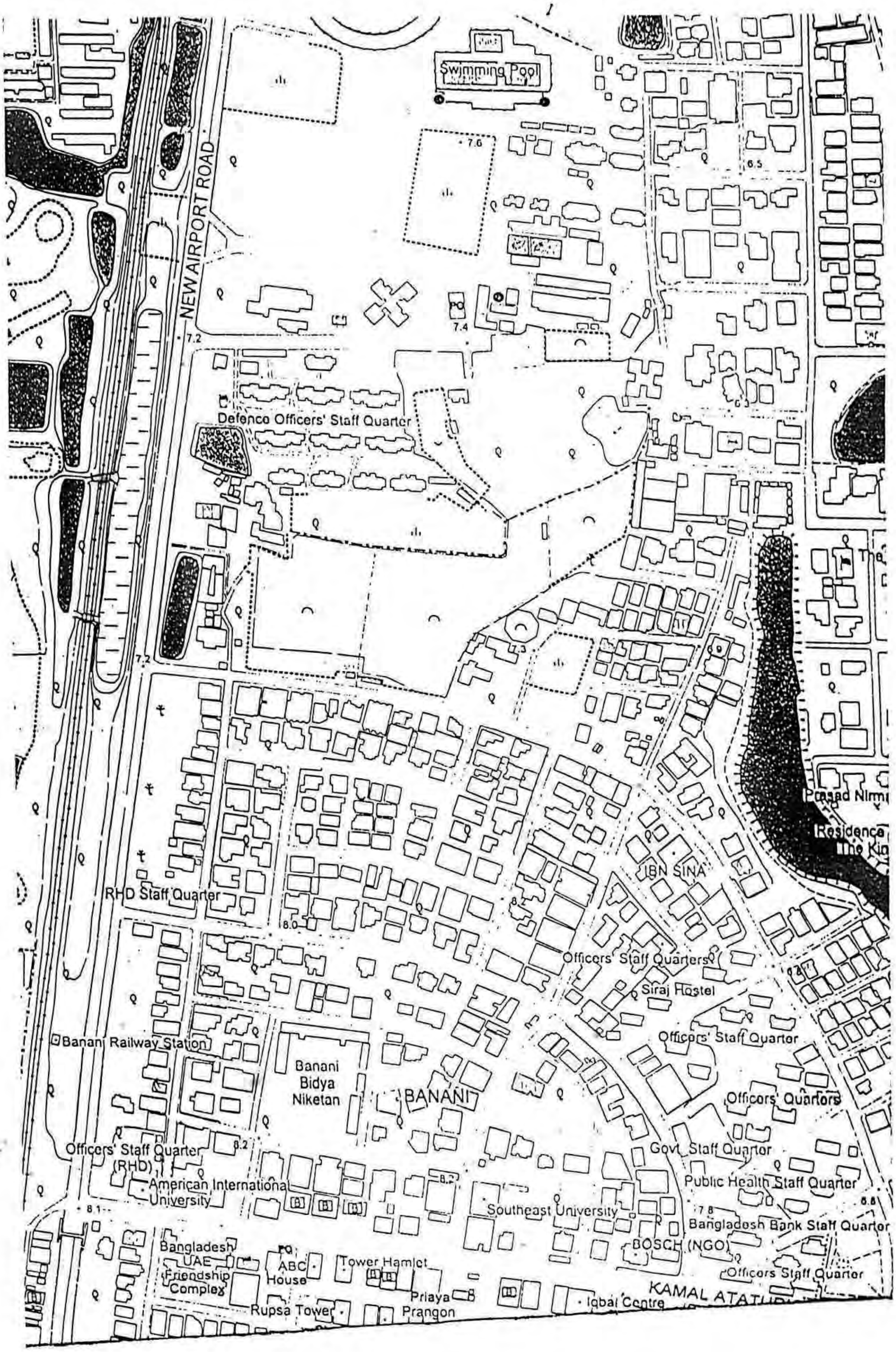


Figure I-5 : Map-5 (Banani to Army Stadium)

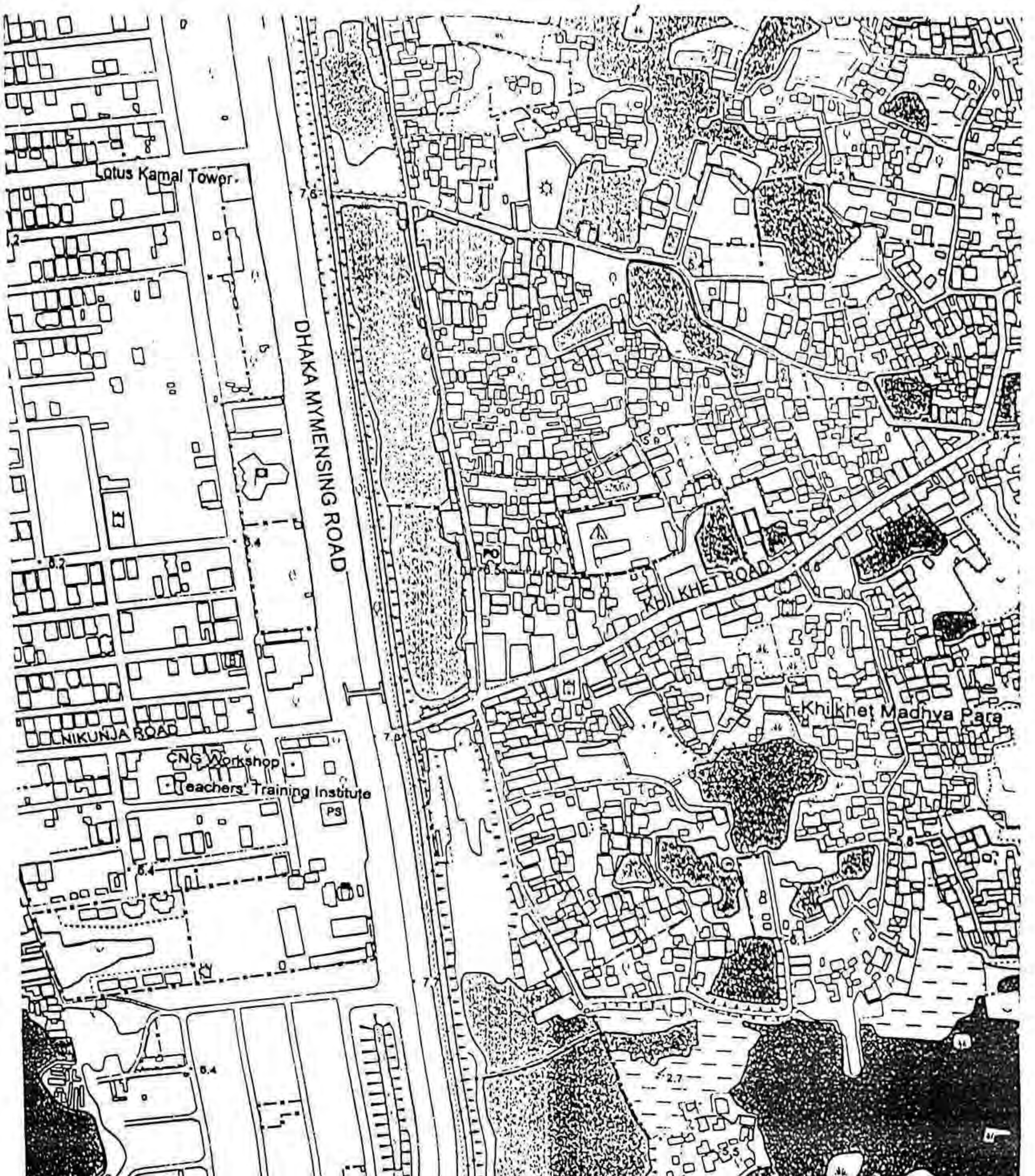


Figure I-6 : Map-6 (Khilkhet to Koala)

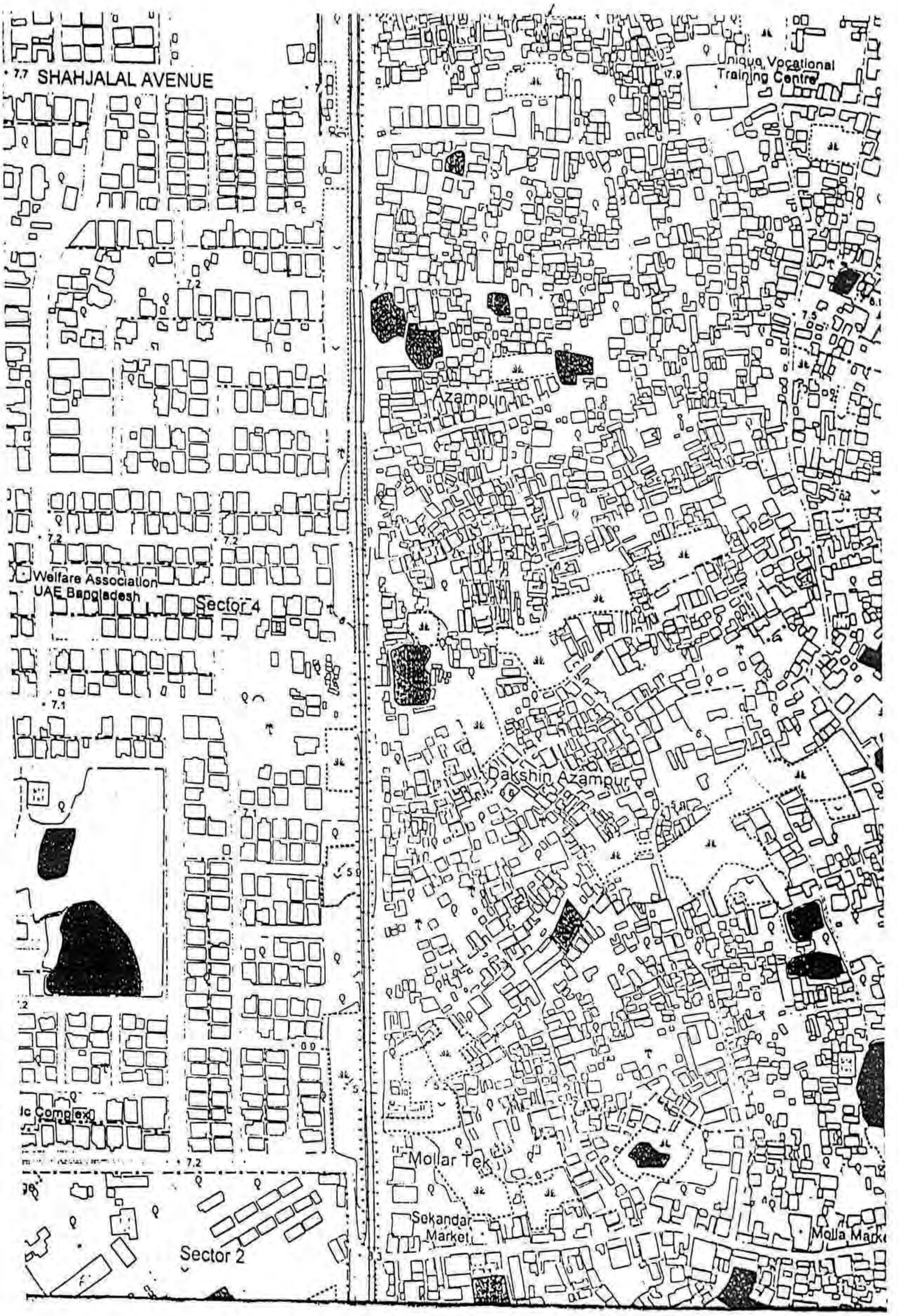


Figure I-7 : Map-7 (Ashkona to Azampur)

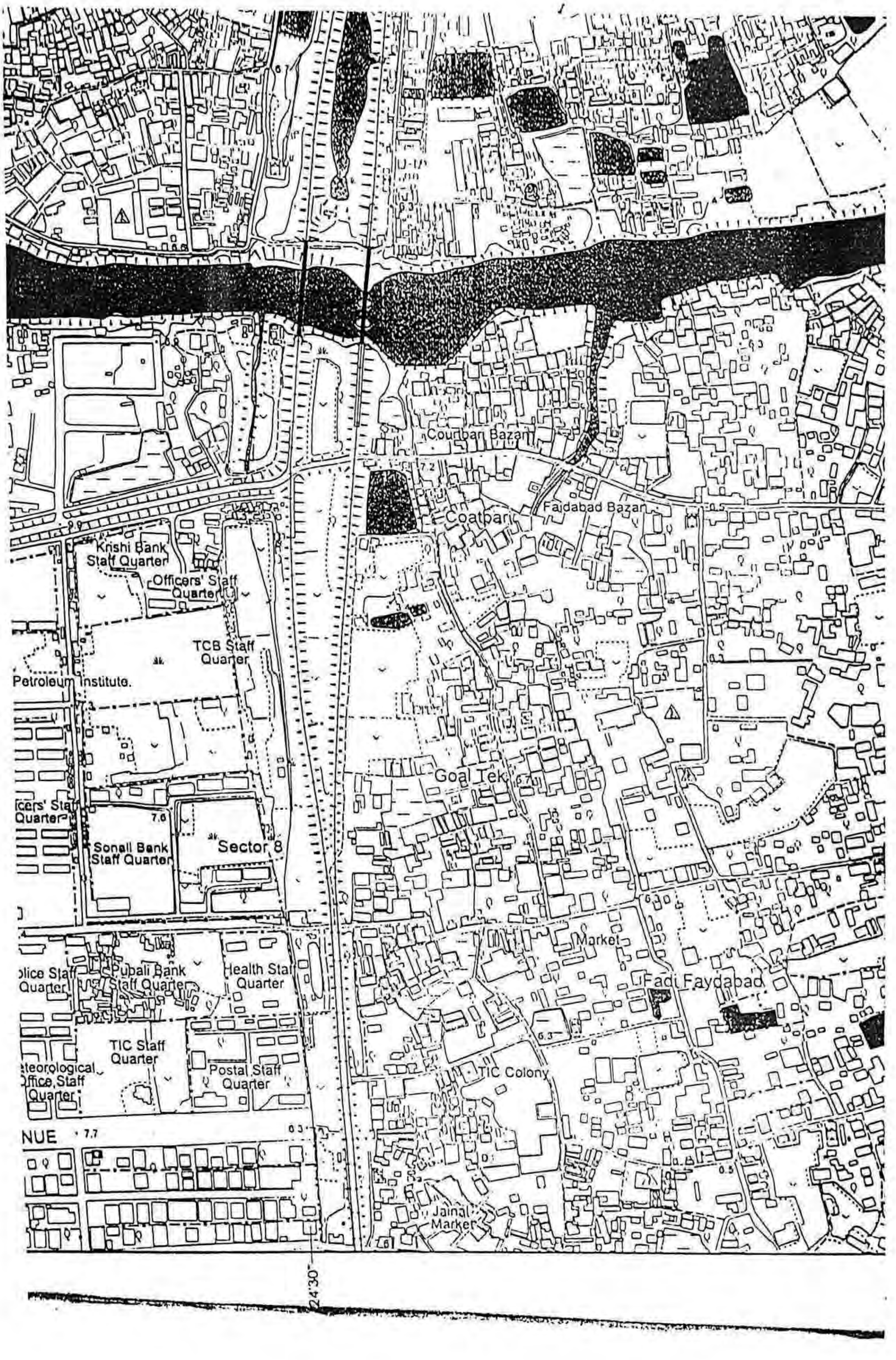


Figure I-8 : Map-8 (Coatbari to Tongi I/A)

Appendix –II

Design Calculation and Hot Rolled Steel Sections

APPENDIX -II

SAMPLE DESIGN CALCULATION AND HOT ROLLED STEEL SECTIONS

II-1 Design of Earth Retaining Systems:

Calculation for the ground condition represented in Case 1 has been shown here as the sample design calculation. The soil parameters for this condition are estimated as shown below:

Soil type	Depth variation (ft)	Depth Height (ft)	γ_d lb/ft^3	γ_{sat} lb/ft^3	γ' lb/ft^3	Min^m C_u (psf)	Max^m C_u (psf)	Ave. C_u (psf)	φ (deg)	K_a	K_p
Clay,C_4	0-20	20	111.57	138.35	75.85	2232	2520	2376	0	1	1
Sand,S_4	20-50	30	111.57	138.35	75.85	0	0	0	35	0.271	3.69

Earth pressures and wall forces are calculated based on the soil parameters for each of the earth retaining system.

(a) Sheet Pile :

Earth pressures against the sheet pile wall are calculated as shown below:

$p_1 = \gamma_{sat} L - 2\,C_u = 138.35 \times 0 - 2 \times 2376 = -\,4752$ psf

$p_2 = \gamma_{sat} L_1 - 2\,C_u = 138.35 \times 20 - 2 \times 2376 = -1985$ psf

$p_3 = \gamma_{sat} L_1 K_a = 138.35 \times 20 \times 0.271 = 749.86$ psf

$p_3 = \gamma_{sat} (L_1 + L_2) K_a = 138.35 \times 50 \times 0.271 = 1874.6$ psf

$p_5 = \gamma'(K_p - K_a) D_1 = 111.57 \times (3.69-0.271) \times D_1 = 381.46\, D_1$ psf

Earth pressures are shown in Figure II-1 neglecting the negative pressure. Resulting bending diagram is shown in Figure II-2, which reveal the maximum bending moment as 1031.6 k-ft per ft of wall length. For the maximum bending moment the sectional modulus of sheet pile is obtained as,

$$S_x = \frac{1031.6x12}{32} = 386.85 \text{ in}^3/\text{ft}$$

Sheet pile with such a high section modulus is not usually available.

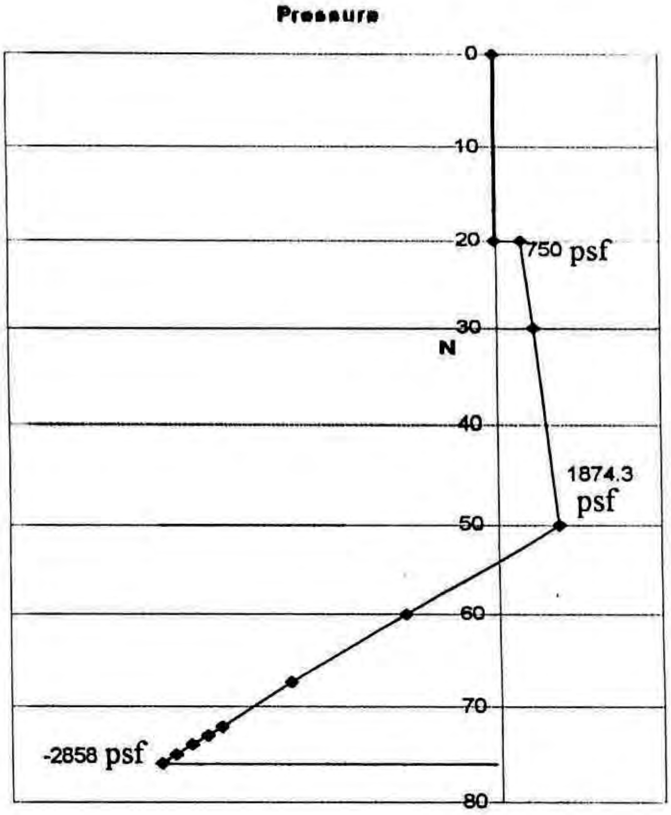


Figure II-1 : Pressure distribution diagram in Anchored Sheet Pile (Case 1)

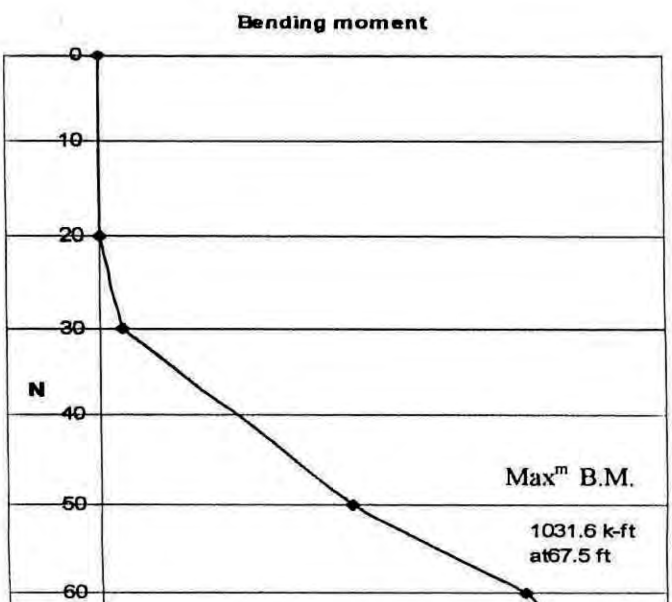


Figure II-2 : Bending moment diagram in Anchored Sheet Pile (Case 1)

(b) Braced Cut :

Equivalent unit weight of the soil is calculated according to Peck (1969) for the soil condition is in Case 1 as

$$\gamma_{ave} = (1/50) \times (138.35 \times 20 + 111.57 \times 30) = 122.3 \text{ lb/ft}^3$$

The equivalent undrained cohesion is calculated as,

$$C_{avo} = (1/(2 \times 50)) \times (20 \times .75 \times 2376 + 111.57 \times 1 \times 30^2 \times \tan 35)$$
$$= 1.06 \text{ ksf}$$

For the excavation depth of 50 ft,

$$\gamma_{ave} H / C_{ave} = (122.3 \times 50 / 1059.5) = 5.7716, \text{ which is greater than 4.}$$

Thus, the apparent pressure as per Peck (1969) would be as shown in Figure II-3, where

$$p_a = \gamma_{ave} H(1 - 4 C_{ave} / \gamma_{ave} H) = 122.3 \times 50 \times (1 - (4 \times 1059.5 / 122.3 \times 50)) = 1877 \text{ psf}$$

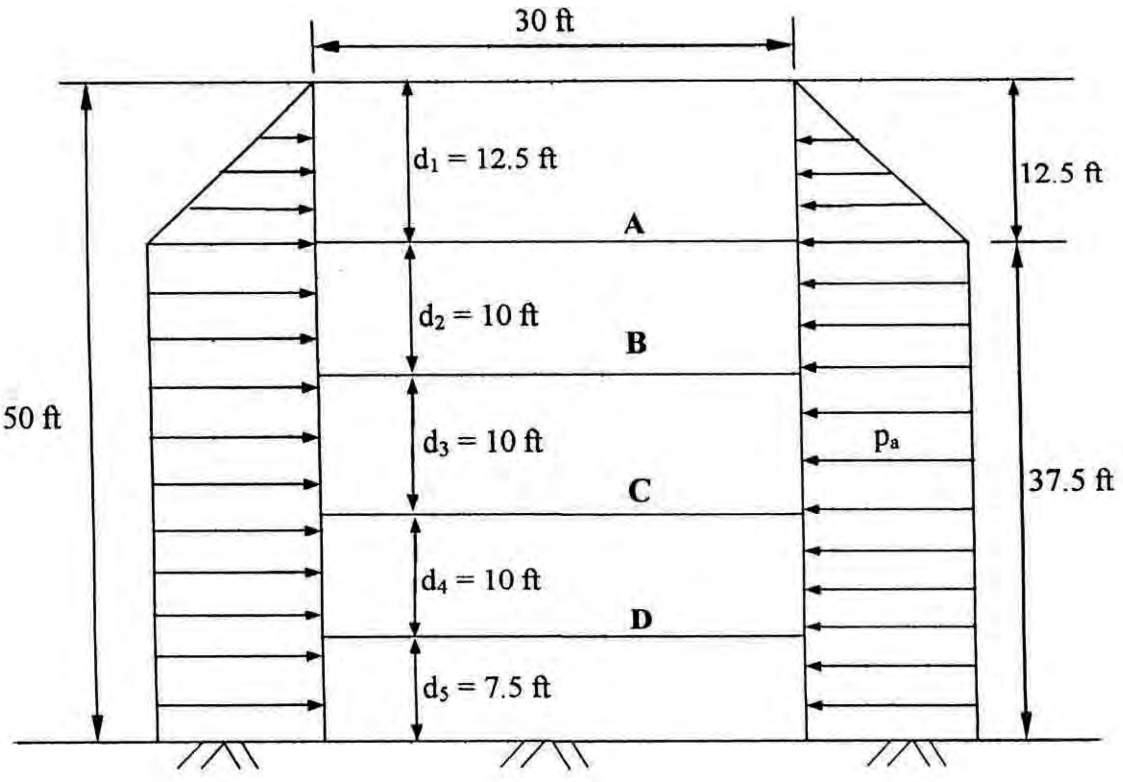


Figure II-3: Pressure distribution diagram in Braced Cut (Case 1)

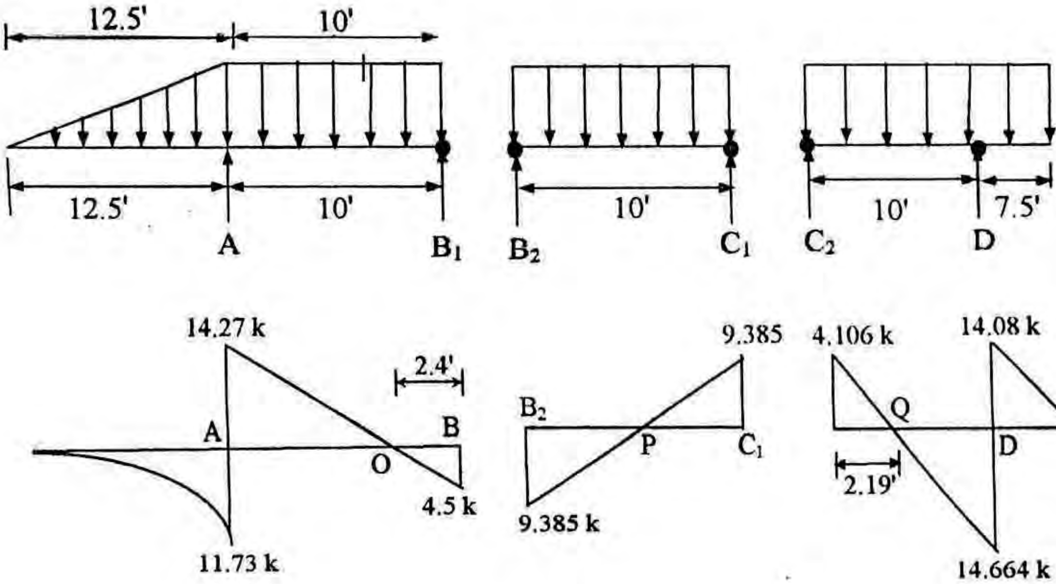


Figure II-4 : Pressure distribution and Shear force diagram in Braced Cut (Case 1)

The bending moments and the strut loads (A,B,C, and D) are calculated using the pressure diagram in Figure II-4 based on the assumption that the sheet piles or soldier beams are hinged at the strut level (Das, 1984) as shown below:

From $\Sigma M_{B1} = 0$,

$A = ((1/2)x12.5x1.877x14.17+1.877x10x(10/2))/10 = 26$ kip/ft and

$B_1 = 30.5-26 = 4.5$ kip/ft

From $\Sigma M_{C1} = 0$,

$B_2 = ((1.877x10x5)/10) = 9.385$ kip/ft and

$C_1 = 1.877x10-9.385 = 9.385$ kip/ft

Again from $\Sigma M_{D1} = 0$

$C_2 = (32.8475x1.25/10)$ kip $= 4.106$ kip/ft and

$D = 17.5x1.877- 4.106 = 28.7415$ kip/ft

Assuming the spacing of the strut as 10 ft center to center, the total strut loads are obtained as:

A = 26 x 10 = 260 kip

B =(4.5+9.385) x 10 = 138.85 kip

C = (9.385+4.106)x10 = 134.91 kip and

D = 28.7415x10 = 287.415 kip.

The points of zero shears are obtained for the maximum bending moment as shown below:

$x = (\text{reaction at } B_1/1.877) = (4.5/1.877) = 2.4$ ft

Thus the bending moment at different point on the wall of braced cut are given by:

$M_A = (1/2)x12.5x1.877)x(1x12.5/3)) = 48.88$ kip-ft/ ft of wall

$M_O = 4.5x2.4-1.877x(2.4)^2/2 = 5.39$ kip-ft/ft of wall

$M_P = 9.385x5-1.877x(5)^2/2 = 23.4625$ kip-ft/ft of wall

$M_Q = -4.106x2.19+1.877x2.19^2/2 = -4.491$ kip-ft/ft of wall

$M_D = -1.877x(7.5)^2/2 = -52.791$ kip-ft/ft of wall

Thus, the maximum moment is at point D. Section modulus for the maximum bending moment is given by:

$$S_x = (M_{max}/\sigma_{allow})$$

Or, $S_x = (52.791x12/24) = 26.39$ in^3/ft

Section AZ-17 gives $S_x = 30.97$ in^3/ft of wall, which can be used.

Wale Design :

For the wale at level A, $M_{max} = (A \times S^2/8) = (26x10^2/8) = 325$ kip-ft/ft of wall.

Thus, $S_x = (325x12/24) = 162.5$ in^3/ft of wall.

Section W 12x120 can be used for the wale that provide a $S_{x-x} = 163$ in^3/ft of wall.

For the wale at level B, $M_{max} = (B \times S^2/8) = (13.88x10^2/8) = 173.5$ kip-ft/ft of wall.

Thus, $S_x = (173.5x12/24) = 86.75$ in^3/ft of wall.

Section W 12x65 can be used for the wale that provide a $S_{x-x} = 87.9$ in^3/ft of wall.

For the wale at level C, $M_{max} = (C \times S^2/8) = (13.491x10^2/8) = 168.64$ kip-ft/ft of wall.

Thus, $S_x = (168.64x12/24) = 84.32$ in^3/ft of wall.

Section W 10x77 can be used for the wale that provide a $S_{x-x} = 85.9$ in^3/ft of wall.

For the wale at level D, $M_{max} = (D \times S^2/8) = (28.7415x10^2/8) = 359.3$ kip-ft/ft of wall.

Thus, $S_x = (359.3x12/24) = 179.63$ in^3/ft of wall

Section W 18x97 can be used for the wale that provide a S_{x-x}= 188 in^3/ft of wall

Strut Design : Use 60 GR Steel.

At A : Strut load = 260 kip.

Here, $F_a = \dfrac{149000}{[KL/r]^2}\ for \dfrac{KL}{r} \geq C_c$ and $F_a = \dfrac{F_y\left[1-\dfrac{1}{2}\left[\dfrac{KL/r}{C_c}\right]^2\right]}{\dfrac{5}{3}+\dfrac{3}{8}\dfrac{KL/r}{C_c}-\dfrac{1}{8}\left[\dfrac{KL/r}{C_c}\right]^3}\ for \dfrac{KL}{r} \leq C_c$

Now, $C_c = \pi\sqrt{\dfrac{2E}{F_y}}$ = 97.67631; and KL/r = 70.18 $<C_c$ Hence, F_a = 23.56 ksi

Therefore, p = p_s = $F_s A_{st}$ => Fs = (260/(11.8)) = 22.03 ksi<F_a (ok)

Section W 12x40 can be used for the strut that provide a Area = 11.8 in^2 and S_{x-x}= 51.9 in^3/ft of wall.

At B : Strut load = 138.85 kip.

Now, $C_c = \pi\sqrt{\dfrac{2E}{F_y}}$ = 97.67631; and KL/r = 102.56 $>C_c$ Hence, F_a = 14.16 ksi

Therefore, p = p_s = $F_s A_{st}$ => Fs = (138.85/(10.3)) = 13.47 ksi<F_a (ok)

Section W 8x35 can be used for the strut that provide area = 10.3 in^2 and S_{x-x} = 31.2 in^3/ft of wall.

At C : Strut load = 134.91 kip.

Now, $C_c = \pi\sqrt{\dfrac{2E}{F_y}}$ = 97.67631; and KL/r = 102.56 $>C_c$ Hence, F_a = 14.16 ksi

Therefore, p = p_s = $F_s A_{st}$ => Fs = (134.91/(10.3)) = 13.09 ksi<F_a (ok)

section W 8x35 can be used for the strut that provide area = 10.3 in^2 and S_{x-x} = 31.2 in^3/ft of wall.

At D : Strut load = 287.415 kip.

Now, $C_c = \pi\sqrt{\dfrac{2E}{F_y}}$ = 97.67631; and KL/r = 69.9 $<C_c$ Hence, F_a = 23.63 ksi

Therefore, p = p_s = $F_s A_{st}$ => Fs = (287.415/(13.2)) = 21.76 ksi<F_a (ok)

Section W 12x45 can be used Area A= 13.2 in^2, S_{x-x} = 58.1 in^3/ft of wall.

(c) Bored Pile :

From Braced Cuts, Maximum moments at D, $M_D = 52.791$ kip-ft/ft of wall.

When axial force or vertical force on the pile is zero, $e/h = \infty$. Using Appendix A (Graph A.15) of Nelson and Winter (1986) for Column Strength interaction diagram for circular column with $\gamma = 0.75$, $\rho_g = 0.02$ and $\varphi P_n/A_g = 0$,

$$\varphi M_n / A_g h = M_u / A_g h = 0.36$$

Where h is the pile diameter.

Assuming the spacing between the pile as 1.5 ft,

$$A_g h = (52.791 x 12 x 1.5)/0.36$$

This gives,

$$h = ((2639.55 x 4)/(\pi))^{1/3} = 14.98 \text{ inch.} \approx 16 \text{ inch.}$$

$$P = A_g (0.25x4+24x0.02) = (\pi/4)x16^2 \, x1.48 = 297.57 \text{ kip}$$

$$P_c = 0.25x \, A_g \, f_{c'} = 0.25x(\pi/4)x15^2 \, x \, 4 = 201.06 \text{ kip}$$

$$P_s = P - P_c = 96.51 \text{ kip}$$

$$A_{st} = P_s / f_s = (96.51/24) = 4.02 \text{ in}^2$$

If # 8 bar is used the number of bar is calculated as = (4.02/0.79) = 5.1 ≈ 6 Nos.

Actual area of steel is thus, $A_s = 6x0.79 = 4.74 \text{ in}^2$, where

$\rho_s = 0.45x((A_g/A_c) - 1)x(f_{c'}/f_y) = 0.45x(((\pi/4)x16^2)/((\pi/4)x12^2) - 1)x(4/60)) = 0.0233$

$$g \leq (1/6)x \text{ Core dia.} = (1/6)x12 = 2 \text{ inch.}$$

$$\rho_s = 4a_s / gDc = (4x(\pi/4)x(0.375)^2)/(1.5x12) = 0.0245 > 0.0233$$

This is obtained using #3 bar @ 1.5 inch c/c as stirrup

(d) Diaphragm Wall :

From Braced Cuts, Maximum moments at D, $M_D = 52.791$ kip-ft/ft of wall.

Column Strength interaction diagram for rectangular section with $\gamma = 0.75$ was used for design of the wall.

For $\rho_g = 0.02$ and $\varphi P_n/A_g = 0$,

$\varphi M_n/A_g h = M_u/A_g h = 0.42$

Thus, $A_g h = (52.791x12x10)/0.42 = 15083.143$

Assuming the width of Diaphragm Wall, b = 10' = 120" (hence the spacing),

⇨ $h = (15083.143/120)^{1/2} = 11.21$ inch. ≈ 12 inch.

P = 0.85xbxh (0.25x4+24x0.02) = 0.85x120x12 x1.48 = 1811.52 kip

P_c = 0.85x 0.25x4x120x11 = 1224 kip

$P_s = P - P_c$ = 587.52 kip

$A_{st} = P_s/f_s$ = 587.52/(0.85x24) = 28.8 in^2

Use #8 bar, No. of bar = 28.8/ 0.79 = 36.46 ≈ 37 Nos. (in both sides)

No. of bar in one side = 37/2 = 18.5 ≈ 19 Nos.

Spacing = (120-3)/19 = 6.16 inch ≈ 6 inch.

Use #8 bar @ 6 inch c/c.

Tie Design :

Using #3 bar for tie,

Spacing of tie bar = 16x(8/8) = 16 inch = 48x(3/8) = 18 inch

Least dimension of col^m = 12 inch

Use #3 bar @ 12 inch c/c

Construction sequence (NAVFAC Design Manual)

For Case 1:

Assumptions (1) No surcharge Load (2) No wall friction.

A. Stage 1 (Prior to installation of Brace 1)

Sheeting acts as cantilever wall.

Since the active earth pressure is negative, so the maximum moment for stage 1 is only for water pressure. Therefore, Maximum moment,

$M_{max} = (\frac{1}{2} x0.0625x13^2 x(\frac{2}{3} x13)) = 46 ft - kips$

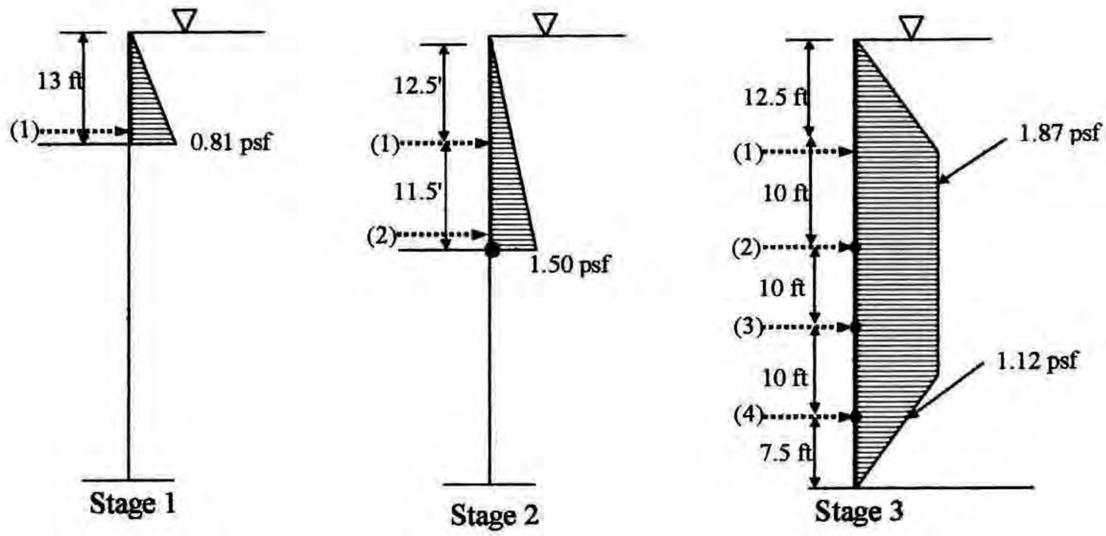


Figure II-5 : Pressure distribution diagram for Case-1

B. Stage 2

1. Active Pressure :

At water level, $\sigma_A = \gamma H - 2C_U = (75.85x24 - 2x1060)/1000 = -299.6psf = -0.299psf$

Water pressure on active side, $P_w = 62.5x24 = 1.5ksf$

Total pressure = $\sigma_a + P_w = 1.5ksf$ (Omitted negative active earth pressure)

2. Point of zero net pressure: $D = \dfrac{1.5}{((4x1060 - 75.85x24)/1000)} = 0.62ksf$

3. Reaction at (1) and (A) per linear foot of wall. Assume hinge (Zero bending moment) at A : R(1) = $((1/2)x1.5x24x(1/3)x24)/11.5 = 12.52kip$ Using R(1) = $1.15x12.52 = 14.4kip$; R(A) $((1/2)x1.5x24) - 12.52 = 5.48kip$

4. Point of zero shear :

$12.52 - (1/2)xS_o x(1.5xS_o x/24) = 0$; $S_o = \sqrt{12.52/0.03125} = 20.016ft$

5. Maximum moment :

$M_{max} = 12.52x(20.01 - 12.5) - ((1/2)x20.01x(1.5x20.01/24)x(20.01/3)$

$= 94.1 - 83.53 = 10.57kip - ft$

C. Final Stage :

Pressure distribution : $\gamma' H / C_U = (75.95x50/1060) = 3.58 < 4$

$\sigma_h = \gamma_{ave} H(1 - 4C_{ave} / \gamma_{ave} H) = 122.3x50(1 - {4x1059.5}/{122.3x50}) = 1877 psf = 1.87 ksf;$

2. Strut loads per linear foot of wall: The analysis of strut load is similar to the case -1, which has solved before.

$$R(1) = ((1/2x12.51.87x14.17 + 1.87x10x(10/2))/10 = 26 kip$$

$$R(2) = (4.5 + 9.39) = 13.89 kip$$

$$R(3) = (9.39 + 4.106) = 13.5 kip$$

$$R(4) = (17.5 + 1.87 - 4.106) = 28.74 kip$$

3. Moment: The points of zero shears are obtained for the maximum bending moment as shown below:

x = (4.5/1.877) = 2.4 ft

Thus the bending moment at different point on the wall of braced cut are given by:

M_1 = (1/2)x12.5x1.877)x(1x12.5/3)) = 48.88 kip-ft/ ft of wall

M_{1-2} = 4.5x2.4-1.877x$(2.4)^2$/2 =5.39 kip-ft/ft of wall

M_{2-3} = 9.385x5-1.877x$(5)^2$/2 = 23.4625 kip-ft/ft of wall

M_{3-4} = -4.106x2.19+1.877x2.19^2/2 = -4.491 kip-ft/ft of wall

M_4 = -1.877x$(7.5)^2$/2 = -52.791 kip-ft/ft of wall

Thus, the maximum moment is at point 4.

D. Summary :

Construction Stage	Strut loads (kip)		Moments (ft-kips)
I	-		46
II	R(1) = 14.4		10.57 between (1) and (4)
Final	R(1) = 26	R(2) = 13.89	52.79 between (3) and (4)
	R(3) = 13.49	R(4) = 28.74	

For Case 3 :

Assumptions (1) No surcharge Load (2) No wall friction.

A. Stage 1 (Prior to installation of Brace 1)

Sheeting acts as cantilever wall.

Since the active earth pressure is negative, so the maximum moment for stage 1 is only for water pressure. Therefore, Maximum moment,

$M_{max} = (\frac{1}{2} x0.0625x13^2 x(\frac{2}{3} x13)) = 46 ft - kips$

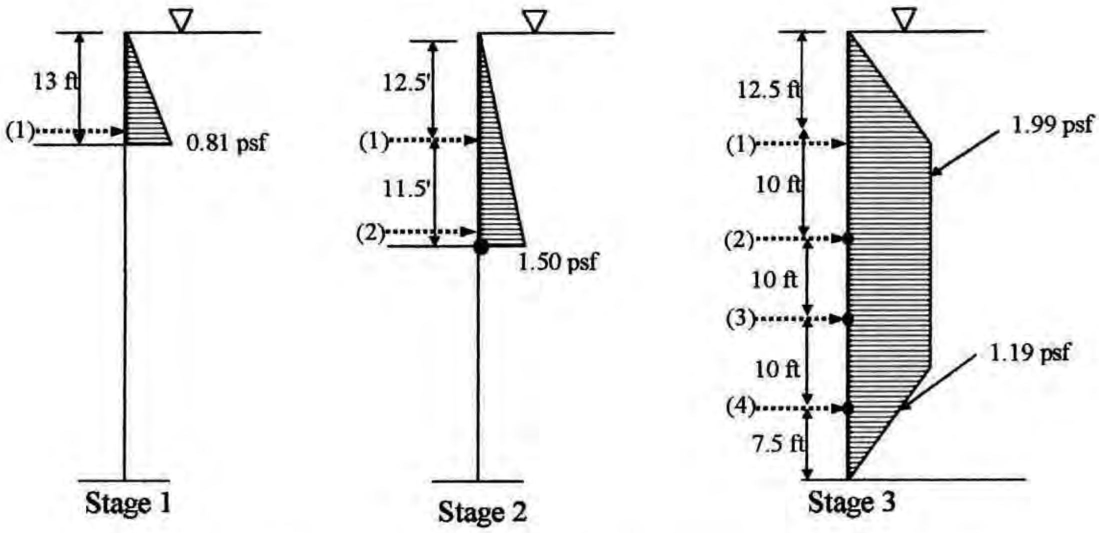


Figure II-6 : Pressure distribution diagram for Case-1

B. Stage 2

1. Active Pressure :

At water level, $\sigma_A = \gamma H - 2C_U = (70.48x24 - 2x1260)/1000 = -828.5 psf = -0.83 psf$

Water pressure on active side, $P_w = 62.5x24 = 1.5 ksf$

Total pressure = $\sigma_a + P_w = 1.5 ksf$ (Omitted negative active earth pressure)

2. Point of zero net pressure: $D = \dfrac{1.5}{((4x1260 - 70.48x24)/1000)} = 0.45 ksf$

3. Reaction at (1) and (A) per linear foot of wall. Assume hinge (Zero bending moment) at A : R(1) = $((1/2)x1.5x24x(1/3)x24)/11.5 = 12.52 kip$ Using R(1) = $1.15x12.52 = 14.4 kip$; R(A) $((1/2)x1.5x24) - 12.52 = 5.48 kip$

4. Point of zero shear :

$12.52 - (1/2)xS_o x(1.5xS_o x/24) = 0$; $S_o = \sqrt{12.52/0.03125} = 20.016 ft$

5. Maximum moment :

$M_{max} = 12.52x(20.01 - 12.5) - ((1/2)x20.01x(1.5x20.01/24)x(20.01/3)$

$= 94.1 - 83.53 = 10.57 kip - ft$

C. Final Stage :

Pressure distribution : $\gamma' H / C_U = (70.48x50/1260) = 2.8<4$

$\sigma_h = 0.3\gamma H = 0.3x132.98x50 = 1995psf = 1.99ksf$;

2. Strut loads per linear foot of wall : The analysis of strut load is similar to the case -3, which has solved before.

$R(1) = ((1/2)x12.5x1.995x14.17 + 1.995x10x(10/2))/10 = 27.63kip$

$R(2) = (4.88 + 9.975) = 14.86kip$

$R(3) = (9.975 + 4.36) = 14.335kip \quad R(4) = (17.5x1.995 - 4.36) = 30.55kip$

3. Moment: The points of zero shears are obtained for the maximum bending moment as shown below:

x = (4.88/1.995) = 2.4 ft

Thus the bending moment at different point on the wall of braced cut are given by:

M_1 = (1/2)x12.5x1.995)x(1x12.5/3)) = 51.94 kip-ft/ ft of wall

M_{1-2} = 4.78x2.4-1.995x$(2.4)^2$/2 = 5.72 kip-ft/ft of wall

M_{2-3} = 9.97x5-1.995x$(5)^2$/2 = 24.91 kip-ft/ft of wall

M_{3-4} = -4.36x2.19+1.995x2.19^2 /2 = -4.77 kip-ft/ft of wall

M_4 = -1.995x$(7.5)^2$ /2 = -56.11 kip-ft/ft of wall

From the above moments, M_4 is maximum.

D. Summary :

Construction Stage	Strut loads (kip)		Moments (ft-kips)
I	-		46
II	R(1) = 14.4		10.57 between (1) and (4)
Final	R(1) = 27.63	R(2) = 14.75	56.11 between (3) and (4)
	R(3) = 14.34	R(4) = 30.55	

II-2 Design of Tunnel Structure:

Metro Rail Tunnel for a typical locations in Dhaka City is design considering the soil condition encountered along the route. Table shows the soil data used in the design.

Design was conducted using the maximum long term horizontal pressure to represent the worst case scenario.

Soil type	Depth variation (ft)	Depth Height (ft)	γ_d lb/ft^3	γ_{sat} lb/ft^3	γ' lb/ft^3	Min^m C_u (psf)	Max^m C_u (psf)	Ave. C_u (psf)	φ (deg)	K_a	K_p
Clay,C_2	0-65	65	104.71	132.98	70.58	1008	1512	1260	0	1	1
Sand,S_5	65-100	35	113.64	139.51	77.11	0	0	0	40	0.217	4.59

Here, the maximum Long term horizontal pressure coefficient, $K_{max} = 1.3 \times K_o = 1.3x(1-\sin\varphi)$ For $\varphi = 0$, $K_{max} = 1.3$

Full Vertical Roof Load :

(i) Earth Cover = 32x132.98 = 4255.36 psf

(ii) Roof Slab = 150x2.5 = 375 psf

(iii) Surcharge q = 300 psf

Total Roof Load = 4930.36 psf ≈ 4931 psf

(a) Vertical Invert Reaction : (Cases 1 and 3 in Figure 6 and 7)

(i) Total Roof Load = 4930.36 psf

(ii) Weight of Walls (((1/2)x2.5x2.5x8+3x2x13)x150/30) = 515.00 psf

Total Load = 5445.36 psf

≈ 5446 psf

(b) Vertical Invert Reaction : (Cases 2)

Modify invert reaction as required to account for unbalanced horizontal loading.

(c) Vertical Stresses in Soil :

At elevation b, p_1 = 132.98x(32+2.5/2) = 4421.585 psf ≈ 4422 psf

At elevation d, p_1 = 132.98x(32+2.5/2)+132.98x15.5 = 6482.775 psf ≈ 6483 psf

(c) Horizontal Pressure on Tunnel Structure :

1. Let, p_2 = Maximum Long-term horizontal pressure :

At elevation b, $p_2 = p_1 \times K_{max} = 4421.585 \times 1.3 = 5748.06$ psf $\approx$ 5749 psf

At elevation d, $p_2 = p_2 \times K_{max} = 6482.775 \times 1.3 = 8427.6075$ psf $\approx$ 8428 psf

2. Let, p_3 = Medium Underwater Short-term horizontal pressure :

At elevation b, $p_3 = p_1 \times K_a = 4421.585 \times 1 = 4421.585$ psf $\approx$ 4422 psf

At elevation d, $p_3 = p_2 \times K_a = 6482.775 \times 1 = 6482.775$ psf $\approx$ 6483 psf

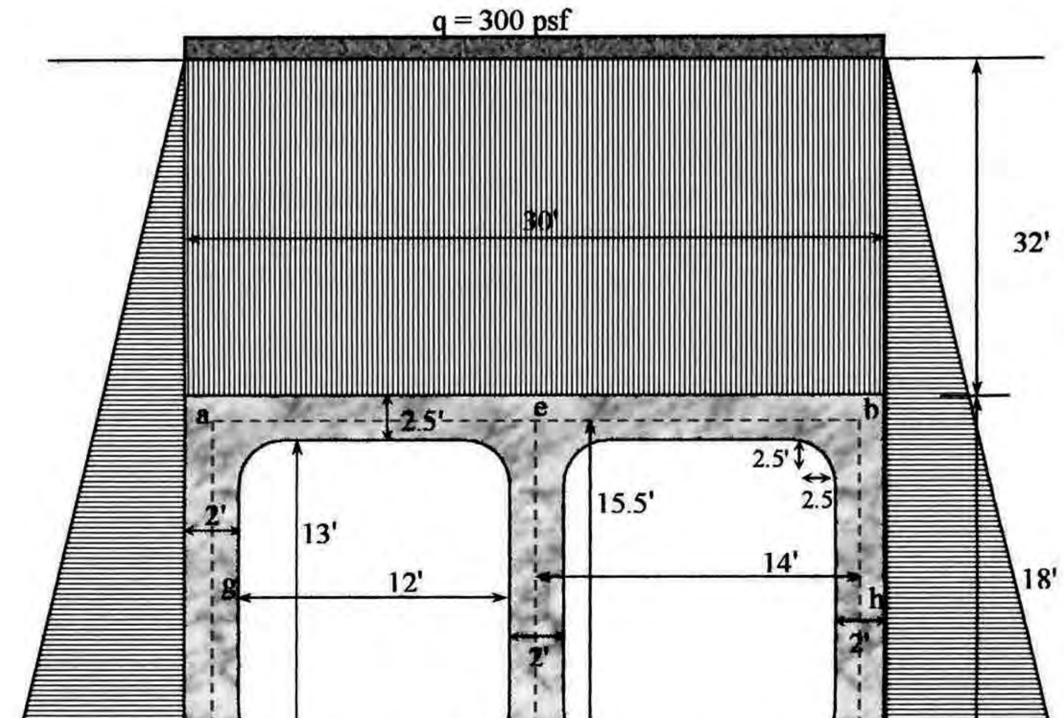


Figure II-7: Vertical & Horizontal pressure effect on Tunnel Structure.

Moment Distribution : (For Case – 1)

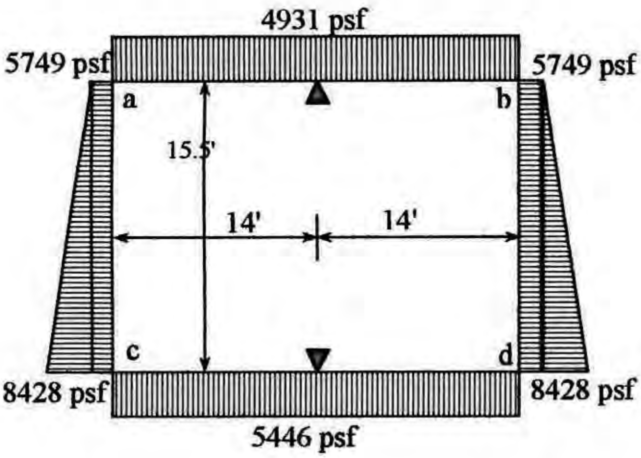


Figure II-8 (a) Pressure distribution diagram for Case – 1 (Long-Term & Balanced Condition)

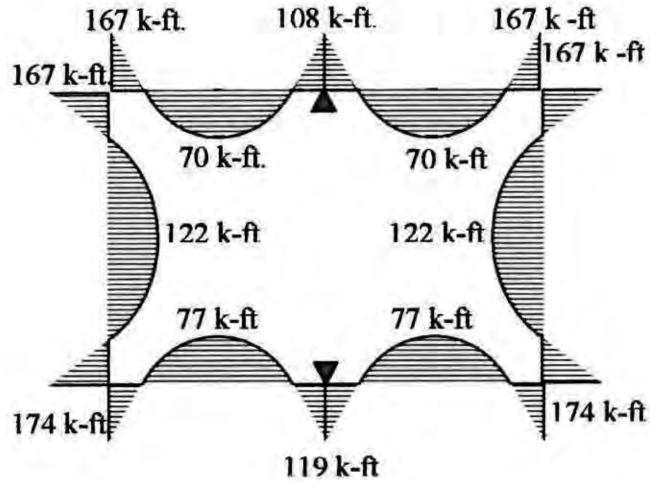


Figure II-8 (b) Moment distribution diagram for Case – 1 (Long-Term & Balanced Condition)

3. Let, p_4 = Active earth pressure if ground water is dewatered.

At elevation b, $p_4 = K_a \times (\gamma H + q) = 1x(104.71x33.25+300) = 3781.61$ psf ≈ 3782 psf

At elevation d, $p_4 = K_a \times (\gamma H + q) = 1x(104.71x48.75+300) = 5404.61$ psf ≈ 5405 psf

For Case – 2 :

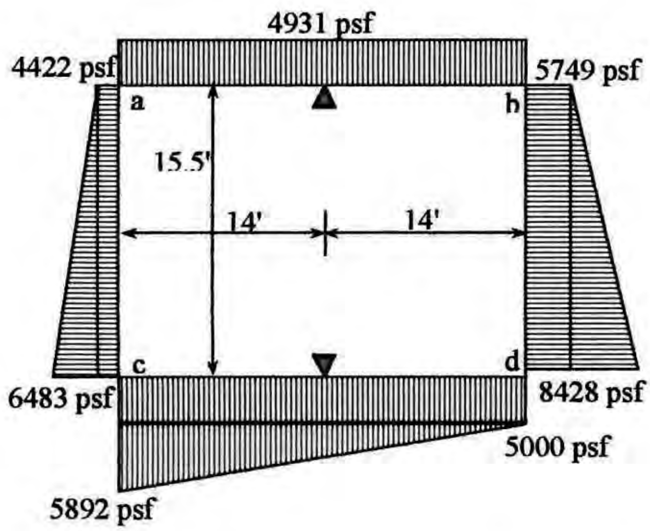


Figure II-9: Pressure distribution diagram for (Long & Short Term & Un-Balanced Condition)

For Case – 3 :

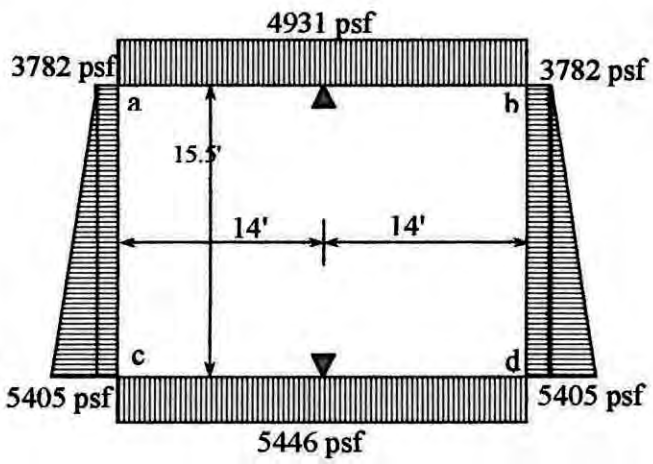


Figure II-9: Pressure distribution diagram for (De-Watered & Balanced Condition)

On the basis of Horizontal pressure acting on Tunnel Structure, if considering the Long-term effect then it satisfied from all conditions of Factor of Safety. So, Case–1 is described here:

(1) **Negative Moment,** M_a : at interior faces of exterior supports for members built integrally with their supports, where the support is a column = (1/16) x(4931/1000)x14^2 + (1/16)x(((5749+8428)/2)/1000)x15.5^2 = 166.84 ≈ **167 kip-ft = M_b**

(2) **Negative Moment,** M_e : at exterior faces of first interior support for two spans = (1/9) x(4931/1000)x14^2 = 107.39 ≈ 108 kip-ft.

(3) **Positive Moment** (between a & e) **: M_{a-e} = If** discontinuous end is integral with the support = (1/14) x(4931/1000)x14^2 = 69.034 ≈ 70 kip-ft. M_{e-b}

(4) **Positive Moment,** M_g and M_h **:** M_g = (1/14) x(((5749+8428)/2)/1000)x15.5^2 = 121.64 ≈ 122 kip-ft. = M_h

(5) **Negative Moment,** M_c and M_d **:** M_c = (1/16) x(5446/1000)x14^2 + (1/16)x(((5749 +8428)/2)/1000)x15.5^2 = 173.15 ≈ 174 kip-ft. = M_d

(6) **Negative Moment,** M_f **:** M_f = (1/9) x(5446/1000)x14^2 = 118.6 ≈ 119 kip-ft.

(7) **Positive Moment** (between c & f) **:** M_{c-f} = (1/14) x(5446/1000)x14^2 = 76.24 ≈ 77 kip-ft = M_{f-d}

<u>Depth Check</u> :

$$\rho_{max} = 0.75\,\rho_b = 0.75 \times 0.85^2 \times (4/60) \times (87/(87+60)) = 0.0214$$

For maximum moment at c, $d^2 = (M_u/(\varphi \rho f_y b(1-0.59\rho f_y/f_{c'}))$

$= ((174 \times 12)/(0.90 \times 0.0214 \times 12 \times 60(1-(0.59 \times 0.0214 \times 60/4))) = 185.883$ in^2

⇨ d = 13.63 inch ≈ 14 inch,

Which is less than the effective depth of, d = 2.5x12-3 = 27 inch.

If the stress-block depth, a = 3 inch, then area of steel required per foot of width in the top of the slab is, $As_1 = M_u/(\varphi f_y(d-a_1/2) = (167 \times 12/(0.90 \times 60 \times (27-3/2))) = 1.46$ in^2

$a_1 = A_s f_y/(0.85 f_{c'} b) = (1.46 \times 60/(0.85 \times 4 \times 12) = 2.15$ **inch**

As_2 = 1.43 in^2 ; a_2 = 2.105 inch; As_3 = 1.43 in^2 ; a_3 = 2.103 inch;

Hence, **For Roof Slab ab :**

at point a, As_a = 1.43 in^2 = As_b (at point b) ;

Use # 8 No. bar @ (0.79x12/1.43) = 6.63 ≈ 6.5 in. c/c

between point a and e, As_{a-e} = 0.6 in^2 = As_{e-b} (between point e and b);

Use # 8 No. bar @ (0.79x12/0.60) = 15.8 ≈ 15.75 in. c/c

at point e, As_e = 0.925 in^2. Use # 8 No. bar @ (0.79x12/0.925) = 10.249 ≈ 10 in. c/c

For Bottom Slab cd :

at point c, As_c = 1.49 in^2 =As_d(at point d) ;

Use # 8 No. bar @ (0.79x12/1.49) = 6.36 ≈ 6.25 in. c/c

between point c and f, As_{c-f} = 0.66 in^2 = As_{f-d}(between point f and d);

Use # 8 No. bar @ (0.79x12/0.66) = 14.36 ≈ 14.25 in. c/c

at point f, As_f = 1.02 in^2. Use # 8 No. bar @ (0.79x12/1.02) =9.29 ≈ 9.25 in. c/c

For Left Wall ac : Stress-block depth,

$As_1 = M_u/(\varphi f_y(d-a_1/2)$ = (167x12/(0.90x60x(21-3/2))) = 1.903 in^2 ; $a_1 = A_s f_y/(0.85 f_c' b)$ = (1.903x60/(0.85x4x12) = 2.8 inch; As_2 = 1.89 in^2 ; a_2 = 2.78 inch; As_3 = 1.89 in^2 ; a_3 = 2.78 inch;

Hence, **For Left Wall ac and For Right Wall bd :**

at point a, As_a = 1.89 in^2 =As_b(at point b) ; but for roof slab As_a = 1.43 in^2 Required As_a = (1.89-1.43) = 0.46 in^2 Use # 4 No. bar @ (0.2x12/0.46) = 5.22 ≈ 5 in. c/c

Between point a and c, As_{a-c} = 1.383 in^2 = As_{b-d}(between point b and d); Use # 8 No. bar @ (0.79x12/1.383) = 6.85 ≈ 6.75 in. c/c

At point c, As_c = 1.972 in^2 = at point d. but for bottom slab As_c = 1.49 in^2 Required As_c = (1.972-1.49) = 0.482 in^2 Use # 4 No. bar @ (0.2x12/0.482) = 4.98 ≈ 4.75 in. c/c

Minimum Reinforcement for control of Shrinkage and Temperature Cracking is :
For Top and Bottom Slab, As = 0.0018x12x30 = 0.648 $inch^2$ per 12 inch strip. Use # 5 No. bar @ (0.31x12/0.648) = 5.74 ≈ 5.5 in. c/c i.e, 11" c/c. both upper & lower direction.
For Left and Right Wall, As = 0.0018x12x24 = 0.52 $inch^2$ per 12 inch strip. Use # 5 No. bar @ (0.31x12/0.52) = 7.15 ≈ 7 in. c/c. 14" c/c. both upper & lower direction.

<u>Shear Check</u> :

(i) For Top Slab, ab :

A small increase in the amount of steel used at the exterior support is,

V_u = 1.15x(4931x14/2)-(4931x27/12) = 28599.8 lbs = 28.6 kips

The Nominal Shear strength of the concrete slab is,

$V_u = V_c = 2(f_{c'})^{1/2}bd = 2x(4000)^{1/2}x12x27 = 40983.12$ lbs = 40.98 kips

The Design Strength of the concrete slab, φV_c = 0.85x40983.12 = 34835.65 lbs = 34.84 kips is well above the required strength in Shear of V_u = 28599.8 lbs = 28.6 kips.

(ii) For Bottom Slab, cd :

V_u = 1.15x(5446x14/2)-(5446x27/12) = 31590 lbs = 31.59 kips

$\varphi V_c = 0.85x2(f_{c'})^{1/2}bd = 0.85x2x(4000)^{1/2}x12x27 = 34840$ lbs = 34.84 kips

(ii) For Wall ac & cd :

V_u = 1.15x((5.749+8.428)/2)x(15.5/2)-(7.09x21/12) = 50.77 kips

$\varphi V_c = 0.85x2(f_{c'})^{1/2}bd = 0.85x2x(4000)^{1/2}x12x21 = 27.094$ kips $< V_u$.

So, Stirrup will be required. Use **#3** bar.

Spacing, S = ((0.85x0.22x60x21)/(50.77-27.094)) = 9.95″ ≈ 9.75″ c/c.

S_{max} = 24″ or S_{max} = d/2 = (21/2) = 10.5″ c/c. or S_{max} = (0.22x60000/(50x12)) = 22″ c/c.

Average Spacing, S = (9.75+10.5)/2 = 10.125″ ≈ 10″ c/c. Use **#3** @ 10″ c/c.

<u>Design of Middle Column</u> :

(i) If Tied Column is used :

let, b = 4 inch, h = 24 inch

P = 4931x14x10 = 690340 lbs = 691 kips

P_c = 0.85x 0.25x4x24x24 = 489.6 kips = 490 kips

$P_s = P-P_c$ = (691-490) = 201 kips

$A_{st} = P_s/ f_s$ = (201/(0.85x24)) = 9.853 $inch^2$

Use #8 bar, No. of bar = (9.853/0.79) = 12.47 Nos = 13 Nos

For similarity, **Use # 8 bar 12 Nos and # 5 bar 2 Nos**

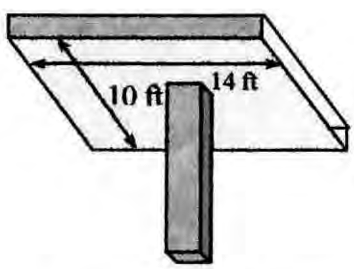


Figure II-10: 3-D view of Top slab & Col^m joint

<u>Tie Design</u> :

Using #3 bar.

Spacing of tie bar = 16x(8/8) = 16 inch. = 48x(3/8) = 18 inch

Least dimension of col^m = 24 inch

Use #3 bar @ 16 inch c/c

(ii) If Spiral Column is used :

P = 691 kips

$P_c = 0.25x\ A_g\ f_{c'} = 0.25x(\pi/4)x24^2\ x\ 4 = 452$ kips

$P_s = P - P_c = 239$ kips

$A_{st} = P_s / f_s = (239/24) = 9.96\ in^2$

Use # 8 bar, No. of bar = 9.96/ 0.79 = 12.61 ≈ 13 Nos

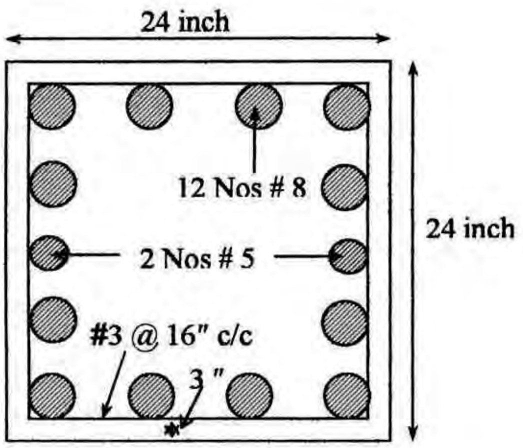


Figure II-11 (a) Cross section of Middle Tied Column .

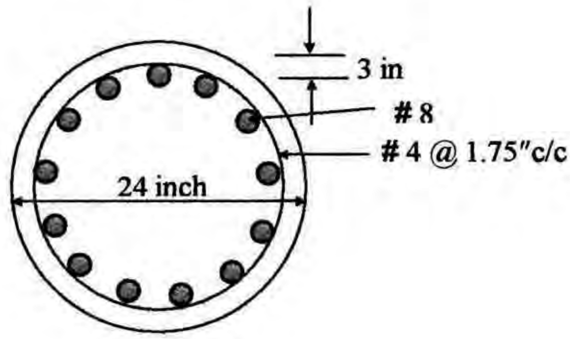


Figure II-11 (b) Cross section of Middle Spiral Column.

Actual areas, $A_s = 13x0.79 = 10.27\ in^2$

$\rho_s = 0.45x((A_g/A_c) - 1)x(f_{c'}/f_y) = 0.45x(((\pi/4)x24^2)/((\pi/4)x21^2) - 1)x(4/60)) = 0.00918$

$g \le (1/6)x$ Core dia. $= (1/6)x21 = 3.5$ inch.

$\rho_s = 4a_s / gDc = (4x(\pi/4)x(0.375)^2)/(2.25x21) = 0.00935 > 0.00918$

Use # 3 bar @ 2.25 inch c/c.

II-3 Design of Station Building:

(a) Design of Slab :

Roof Slab :

1) Load Calculation :

Dead load :

(i)	Self load	= (5.5/12)x150 = 68.75	psf
(ii)	Lime concrete	= (4/12)x120 = 40	psf
(iii)	Floor finish	= 15	psf
	Total dead load	=123.75	psf

Live load : For Habitual Atties (Table 1.1, Design of Concrete Structures- Arthur H. Nilson, Twelfth edition,) = 30 psf

Total factored load = 1.4 x D.L + 1.7 x L.L = 1.4 x 123.75 + 1.7 x 30 = 173.75 + 51 = 224.25 psf

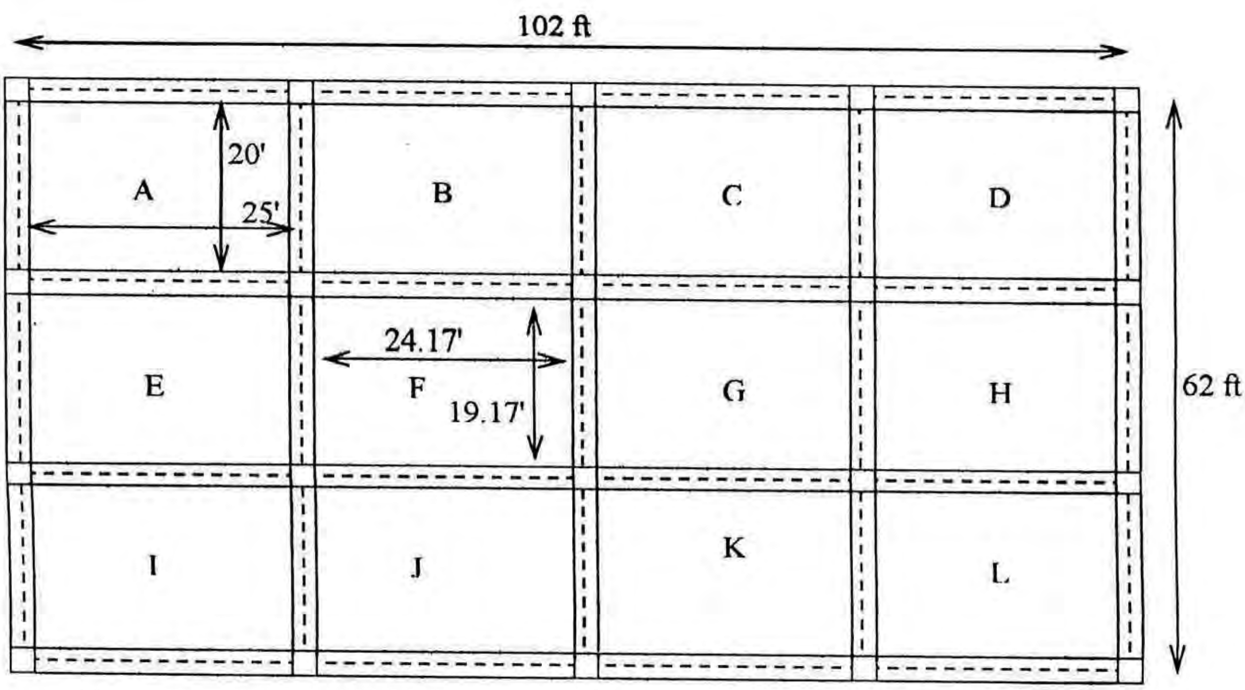


Figure II-12: Grid showing station building.

2) Moment Calculation :

(i) For Shorter Direction :

Negative moment at continuous edge :

- For A type Slab, $M_{a\,(-ve)} = C_{a\text{-}neg.}\, wl_a^2 = 0.071x224.25x19.17^2 = 5.851$ kip-ft
- For B type Slab, $M_{b\,(-ve)} = C_{a\text{-}neg.}\, wl_a^2 = 0.055x224.25x19.17^2 = 4.533$ kip-ft
- For E type Slab, $M_{e\,(-ve)} = C_{a\text{-}neg.}\, wl_a^2 = 0.075x224.25x19.17^2 = 6.181$ kip-ft
- For F type Slab, $M_{f\,(-ve)} = C_{a\text{-}neg.}\, wl_a^2 = 0.065x224.25x19.17^2 = 5.36$ kip-ft

Positive moment at Mid-Span Section :

- For A type Slab, $M_{a\,(+ve)} = C_a.dl\ wl_a^2 + C_a.ll.\ wl_a^2 = 0.039x173.75\ x19.17^2 + 0.048x51x19.17^2$ kip-ft = 3.383kip-ft
- For B type Slab, $M_{b\,(+ve)} = 0.032x173.75\ x19.17^2 + 0.044x51x19.17^2$ kip-ft = 2.862 k-ft
- For E type Slab, $M_{e\,(+ve)} = 0.029x173.75\ x19.17^2 + 0.042x51x19.17^2$ kip-ft = 2.634 k-ft
- For F type Slab, $M_{f\,(+ve)} = 0.026x173.75\ x19.17^2 + 0.041x51x19.17^2$ kip-ft = 2.424 k-ft

Negative moment at discontinuous edge :

- For A type Slab, $M_{a\,(-ve)} = (1/3) \times 3.383 = 1.128$ kip-ft
- For B type Slab, $M_{b\,(-ve)} = (1/3) \times 2.862 = 0.954$ kip-ft
- For E type Slab, $M_{e\,(-ve)} = (1/3) \times 2.634 = 0.878$ kip-ft
- For F type Slab, $M_{f\,(-ve)} = (1/3) \times 2.424 = 0.808$ kip-ft

(i) **For Longer Direction :**

Negative moment at continuous edge :

- For A type Slab, $M_{a\,(-ve)} = C_{b\text{-}neg.}\, wl_b^2 = 0.029x224.25x24.17^2 = 3.8$ kip-ft

- For B type Slab, $M_{b\,(-ve)} = C_{b.neg.}\, wl_b^2 = 0.041x224.25x24.17^2 = 5.371$ kip-ft
- For E type Slab, $M_{e\,(-ve)} = C_{b.neg.}\, wl_b^2 = 0.017x224.25x24.17^2 = 2.23$ kip-ft
- For F type Slab, $M_{f\,(-ve)} = C_{b.neg.}\, wl_b^2 = 0.027x224.25x24.17^2 = 3.54$ kip-ft

Positive moment at Mid-Span Section :

- For A type Slab, $M_{a(+ve)} = C_a.dl\ wl_a^2 + C_a.ll.\ wl_a^2 = 0.016x173.25x24.17^2 + 0.020x51x24.17^2$ kip-ft = 2.215 kip-ft
- For B type Slab, $M_{b\,(+ve)} = 0.015x173.25x24.17^2 + 0.019x51x24.17^2$ kip-ft = 2.084 k-ft
- For E type Slab, $M_{e\,(+ve)} = 0.010x173.25x24.17^2 + 0.017x51x24.17^2$ kip-ft = 1.5186 k-ft
- For F type Slab, $M_{f\,(+ve)} = 0.011x173.25x24.17^2 + 0.017x51x24.17^2$ kip-ft = 1.6198 k-ft

Negative moment at discontinuous edge :

- For A type Slab, $M_{a\,(-ve)} = (1/3) \times 2.215 = 0.738$ kip-ft
- For B type Slab, $M_{b\,(-ve)} = (1/3) \times 2.084 = 0.695$ kip-ft
- For E type Slab, $M_{e\,(-ve)} = (1/3) \times 1.5186 = 0.5062$ kip-ft
- For F type Slab, $M_{f\,(-ve)} = (1/3) \times 1.6198 = 0.54$ kip-ft

3) Depth Check :

For $f_{c'} = 4000$ psi, and $f_y = 60000$ psi

$\rho = 0.75x0.85x\beta_1x(f_{c'}/ f_y)x(87000/(87000+ f_y)) = 0.0214$

$R = \rho f_y\ (1- 0.59x\ f_y/ f_{c'}) = 0.0214x60000x(1-0.59x0.0214x60000/4000) = 1040.081$ psi

$d = (M_u/\varphi Rb)^{1/2} = (6.181x1000x12/(0.90x1040.081x12)) = 2.57'' << 5.5''$ (ok)

4) Shear Check :

$V_u = 1.15x0.22425x24.17/2-0.22425x4.75/12 = 3.03$ kip

$\varphi V_c = (0.85x2x(4000)^{1/2}\ x12x4.75)/1000 = 6.13$ kip> V_u (ok)

5) Reinforcement Calculation :

(a) Short Direction:

For maximum negative moment at discontinuous edge, $A_s = 0.055$ in^2, use #3 @ 11" c/c.

For maximum positive moment at mid-span, $A_s = 0.163$ in^2, use #3 @ 8" c/c.

For maximum negative moment at continuous edge, $A_s = 0.304$ in^2, use #3 @ 5.75" c/c.

(b) Long Direction :

For maximum negative moment at discontinuous edge, $A_s = 0.035$ in^2, use #3 @ 11" c/c.

For maximum positive moment at mid-span, $A_s = 0.106$ in^2, use #3 @ 11" c/c.

For maximum negative moment at continuous edge, $A_s = 0.262^2$, use #3@ 6.5"c/c.

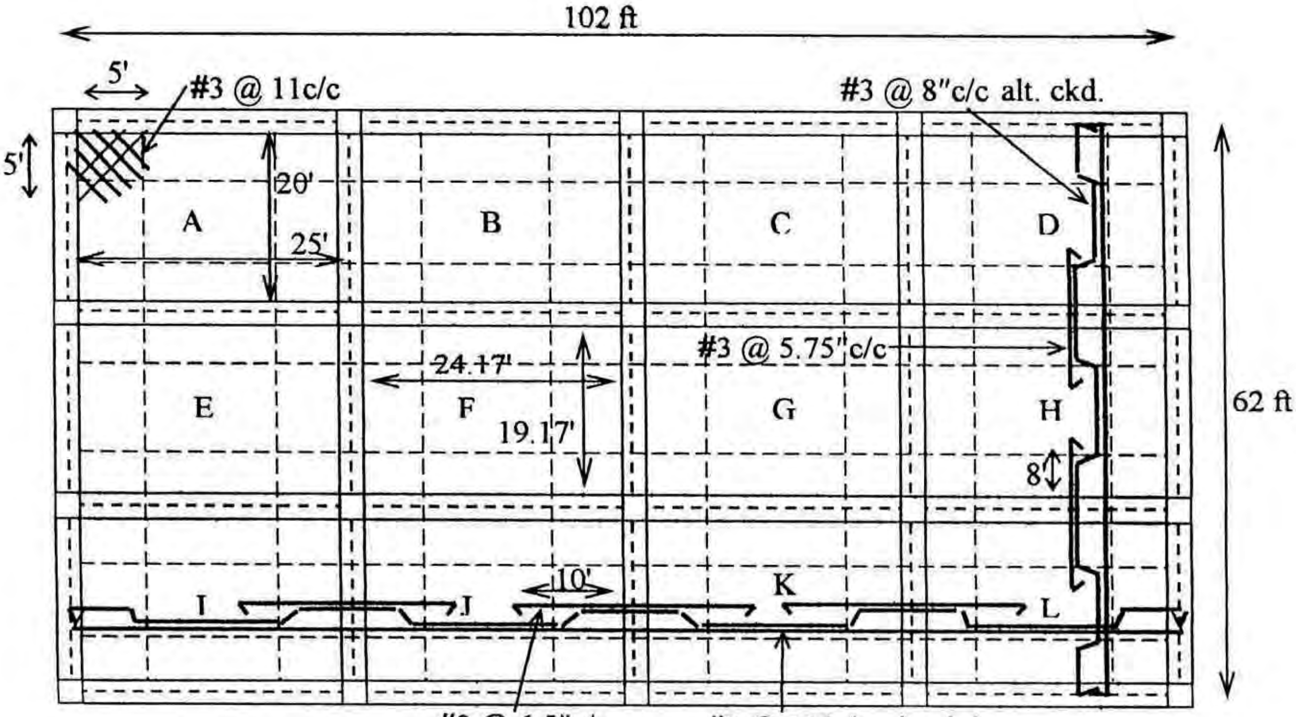


Figure II-13: Detail reinforcements of Roof Slab.

Second Floor Slab :

Assuming thickness of floor slab = 7.5″

1) Load Calculation :

Dead load :

(i)	Self load	= (7.5/12)x150 =	93.75 psf
(ii)	Floor finish	=	12 psf
(iii)	Loads due to partition wall	=	231.45 psf
	Total dead load	=	337.2 psf

Live load : For Lobbies or Platforms (Table 1.1, Design of Concrete Structures- Arthur H. Nilson, Twelfth edition,) = 100 psf

Total factored load = 1.4 x D.L + 1.7 x L.L = 1.4 x 337.2 + 1.7 x 100= 472.08 + 170 = 642.08 psf

2) Moment Calculation :

(i) For Shorter Direction :

Maximum Negative moment at continuous edge :

- For E type Slab, $M_{a\,(-ve)} = C_{a\text{-}neg}\, wl_a^2 = 0.075 \times 642.08 \times 18.75^2 = 16.93$ kip-ft

Maximum Positive moment at Mid-Span Section :

- For A type Slab, $M_{a\,(+ve)} = C_{a,dl}\, wl_a^2 + C_{a,ll}\, wl_a^2 = 0.039 \times 472.08 \times 18.75^2 + 0.048 \times 170 \times 18.75^5$ kip-ft = 9.341 kip-ft

Maximum Negative moment at discontinuous edge :

- For A type Slab, $M_{a\,(-ve)} = (1/3) \times 9.341 = 3.114$ kip-ft

(ii) For Longer Direction :

Maximum Negative moment at continuous edge :

- For B type Slab, $M_{b\,(-ve)} = C_{b\text{-}neg}\, wl_b^2 = 0.041 \times 642.08 \times 23.75^2 = 14.85$ kip-ft

Maximum Positive moment at Mid-Span Section :

- For A type Slab, $M_{a(+ve)} = C_a.dl\ wl_a^2 + C_a.ll.\ wl_a^2 = 0.016x472.08x23.75^2 + 0.020x170x23.75^2$ kip-ft = 6.1783 kip-ft

Maximum Negative moment at discontinuous edge :

- For A type Slab, $M_{a(-ve)}$ = (1/3) x 6.1783 = 2.0594 kip-ft

3) Depth Check :

For $f_{c'}$ = 4000 psi, and f_y = 60000 psi

$\rho = 0.75x0.85x\beta_1x(f_{c'}/ f_y)x(87000/(87000+ f_y)) = 0.0214$

$R = \rho f_y (1- 0.59x\ f_y/ f_{c'}) = 0.0214x60000x(1-0.59x0.0214x60000/4000) = 1040.081$ psi

$d = (M_u/\varphi Rb)^{1/2} = (16.93x1000x12/(0.90x1040.081x12))^{1/2} = 4.25'' << 7.5''$ (ok)

4) Shear Check :

V_u = 1.15x0.64208x23.75/2-0.64208x6.75/12 = 8.4072 kip

$\varphi V_c = (0.85x2x(4000)^{1/2}\ x12x6.75)/1000 = 8.709$ kip> V_u (ok)

5) Reinforcement Calculation :

(a) Short Direction :

For maximum negative moment at discon. edge, A_s = 0.104 in^2, use #3 @ 12.5" c/c.

For maximum positive moment at mid-span, A_s = 0.32 in^2, use #4 @ 7.5" c/c.

For maximum negative moment at continuous edge, A_s = 0.6 in^2, use #4 @ 5.75" c/c.

(a) Long Direction :

For maximum negative moment at discontinuous edge, A_s = 0.069 in^2, use #3 @ 15" c/c.

For maximum positive moment at mid-span, A_s = 0.21 in^2, use #3 @ 6.25" c/c.

For maximum negative moment at continuous edge, A_s = 0.52 in^2, use #4@ 5.75"c/c.

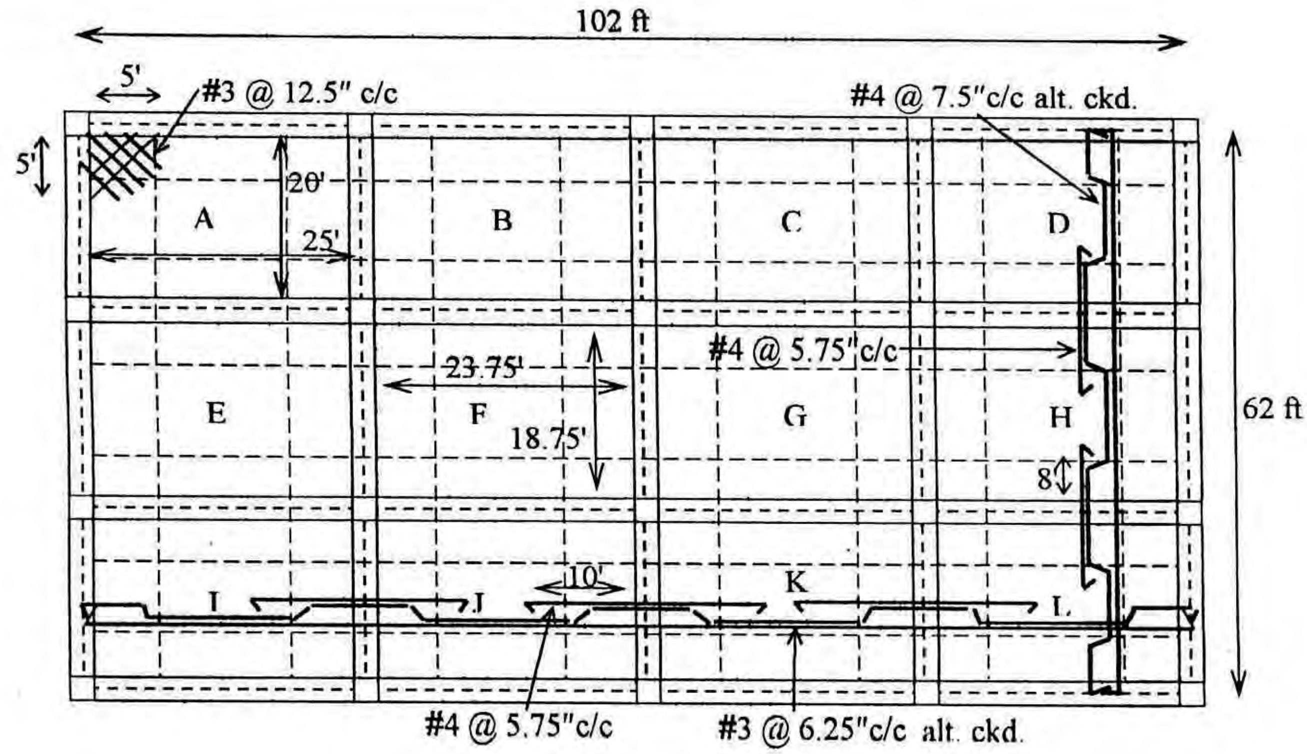


Figure II-14: Detail reinforcement of second Floor Slab.

First Floor Slab :

Assuming thickness of floor slab = 9.5"

1) Load Calculation :

Dead load :

(i)	Self load	= (9.5/12)x150 = 118.75 psf
(ii)	Floor finish	= 12 psf
(iii)	Loads due to partition wall	= 305.77 psf
	Total dead load	=436.52 psf

Live load : For Lobbies or Platforms (Table 1.1, Design of Concrete Structures- Arthur H. Nilson, Twelfth edition,) = 100 psf

Total factored load = 1.4 x D.L + 1.7 x L.L = 1.4 x 436.52 + 1.7 x 100= 611.12 + 170 = 781.12 psf.

2) Moment Calculation :

(i) For Shorter Direction :

Maximum Negative moment at continuous edge :

- For E type Slab, $M_{e\,(-ve)} = C_{a\text{-}neg.}\, wl_a^2 = 0.075\text{x}781.12\text{x}18.5^2 = 20.05$ kip-ft

Maximum Positive moment at Mid-Span Section :

- For A type Slab, $M_{a\,(+ve)} = C_a.dl\ wl_a^2 + C_a.ll.\ wl_a^2 = 0.039\text{x}611.12\ \text{x}18.5^2 + 0.048\text{x}170\text{x}18.5^2$ kip-ft = 10.95 kip-ft

Maximum Negative moment at discontinuous edge :

- For A type Slab, $M_{a\,(-ve)} = (1/3) \times 10.95 = 3.65$ kip-ft

(ii) For Longer Direction :

Maximum Negative moment at continuous edge :

- For B type Slab, $M_{b\,(-ve)} = C_{b\text{-}neg.}\, wl_b^2 = 0.041\text{x}\ 781.12\ \text{x}23.5^2 = 17.69$ kip-ft

Maximum Positive moment at Mid-Span Section :

- For A type Slab, $M_{a(+ve)} = C_a.dl\ wl_a^2 + C_a.ll.\ wl_a^2 = 0.016\text{x}611.12\text{x}23.5^2 + 0.020\text{x}170\text{x}23.5^2$ kip-ft = 7.28 kip-ft

Maximum Negative moment at discontinuous edge :

- For A type Slab, $M_{a\,(-ve)} = (1/3) \times 7.28 = 2.426$ kip-ft

3) Depth Check :

For $f_{c'} = 4000$ psi, and $f_y = 60000$ psi

$\rho = 0.75\text{x}0.85\text{x}\beta_1\text{x}(f_{c'}/ f_y)\text{x}(87000/(87000+ f_y)) = 0.0214$

$R = \rho f_y\ (1- 0.59\text{x}\ f_y/ f_{c'}) = 0.0214\text{x}60000\text{x}(1\text{-}0.59\text{x}0.0214\text{x}60000/4000) = 1040.081$ psi

$d = (M_u/\varphi Rb)^{1/2} = (20.05\text{x}1000\text{x}12/(0.90\text{x}1040.081\text{x}12))^{1/2} = 4.63'' << 9.5''$ (ok)

4) Shear Check :

$V_u = 1.15\text{x}0.78112\text{x}23.5/2\text{-}0.78112\text{x}8.75/12 = 9.985$ kip

$\varphi V_c = (0.85\text{x}2\text{x}(4000)^{1/2}\ \text{x}12\text{x}8.75)/1000 = 11.29$ kip> V_u (ok)

5) Reinforcement Calculation :

(a) Short Direction :

For maximum negative moment at discon. edge, $A_s = 0.094\ in^2$, use #3 @ 14" c/c.

For maximum positive moment at mid-span, $A_s = 0.29\ in^2$, use #4 @ 8.25" c/c.

For maximum negative moment at continuous edge, $A_s = 0.533\ in^2$, use #4 @ 6.5" c/c.

(a) Long Direction :

For maximum negative moment at discontinuous edge, $A_s = 0.063\ in^2$, use #3 @ 19" c/c.

For maximum positive moment at mid-span, $A_s = 0.188\ in^2$, use #3 @ 7" c/c.

For maximum negative moment at continuous edge, $A_s = 0.52\ in^2$, use #4@ 6.25"c/c.

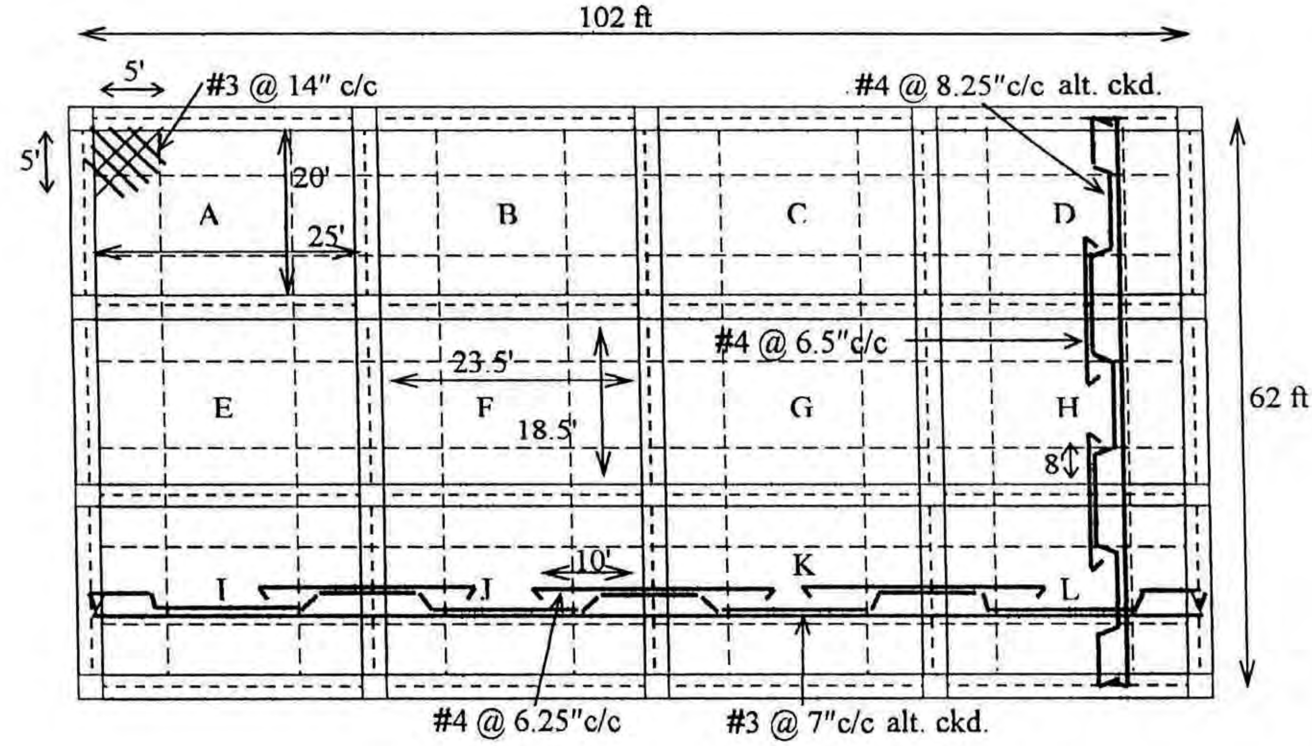


Figure II-15: Detail reinforcement of first Floor Slab.

__Vertical load Analysis for Roof Beam :__

For Transverse Interior (TI) Section :

Dead Load :

wt. of Slab = (5.5/12)x25x.0.15	= 1.72	k/ft
w t. of L.C. = (4/12)x25x.0.15	= 1.25	k/ft
wt. of Plaster = (0.5/12)x25x.0.15	= 0.15625	k/ft
Stem of Beam = 12x18.5x.(0.15/144)	= 0.23125	k/ft
Total Dead Load for Roof Beam	= 3.3575	k/ft

Factored Dead Load for Roof Beam = 1.4x3.3575 = 4.7005 k/ft

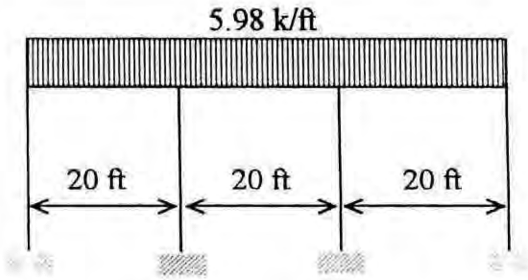
Factored Live Load for Roof Beam = 1.7x30x25 = 1.275 k/ft

Total Factored Load = 5.98 k/ft

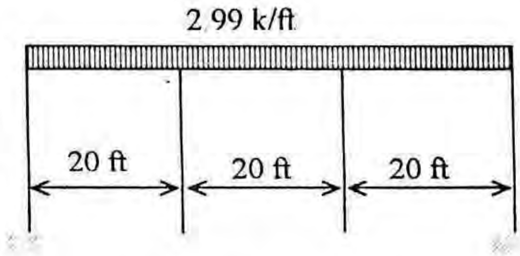
For Transverse Exterior (TE) Section : Total Factored Load = 2.99 k/ft

For Longitudinal Interior (LI) Section : Total Factored Load = 4.844 k/ft

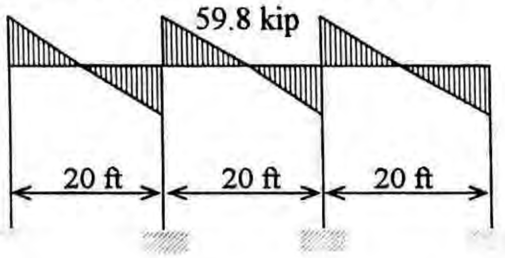
For Longitudinal Exterior (LE) Section : Total Factored Load = 2.422 k/ft



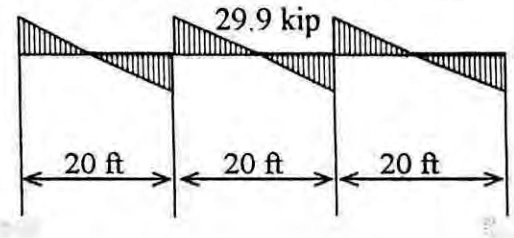
Ver. Load distribution dia.(TI)



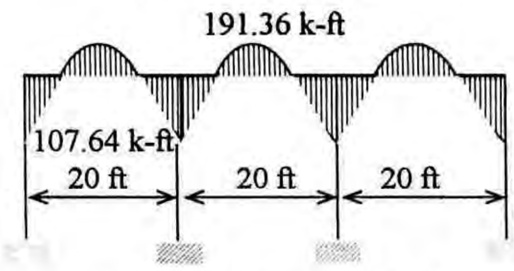
Ver.Load distribution dia.(TE)



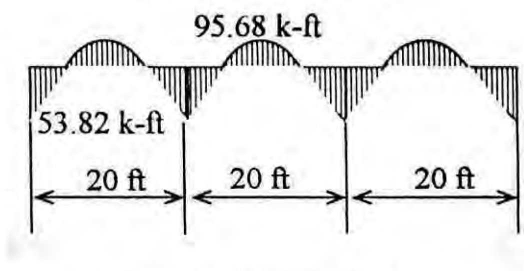
Shear Force dia.(TI)



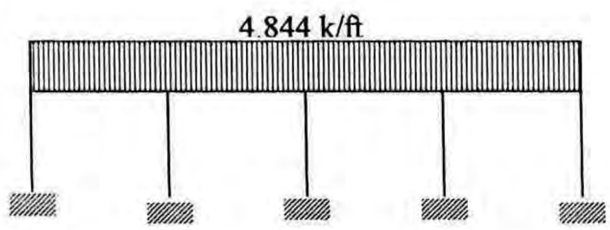
Shear Force dia.(TE)



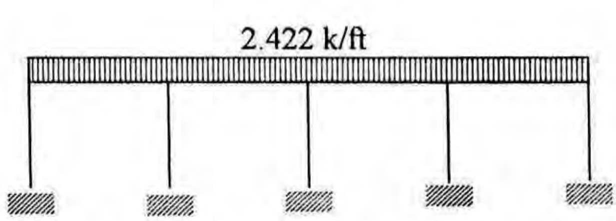
Bending Moment dia.(TI)



Bending Moment dia.(TE)



Ver. Load distribution dia.(LI)



Ver.Load distribution dia.(LE)

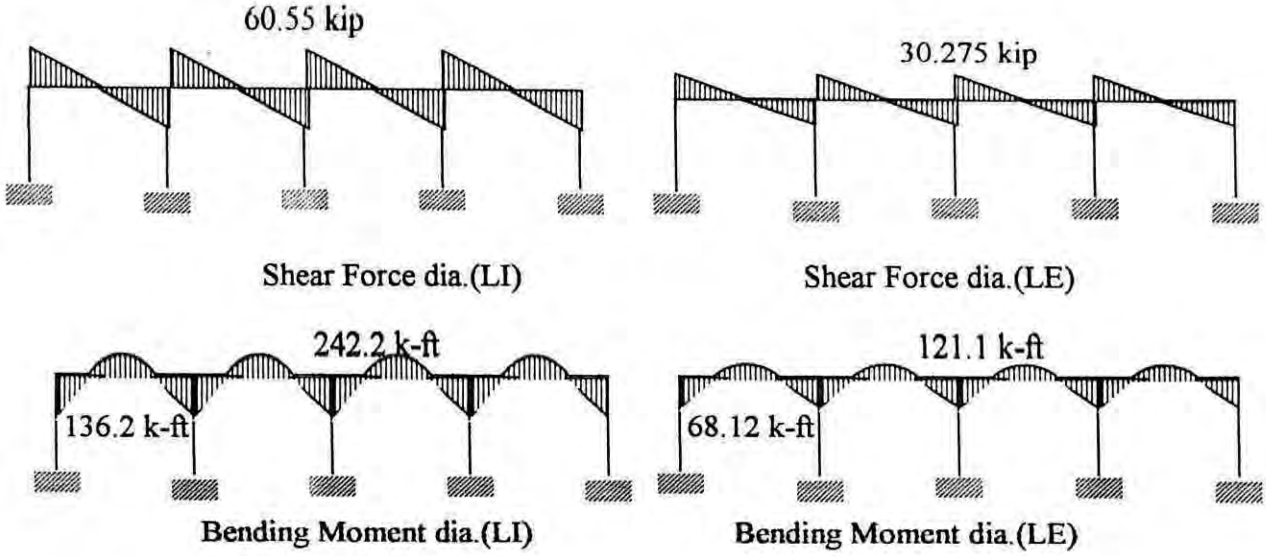


Figure II-16: Ver. Load, Shear force and Bending moment dia. of first Floor Slab.

Chart for Design Moment and Shear (Roof) :

Frame	Design PositiveMoment	Design Negative Moment	Design Shear
T.I	191.36 kip-ft	107.64 kip-ft	59.8 kip
T.E	95.68 kip-ft	53.82 kip-ft	29.9 kip
L.I	242.20 kip-ft	136.24 kip-ft	60.55 kip
L.E	121.10 kip-ft	68.12 kip-ft	30.275 kip

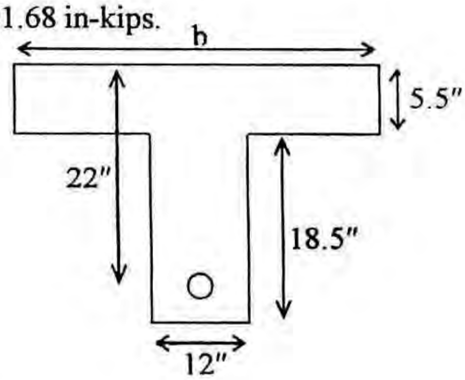
Design of Roof Beam :

For :Longitudinal Interior (LI) frame :

Design PositiveMoment = 191.36 k-ft = 2296.32 in-kips.

Design Negative Moment = 107.64 k-ft = 1291.68 in-kips.

Section of beam = 12"x24"



For f_c' = 4000 psi and f_y = 60000 psi

For positive moment :

d = 24" - 2" = 22"

Effective flange, $16h_f+b_w$ = 16x5.5+12 = 100 inch

Span/4 = (25x12/4) = 75"

Centre line beam spacing = (25x25/2) = 25 ft, i.e, b =75"

Assuming the stress block depth equal to flange thickness of 5.5"

d-a/2 = 22- 5.5/2 = 19.25"; $A_s = (M_u/(\varphi f_y (d-a/2)))$ = (242.2x12/(0.9x60x19.25)) = 2.8 in^2

Now, $\rho = A_s/bd$ = (2.8/75x22) = 0.0017

i.e, $a = \rho f_y d/(0.85x\ f_c')$ = (0.0017x60x22/(0.85x4)) = 0.66 inch.

Since, a is less than h_f, a rectangular analysis is required. For singly reinforced beam,

d = 22", A_s = 0.0214x22x12 =5.6496 in^2

a = (5.6496x60/(0.85x4x12)) = 8.31 in

A_s = (2906.4/(0.9x60x(22-8.31/2))) = 3.02 in^2

By trial & error, at final stage, A_s = 2.688 in^2, Use 4 Nos #8

Depth Check :

d^2 = (242.2x12)/(0.9x0.0214x60x12x(1- 0.59x0.0017x60/4)) = 212.87

⇨ d = 14.59" <22" (ok).

Development Length : For #8 No. bar : $L_d = (0.04f_yd_b/(f_c')^{1/2})$ = 37.95" ≈ 38"

Shear Check at Support : At d depth, Design shear, V_{ud} = (60.55x10.67/12.5) = 51.69 kips. Allowable shear capacity, φV_c = (0.85x2x(4000)$^{1/2}$x 12x22 = 28.384 kip < 51.69 kips. So, Stirrup is required.

Use #3 No. bar, S = (0.85x0.22x60000x22/((51.69 -28.384)x1000))) = 10.6" ≈ 10.5"c/c

S_{max} = (0.4x60000/(50x12)) = 40"

S_{max} = (22/2) = 11"

S_{max} = 24"

Average Spacing, (10.5"+11")/2 =10.75" c/c.

For negative moment :

Negative moment = 136.2375 kip-ft = 1634.85 inch-kips

A_s = 0.0214x22x12 = 5.65 in^2

a = (5.65x60/(0.85x4x12)) = 8.31 inch.

A_s = (1634.85/(0.9x60x(22-8.31/2))) = 1.69 in^2

By trial & error, at final stage, A_s = 1.45 in^2, Use 2 Nos #8

For Longitudinal Exterior (LE) frame :

For positive moment :

Positive moment = 121.1 kip-ft = 1453.2 inch-kips

A_s = 0.0214x22x12 = 5.65 in^2

a = (5.65x60/(0.85x4x12)) = 8.31 inch.

A_s = (1453.2/(0.9x60x(22-8.31/2))) = 1.51 in^2

By trial & error, at final stage, A_s = 1.28 in^2, Use 2 Nos #8

For negative moment :

Negative moment = 68.12 kip-ft = 817.44 inch-kips

A_s = 0.0214x22x12 = 5.65 in^2

a = (5.65x60/(0.85x4x12)) = 8.31 inch.

A_s = (817.44/(0.9x60x(22-8.31/2))) = 0.848 in^2

By trial & error, at final stage, A_s = 0.705 in^2, Use 3 No #5.

For Transverse Exterior (TE) frame :

For positive moment :

Positive moment = 95.65 kip-ft = 1148.16 inch-kips

A_s = 0.0214x22x12 = 5.65 in^2

a = (5.65x60/(0.85x4x12)) = 8.31 inch.

A_s = (1148.16/(0.9x60x(22-8.31/2))) = 1.19 in^2

By trial & error, at final stage, A_s = 1.0 in^2, Use 4 Nos #5

For negative moment :

Negative moment = 53.82 kip-ft = 645.84 inch-kips

A_s = 0.0214x22x12 = 5.65 in^2

a = (5.65x60/(0.85x4x12)) = 8.31 inch.

A_s = (645.84/(0.9x60x(22-8.31/2))) = 0.67 in^2

By trial & error, at final stage, A_s = 0.554 in^2, Use 2 Nos #5.

For Transverse Interior (TI) frame :

For positive moment :

Positive moment = 191.36 kip-ft = 2296.32 inch-kips

A_s = 0.0214x22x12 = 5.65 in^2

a = (5.65x60/(0.85x4x12)) = 8.31 inch.

A_s = (2296.32/(0.9x60x(22-8.31/2))) = 2.383 in^2

By trial & error, at final stage, A_s = 2.08 in^2, Use 3 Nos #8.

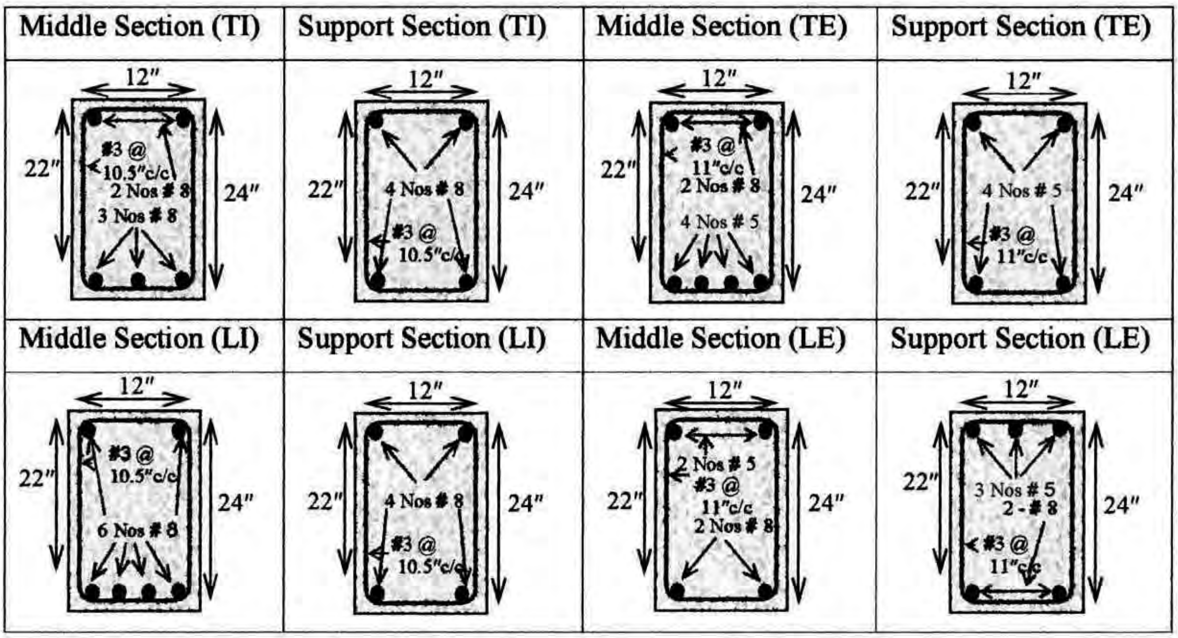


Figure II-17: Transverse Cross Section of Roof Beam.

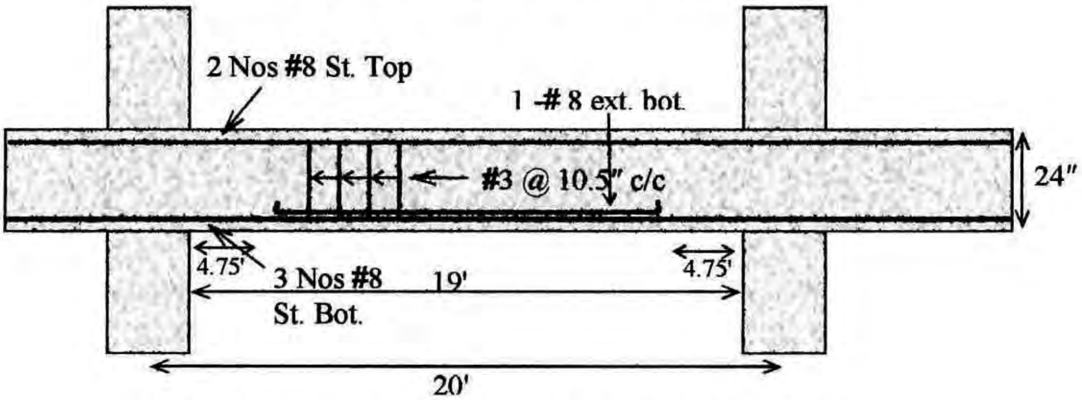


Figure II-18: Longitudinal Cross Section of (TI) Roof Beam.

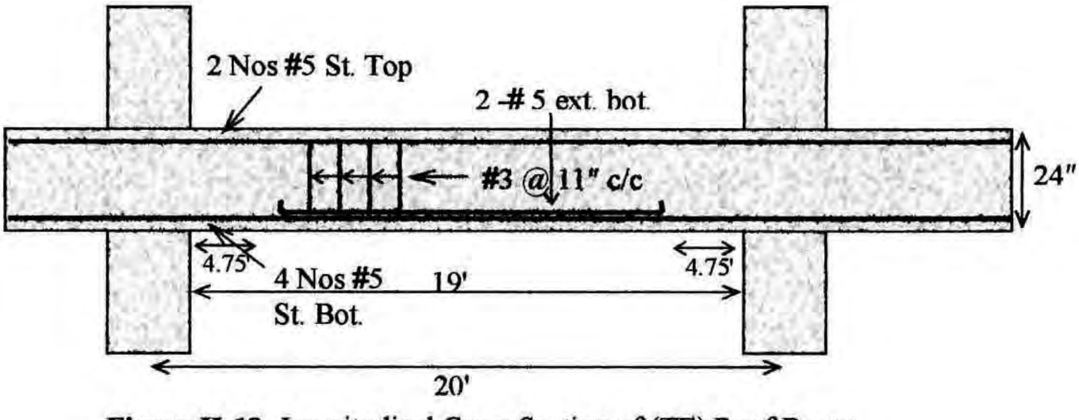


Figure II-19: Longitudinal Cross Section of (TE) Roof Beam.

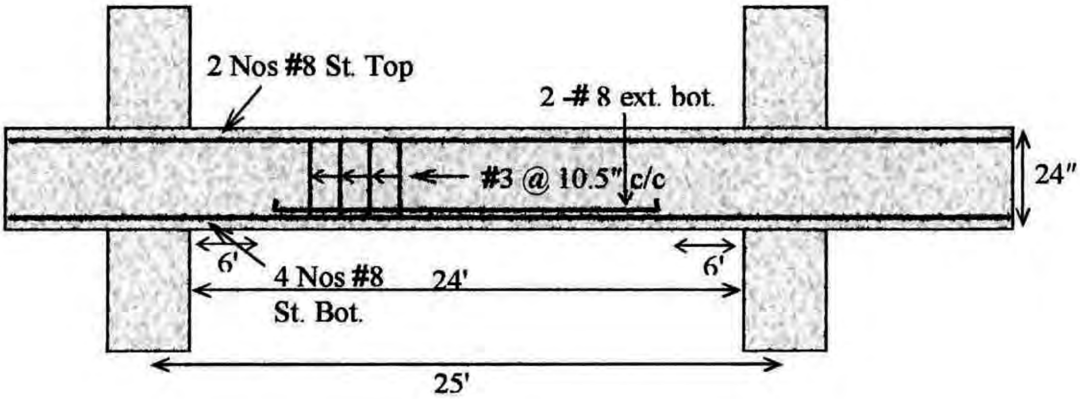


Figure II-20: Longitudinal Cross Section of (LI) Roof Beam.

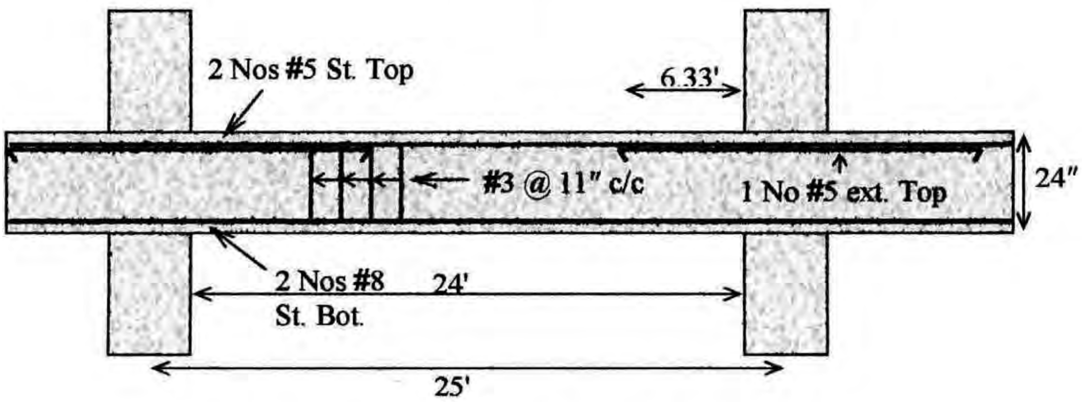


Figure II-21: Longitudinal Cross Section of (LE) Roof Beam.

Vertical load Analysis for Second Floor Beam :

For Transverse Interior (TI) Section :

Dead Load :

wt. of Slab	= (7.5/12)x25x.0.15	= 2.344 k/ft
w t. of Floor Finish	= (1/12)x25x.0.15	= 0.3125 k/ft
wt. of Plaster	= (0.5/12)x25x.0.15	= 0.15625 k/ft
Stem of Beam	= 15x19.5x.(0.15/144)	= 0.305 k/ft
Total Dead Load for 2^{nd} Floor Beam		= 3.12 k/ft

Factored Dead Load for 2^{nd} Floor Beam = 1.4x3.12 = 4.368 k/ft

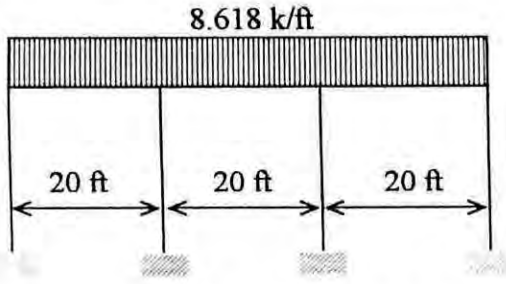
Factored Live Load for 2^{nd} Floor Beam = 1.7x100x25 = 4.25 k/ft

Total Factored Load = 8.618 k/ft

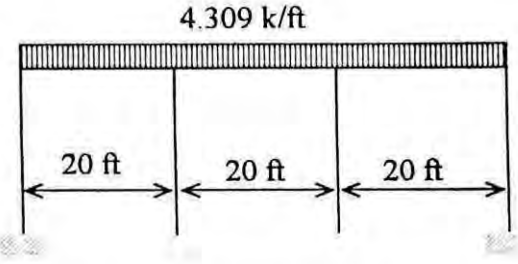
For Transverse Exterior (TE) Section : Total Factored Load = 4.309 k/ft

For Longitudinal Interior (LI) Section : Total Factored Load = 6.977 k/ft

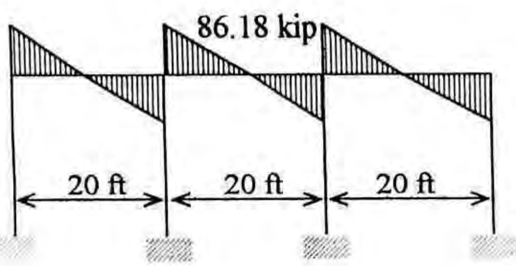
For Longitudinal Exterior (LE) Section : Total Factored Load = 3.4885 k/ft



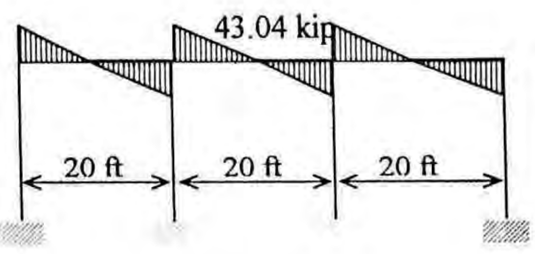
Ver. Load distribution dia.(TI)



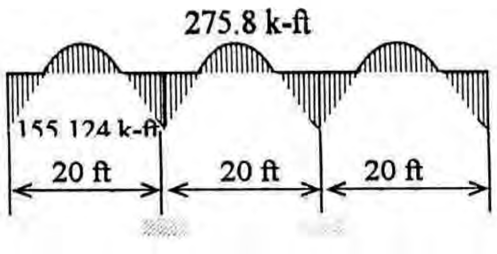
Ver.Load distribution dia.(TE)



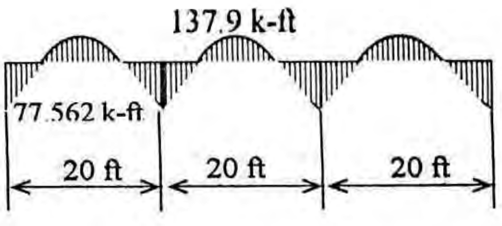
Shear Force dia.(TI)



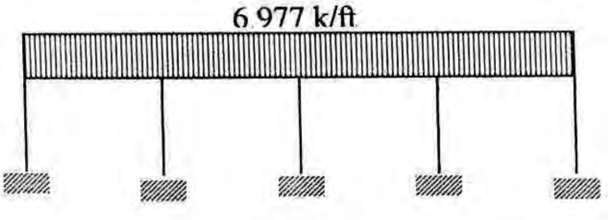
Shear Force dia.(TE)



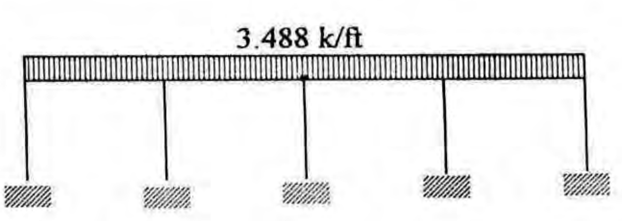
Bending Moment dia.(TI)



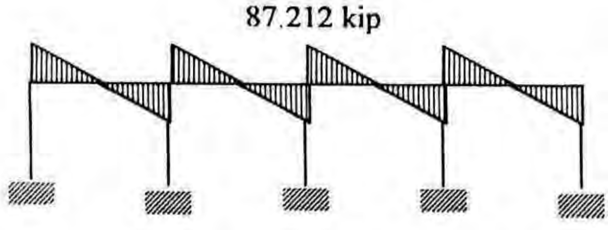
Bending Moment dia.(TE)

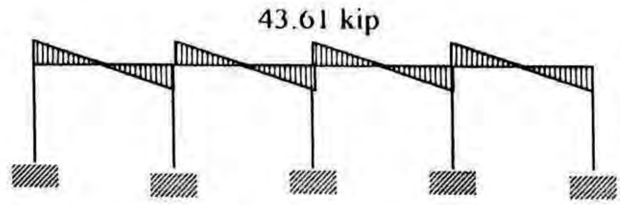


Ver. Load distribution dia.(LI)



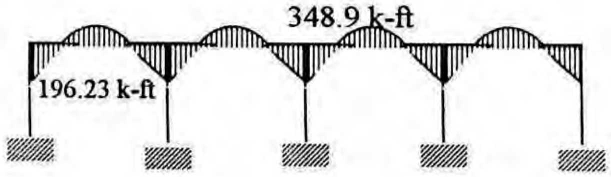
Ver.Load distribution dia.(LE)

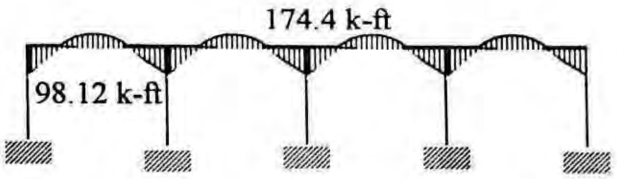




Shear Force dia.(LI)

Shear Force dia.(LE)





Bending Moment dia.(LI)

Bending Moment dia.(LE)

Figure II-22: Ver. load, Shear force and Bending moment dia. of second floor Beam.

Chart for Design Moment and Shear (2nd Floor) :

Frame	Design PositiveMoment	Design Negative Moment	Design Shear
T.I	275.8 kip-ft	155.124 kip-ft	86.18 kip
T.E	137.9 kip-ft	77.562 kip-ft	43.04 kip
L.I	348.9 kip-ft	196.23 kip-ft	87.2125 kip
L.E	174.4 kip-ft	98.12 kip-ft	43.61 kip

Design of Second Floor Beam :

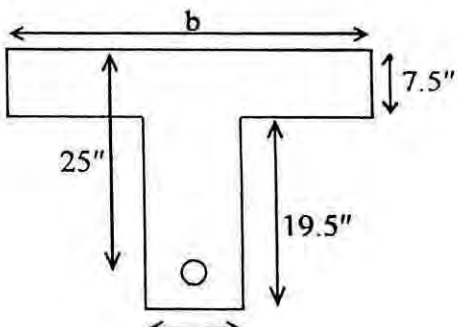
For :Longitudinal Interior (LI) frame :

Design PositiveMoment = 348.9 k-ft = 4186.8 in-kips.

Design Negative Moment = 196.23 k-ft = 2354.76 in-kips.

Section of beam = 15"x27"

For f_c' = 4000 psi and f_y = 60000 psi



For positive moment :

d = 27" - 2" = 25"

Effective flange, $16h_f + b_w$ = 16x7.5+15 = 135 inch

Span/4 = (25x12/4) = 75"

Centre line beam spacing = (25x25/2) = 25 ft, i.e, b =75"

Assuming the stress block depth equal to flange thickness of 7.5″

d-a/2 = 25- 7.5/2 = 21.25″; $A_s = (M_u/(\varphi f_y (d\text{-}a/2)) = (348.9x12/(0.9x60x21.25)) = 3.65$ in^2

Now, $\rho = A_s$ /bd = (3.65/75x25) = 0.00194

i.e, $a = \rho f_y d/(0.85x\ f_c')$ = (0.00194x60x25/(0.85x4)) = 0.86 inch.

Since, a is less than h_f, a rectangular analysis is required. For singly reinforced beam,

d = 27″-4″= 23″, A_s = 0.0214x23x15 =7.383 in^2

a = (7.783x60/(0.85x4x15)) = 8.69 in

A_s = (4186.8/(0.9x60x(23-8.69/2))) = 4.16 in^2

By trial & error, at final stage, A_s = 3.73 in^2, Use 4 Nos #8 and 2 Nos #5

Depth Check :

d^2 = (348.9x12)/(0.9x0.0214x60x15x(1- 0.59x0.0108x60/4)) = 267.06 in^2

=> d = 16.34″ <23″ (ok).

Development Length : For #8 No. bar : $L_d = (0.04 f_y d_b/(\ f_c')^{1/2})$ = 37.95″ ≈ 38″

Shear Check at Support : At d depth, Design shear, $V_{u(d)}$ = (87.2125x7.292/9.375) = 67.835 kips. Allowable shear capacity, $\varphi V_c = (0.85x2x(4000)^{1/2}x\ 15x25)/1000$ = 40.32 kip < 67.835 kips. So, Stirrup is required.

Use #3 No. bar, S = (0.85x0.22x60000x25/((67.835-40.32)x1000))) = 10.19″ ≈ 10″c/c

S_{max} = (0.4x60000/(50x15)) = 32″ S_{max} = (25/2) = 12.5″ S_{max} = 24″

Average Spacing, (10″+12.5″)/2 = 11.25″ c/c.

For negative moment :

Negative moment = 196.23 kip-ft = 2354.76 inch-kips

A_s = 0.0214x25x15 = 8.025 in^2

a = (8.025x60/(0.85x4x15)) = 9.441 inch.

A_s = (2354.76/(0.9x60x(25-9.441/2))) = 2.15 in^2

By trial & error, at final stage, A_s = 1.822 in^2, Use 3 Nos #8

For Longitudinal Exterior (LE) frame :

For positive moment :

Positive moment = 174.4 kip-ft = 2092.8 inch-kips

$A_s = 0.0214 x 25 x 15 = 8.025$ in^2

a = (8.025x60/(0.85x4x15)) = 9.441 inch.

A_s = (2092.8/(0.9x60x(25-9.441/2))) = 1.911 in^2

By trial & error, at final stage, A_s = 1.6113 in^2, Use 2 Nos #8 and 1 No #5

For negative moment :

Negative moment = 98.12 kip-ft = 1177.44 inch-kips

A_s = 0.0214x25x15 = 8.025 in^2

a = (8.025x60/(0.85x4x15)) = 9.441 inch.

A_s = (1177.44/(0.9x60x(25-9.441/2))) = 1.075 in^2

By trial & error, at final stage, A_s = 0.891 in^2, Use 2 Nos #8

For Transverse Exterior (TE) frame :

For positive moment :

Positive moment = 137.9 kip-ft = 1654.8 inch-kips

A_s = 0.0214x25x15 = 8.025 in^2

a = (8.025x60/(0.85x4x15)) = 9.441 inch.

A_s = (1654.8/(0.9x60x(25-9.441/2))) = 1.551 in^2

By trial & error, at final stage, A_s = 1.263 in^2, Use 2 Nos #8

For negative moment :

Negative moment = 77.562 kip-ft = 930.744 inch-kips

A_s = 0.0214x25x15 = 8.025 in^2

a = (8.025x60/(0.85x4x15)) = 9.441 inch.

A_s = (930.744/(0.9x60x(25-9.441/2))) = 0.85 in^2

By trial & error, at final stage, A_s = 0.701 in^2, Use 3 Nos #5

For Transverse Interior (TI) frame :

For positive moment :

Positive moment = 275.8 kip-ft = 3309.6 inch-kips

A_s = 0.0214x25x15 = 8.025 in^2

a = (8.025x60/(0.85x4x15)) = 9.441 inch.

A_s = (3309.6/(0.9x60x(25-9.441/2))) = 3.022 in^2

By trial & error, at final stage, A_s = 2.612 in^2, Use 4 Nos #8

For negative moment :

Negative moment = 155.124 kip-ft = 1861.5 inch-kips

A_s = 0.0214x25x15 = 8.025 in^2

a = (8.025x60/(0.85x4x15)) = 9.441 inch.

A_s = (1861.5/(0.9x60x(25-9.441/2))) = 1.7 in^2

By trial & error, at final stage, A_s = 1.427 in^2, Use 2 Nos #8

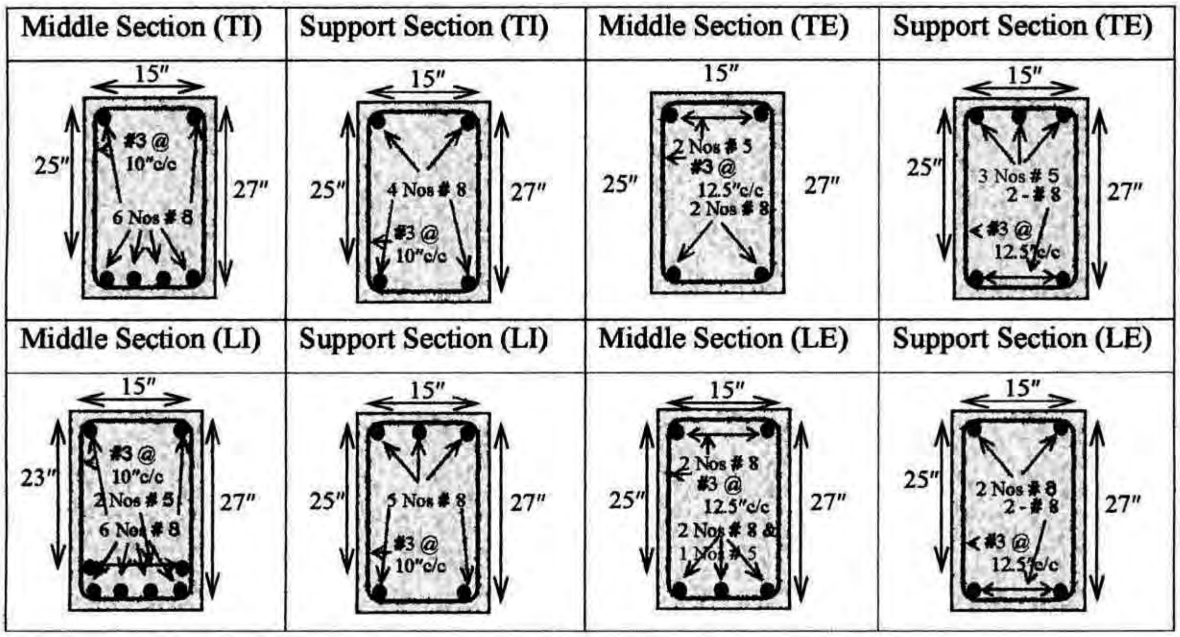


Figure II-23: Transverse Cross Section of 2nd Floor Beam.

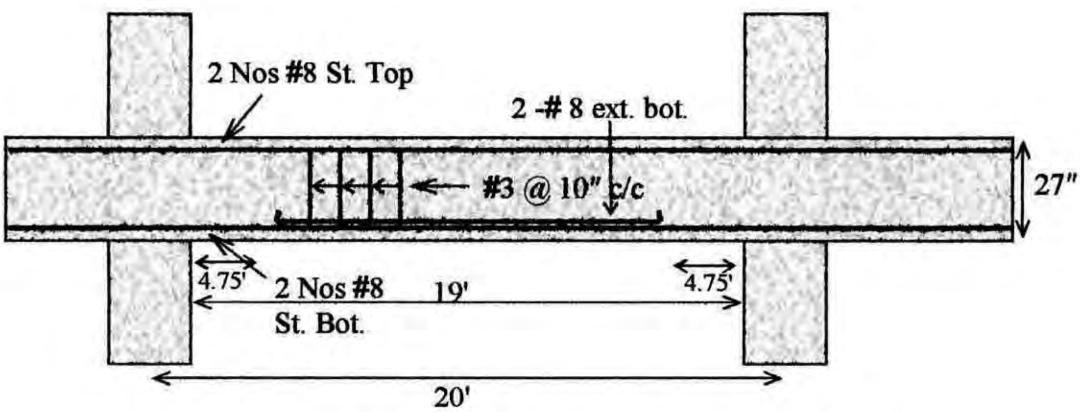


Figure II-24: Longitudinal Cross Section of (TI) 2nd Floor Beam.

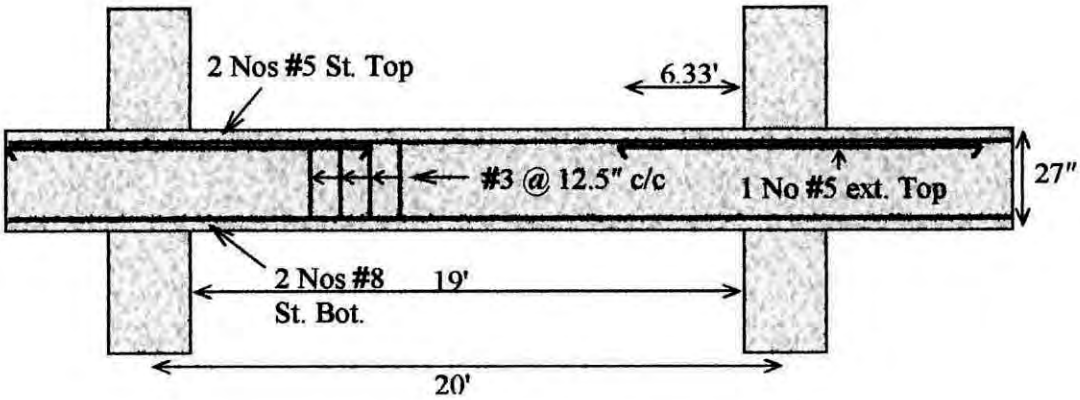


Figure II-25: Longitudinal Cross Section of (TE) 2nd Floor Beam.

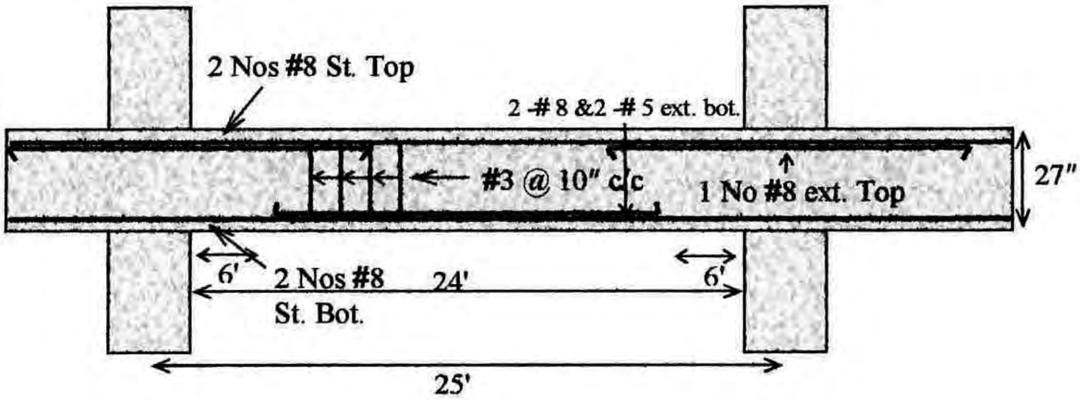


Figure II-26: Longitudinal Cross Section of (LI) 2nd Floor Beam.

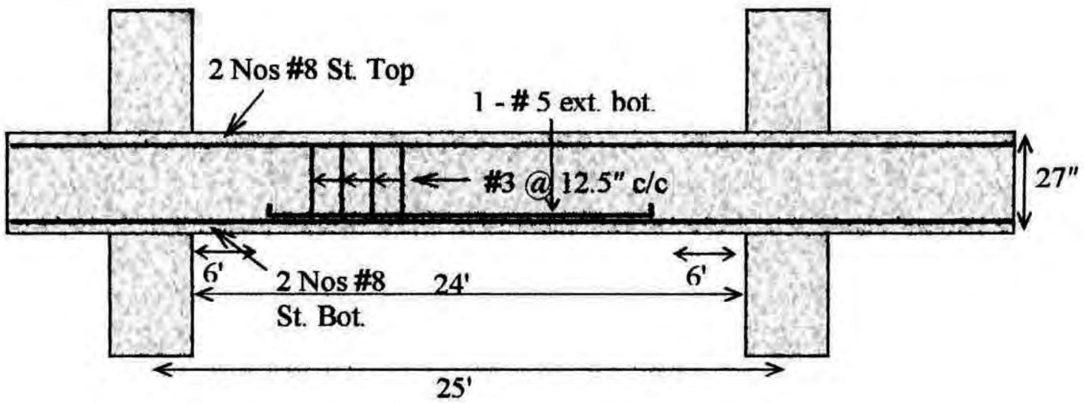


Figure II-27: Longitudinal Cross Section of (LE) 2nd Floor Beam.

Vertical load Analysis for 1st Floor Beam :

For Transverse Interior (TI) Section :

Dead Load :

wt. of Slab	= (9.5/12)x25x.0.15	= 2.97	k/ft
w t. of Floor Finish	= (1/12)x25x.0.15	= 0.3125	k/ft
wt. of Plaster	= (0.5/12)x25x.0.15	= 0.15625	k/ft
Stem of Beam	= 18x20.5x.(0.15/144)	= 0.3844	k/ft

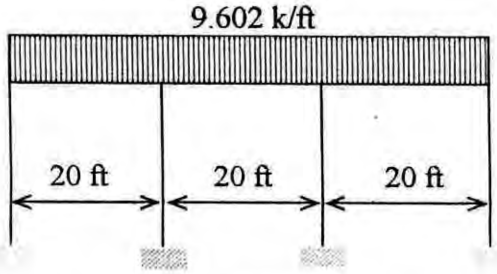
Total Dead Load for 2nd Floor Beam = 3.823 k/ft

Total Factored Load for 1st Floor Beam = 9.6022 k/ft

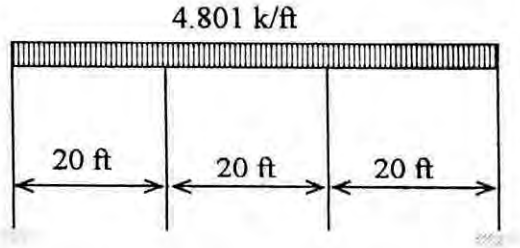
For Transverse Exterior (TE) Section : Total Factored Load = 4.8011 k/ft

For Longitudinal Interior (LI) Section : Total Factored Load = 7.7881 k/ft

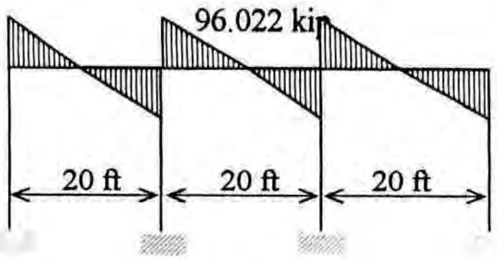
For Longitudinal Exterior (LE) Section : Total Factored Load = 3.8941 k/ft



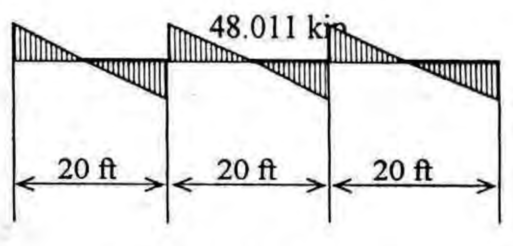
Ver. Load distribution dia.(TI)



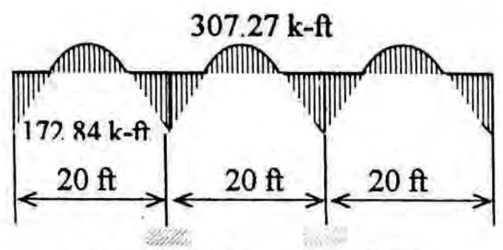
Ver.Load distribution dia.(TE)



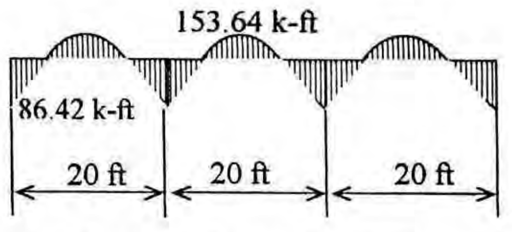
Shear Force dia.(TI)



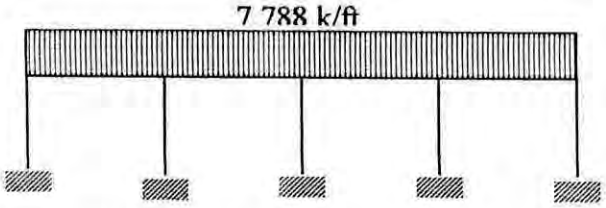
Shear Force dia.(TE)



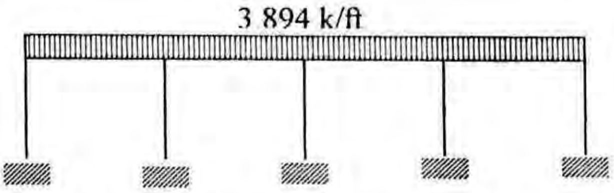
Bending Moment dia.(TI)



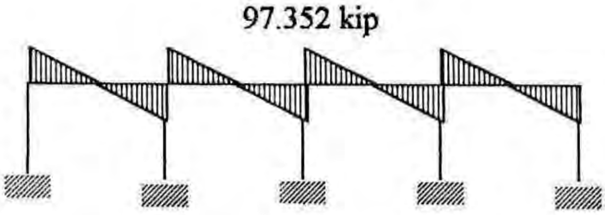
Bending Moment dia.(TE)



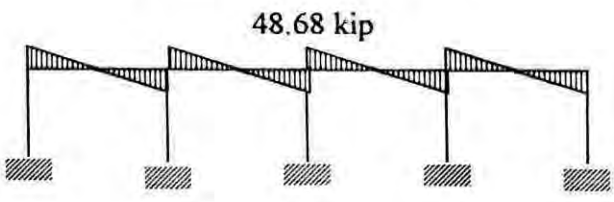
Ver. Load distribution dia.(LI)



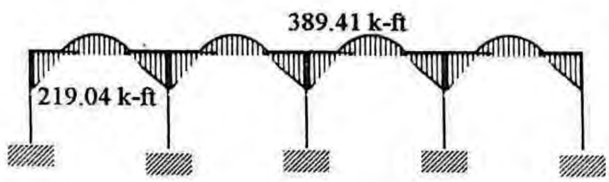
Ver.Load distribution dia.(LE)



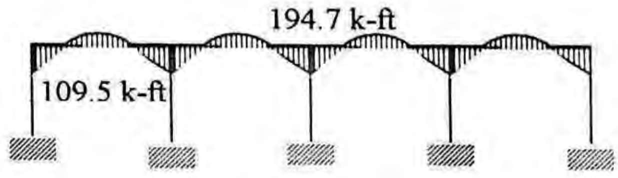
Shear Force dia.(LI)



Shear Force dia.(LE)



Bending Moment dia.(LI)



Bending Moment dia.(LE)

Figure II-28: Ver. load, Shear force and Bending moment dia. of first floor Beam.

Table for Design Moment and Shear (1st Floor) :

Frame	Design PositiveMoment	Design Negative Moment	Design Shear
T.I	307.27 kip-ft	172.84 kip-ft	96.022 kip
T.E	153.64 kip-ft	86.42 kip-ft	48.011 kip
L.I	389.408 kip-ft	219.042 kip-ft	97.352 kip
L.E	194.704 kip-ft	109.52 kip-ft	48.68 kip

Design of 1st Floor Beam :

For :Longitudinal Interior (LI) frame :

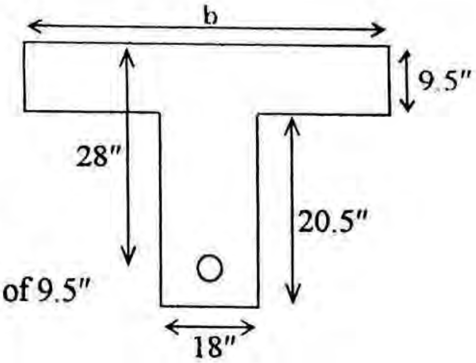
Design PositiveMoment = 389.408 k-ft = 4672.9 in-kips.

Design Negative Moment = 219.402 k-ft = 2628.504 in-kips.

Section of beam = 15″x27″

For f_c' = 4000 psi and f_y = 60000 psi

For positive moment :



$d = 30'' - 2'' = 28''$

Effective flange, $16h_f + b_w = 16x9.5+18 = 170$ inch

Span/4 = (25x12/4) = 75"

Centre line beam spacing = (25x25/2) = 25 ft, i.e, b =75"

Assuming the stress block depth equal to flange thickness of 9.5"

$d\text{-}a/2 = 21.25''$; $A_s = (M_u/(\varphi f_y\,(d\text{-}a/2))$

$= (389.408x12/(0.9x60x21.25)) = 4.072\ in^2$

Now, $\rho = A_s/bd = (4.072/75x28) = 0.00194$

i.e, $a = \rho f_y d/(0.85x\ f_c') = (0.00194x60x28/(0.85x4)) = 0.958$ inch.

Since, a is less than h_f, a rectangular analysis is required. For singly reinforced beam,

$d = 30''\text{-}4'' = 26''$, $A_s = 0.0214x26x18 = 10.0152\ in^2$

$a = (10.0152x60/(0.85x4x18)) = 9.82$ in

$A_s = (4672.9/(0.9x60x(26\text{-}9.82/2))) = 4.103\ in^2$

By trial & error, at final stage, $A_s = 3.57\ in^2$, Use 4 Nos #8 and 2 Nos #5

Depth Check :

$d^2 = (389.408x12)/(0.9x0.0214x60x18x(1\text{-}\ 0.59x0.00762x60/4)) = 216.806\ in^2$

⇨ d = 14.724" <26" (ok).

Development Length : For #8 No. bar : $L_d = (0.04 f_y d_b/(\ f_c')^{1/2}) = 37.95'' \approx 38''$

Shear Check at Support : At d depth, Design shear, $V_{u(d)} = (97.352x(11.75\text{-}2.17)/11.75) = 79.373$ kips. Allowable shear capacity, $\varphi V_c = (0.85x2x(4000)^{1/2}x\ 18x26)/1000 = 50.32$ kip < 79.373 kips. So, Stirrup is required.

Use #3 No. bar, S = (0.85x0.22x60000x25/((79.373 -50.32)x1000))) = 9.655" ≈ 9.5"c/c

$S_{max} = (0.4x60000/(50x18)) = 26.67''$ $S_{max} = (28/2) = 14''$ $S_{max} = 24''$

Average Spacing, (9.5"+14")/2 = 11.75" c/c.

For negative moment :

Negative moment = 219.042 kip-ft = 2628.504 inch-kips

$A_s = 0.0214x28x18 = 10.7856\ in^2$

$a = (10.7856x60/(0.85x4x18)) = 10.574$ inch.

$A_s = (2628.504\ /(0.9x60x(28\text{-}10.574/2))) = 2.143\ in^2$

By trial & error, at final stage, $A_s = 1.795\ in^2$, Use 2 Nos #8 & 1 No. #5

For Longitudinal Exterior (LE) frame :

For positive moment :

Positive moment = 194.704 kip-ft = 2336.45 inch-kips

$A_s = 0.0214x28x18 = 10.7856\ in^2$

$a = (10.7856x60/(0.85x4x18)) = 10.574$ inch.

$A_s = (2336.45\ /(0.9x60x(28-10.574/2))) = 1.905\ in^2$

By trial & error, at final stage, $A_s = 1.59\ in^2$, Use 2 Nos #8 & 1 No. #5

For negative moment :

Negative moment = 109.521 kip-ft = 1314.252 inch-kips

$A_s = 0.0214x28x18 = 10.7856\ in^2$

$a = (10.7856x60/(0.85x4x18)) = 10.574$ inch.

$A_s = (1314.252\ /(0.9x60x(28-10.574/2))) = 1.0715\ in^2$

By trial & error, at final stage, $A_s = 0.883\ in^2$, Use 3 Nos #5

For Transverse Exterior (TE) frame :

For positive moment :

Positive moment = 153.64 kip-ft = 1843.68 inch-kips

$A_s = 0.0214x28x18 = 10.7856\ in^2$

$a = (10.7856x60/(0.85x4x18)) = 10.574$ inch.

$A_s = (1843.68\ /(0.9x60x(28-10.574/2))) = 1.503\ in^2$

By trial & error, at final stage, $A_s = 1.247\ in^2$, Use 2 Nos #8

For negative moment :

Negative moment = 86.42 kip-ft = 1037.04 inch-kips

$A_s = 0.0214x28x18 = 10.7856\ in^2$

$a = (10.7856x60/(0.85x4x18)) = 10.574$ inch.

$A_s = (1037.04\ /(0.9x60x(28-10.574/2))) = 0.845\ in^2$

By trial & error, at final stage, $A_s = 0.6943\ in^2$, Use 3 Nos #5

For Transverse Interior (TI) frame :

For positive moment :

Positive moment = 307.27 kip-ft = 3687.24 inch-kips

$A_s = 0.0214x28x18 = 10.7856\ in^2$

$a = (10.7856x60/(0.85x4x18)) = 10.574$ inch.

$A_s = (3687.24\ /(0.9x60x(28-10.574/2))) = 3.01\ in^2$

By trial & error, at final stage, $A_s = 2.553$ in^2, Use 4 Nos #8

For negative moment :

Negative moment = 172.84 kip-ft = 2074.08 inch-kips

$A_s = 0.0214 \text{x} 28 \text{x} 18 = 10.7856$ in^2

a = (10.7856x60/(0.85x4x18)) = 10.574 inch.

$A_s = (2074.08 /(0.9 \text{x} 60 \text{x} (28-10.574/2))) = 1.691$ in^2

By trial & error, at final stage, $A_s = 1.406$ in^2, Use 2 Nos #8

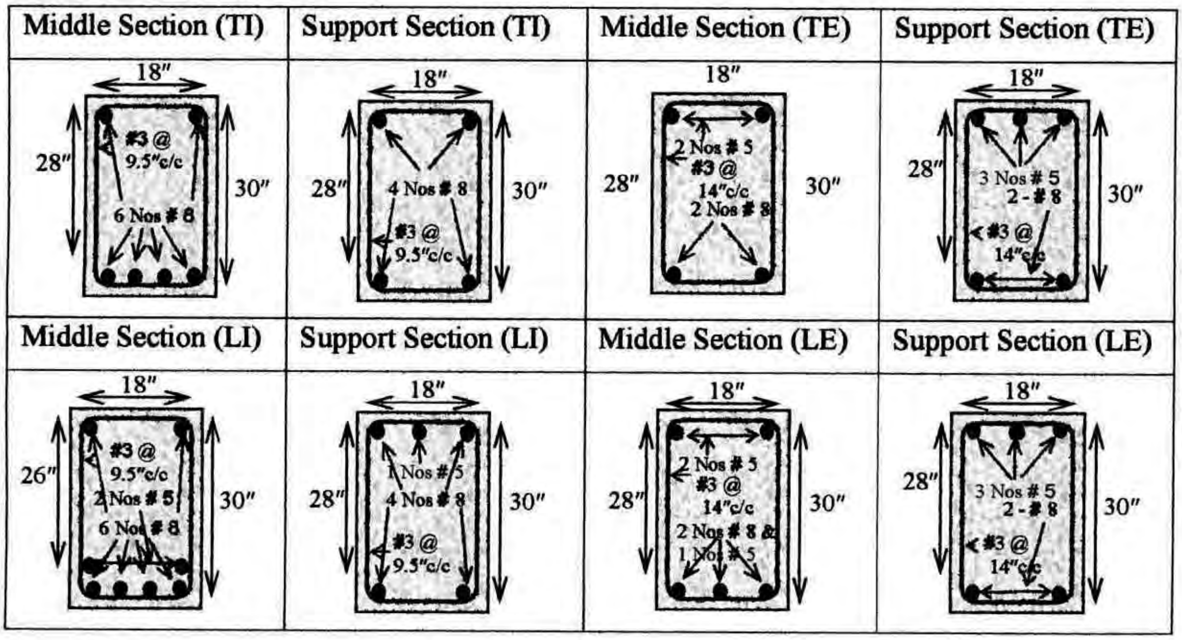


Figure II-29 : Transverse Cross Section of 1st Floor Beam.

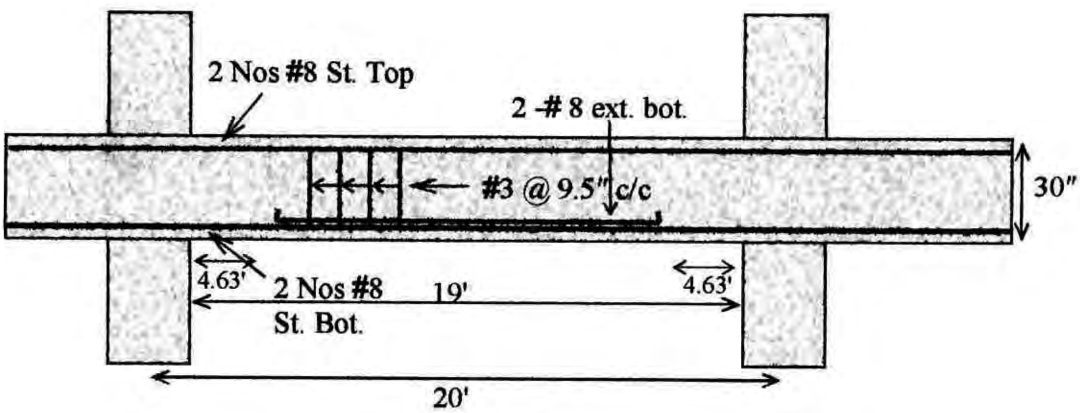


Figure II-30 : Longitudinal Cross Section of (TI) 1st Floor Beam.

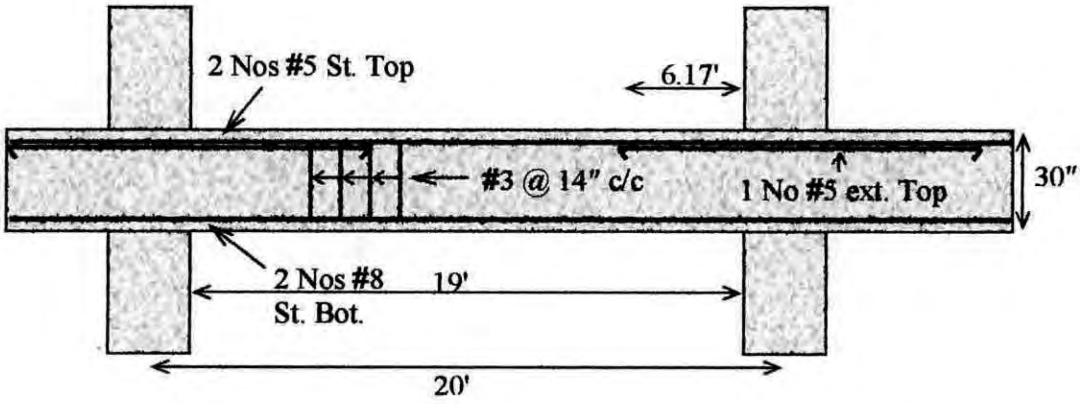


Figure II-31 : Longitudinal Cross Section of (TE) 1st Floor Beam.

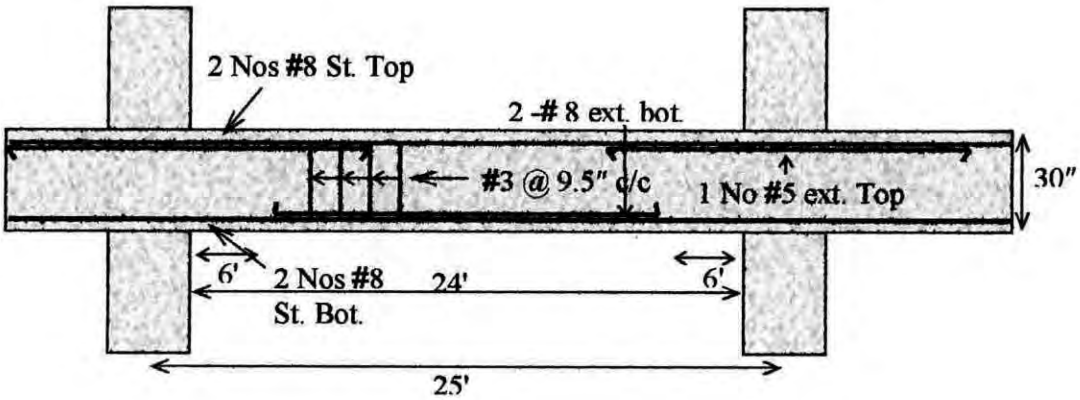


Figure II-32 : Longitudinal Cross Section of (LI) 1st Floor Beam

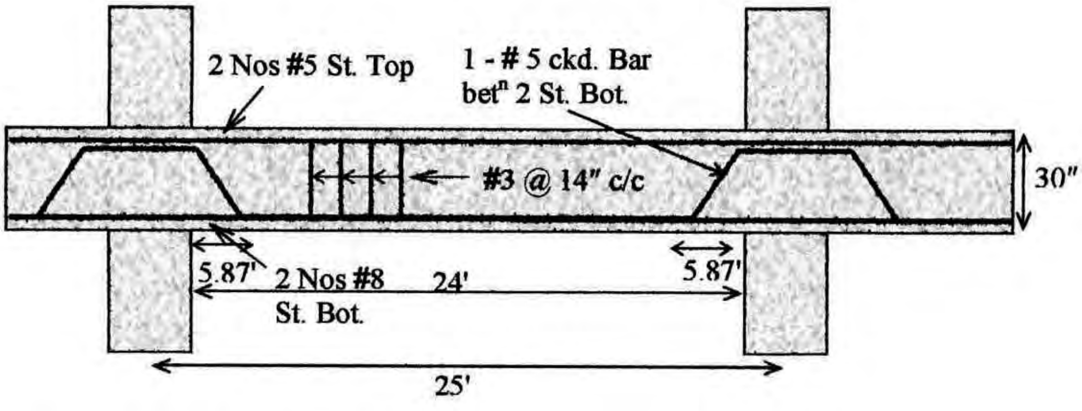


Figure II-33 : Longitudinal Cross Section of (LE) 1st Floor Beam.

Design of Column on 2nd Floor :

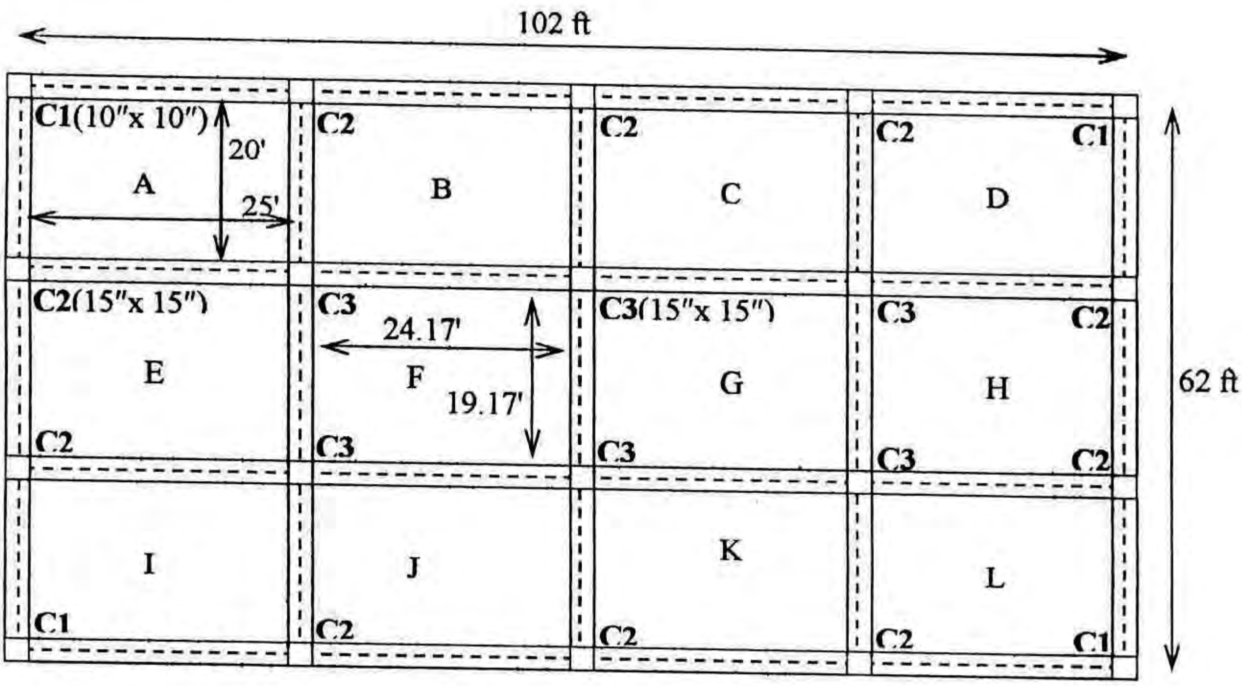


Figure II-34 : Specification of type & size of Column on 2nd Floor.

Design of Column C1 :

Total Load acting on Column, P = 2.99x10+2.422x12.5 = 60.175 kips

P_c = 0.85x0.25x4x10x10 = 85 kips

Here, P< P_c, So, Minimum Reinforcement will be apply. Assume, $\rho_g = 0.01$

A_{st} = 0.01x10x10 = 1 in^2 Use 4 Nos #5.

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x5/8 = 10", S_2 = 48x3/8 = 18", S_3 = Least dimention of column = 10"

Hence, Use #3 No bar @ 10" c/c.

Design of Column C2 :

Total Load acting on Column, P_1 = 5.98x10+2.422x25 = 120.35 kips, P_2 = 2.99x20+4.844x12.5 = 120.35 kips, i.e, P = 120.35 kips.

P_c = 0.85x0.25x4x15x15 = 191.25 kips

Here, P< P_c, So, Minimum Reinforcement will be apply. Assume, $\rho_g = 0.01$

A_{st} = 0.01x15x15 = 2.25 in^2 Use 8 Nos #5.

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x5/8 = 10″, S_2 = 48x3/8 = 18″, S_3 = Least dimention of column = 15″

Hence, Use #3 No bar @ 10″ c/c.

Design of Column C3 :

Total Load acting on Column, P = 5.98x20+4.844x25 = 240.7 kips

P_c = 0.85x0.25x4x18x18 = 275.4 kips

Here, P< P_c , So, Minimum Reinforcement will be apply. Assume, ρ_g = 0.01

A_{st} = 0.01x18x18 = 3.24 in^2 Use 12 Nos #5.

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x5/8 = 10″, S_2 = 48x3/8 = 18″, S_3 = Least dimention of column = 18″

Hence, Use #3 No bar @ 10″ c/c.

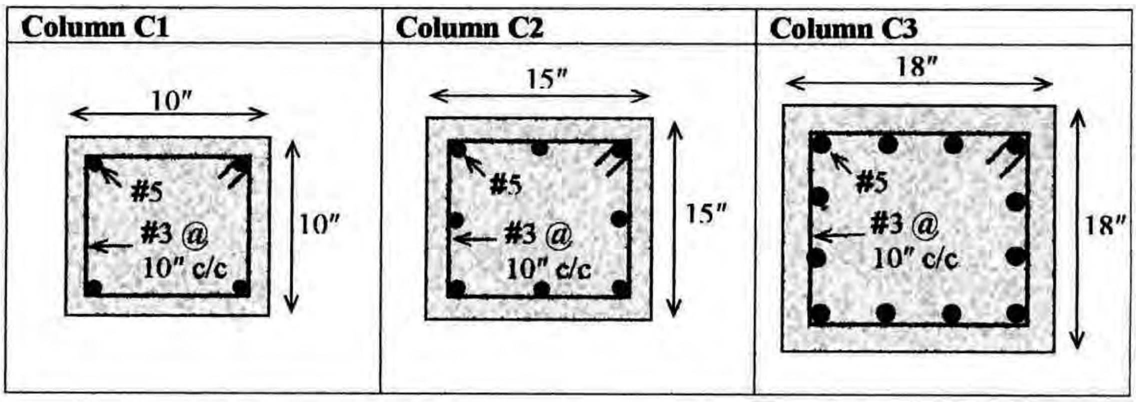


Figure II-35 : Cross Section of Column on 2nd Floor.

<u>**Design of Column on 1st Floor**</u> **:**

For Longitudinal Interior (LI) Direction :

(i) From Beam = 11.82 k/ft

(ii) From Col^m = ((14.542x(4x0.694+10x1.5625+6x2.25)x0.15)/(62x102))x20 = 0.22 k/ft

(iii) From Boundary Wall = (((5x0.83x320x0.15)/(62x102)))x20 = 0.63 k/ft

(iv) From Partition Wall = ((8x(320+80+4x2x15)x0.42)x0.12/(62x102)))x20 = 0.66 k/ft

Total Load, w = (11.821+1.4x(0.22+0.632+0.658) = 13.935 k/ft

For LE : w = 6.97 k/ft, For TI : w = (5.98+8.618)+1.4x(0.275+0.79+0.822) = 17.24 k/ft,

For TE : w = 8.62 k/ft

Design of Column C1 :

Total Load acting on Column, P = 8.62x10+6.97x12.5 = 173.325 kips

P_c = 0.85x0.25x4x24x24 = 489.6 kips

Here, P< P_c, So, Minimum Reinforcement will be apply. Assume, $\rho_g = 0.01$

A_{st} = 0.01x24x24 = 5.76 in^2 Use 8 Nos #8.

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x8/8 = 16", S_2 = 48x3/8 = 18", S_3 = Least dimention of column = 24"

Hence, Use #3 No bar @ 16" c/c.

Design of Column C2 :

Total Load acting on Column, P_1 = 17.24x10+6.97x25 = 346.65 kips, P_2 = 8.62x20+13.935x12.5 = 346.59 kips, i.e, P = 346.65 kips.

P_c = 0.85x0.25x4x24x24 = 489.6 kips

Here, P< P_c, So, Minimum Reinforcement will be apply. Assume, $\rho_g = 0.01$

A_{st} = 0.01x24x24 =5.76 in^2 Use 8 Nos #8.

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x8/8 = 16", S_2 = 48x3/8 = 18", S_3 = Least dimention of column = 24"

Hence, Use #3 No bar @ 16" c/c.

Design of Column C3 :

Total Load acting on Column, P = 17.24x20+13.935x25 = 693.175 kips

P_c = 0.85x0.25x4x24x24 = 489.6 kips

Here, P_s = 693.175-489.6 = 203.575 kips

A_{st} = (203.575/(0.85x24) = 9.98 in^2 Use 12 Nos #8 & 2 Nos #5

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x8/8 = 16″, S_2 = 48x3/8 = 18″, S_3 = Least dimention of column = 24″

Hence, Use #3 No bar @ 16″ c/c.

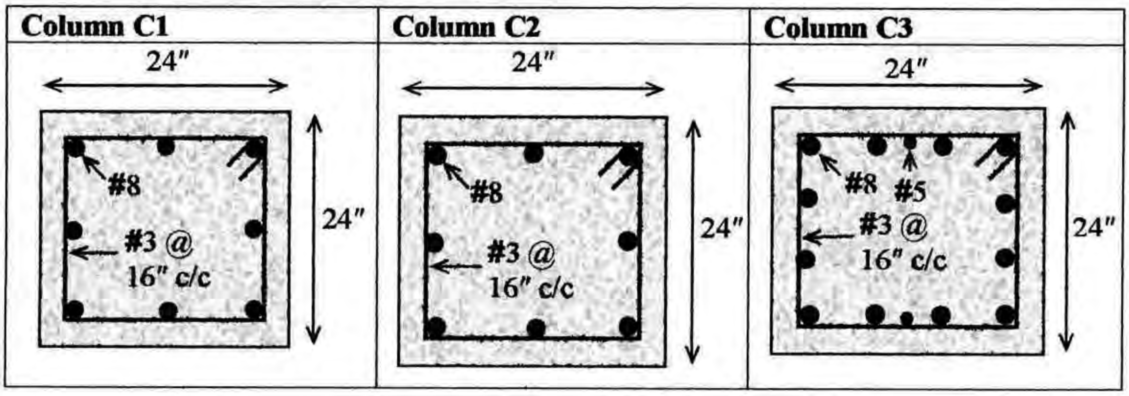


Figure II-36 : Cross Section of Column on 1st Floor.

<u>Design of Column on G.F. Floor</u> :

For Longitudinal Interior (LI) Direction :

(i) From Beam = 19.61 k/ft

(ii) From Col^m = ((19.375x2x2x0.15x6)/(62x102))x20 = 0.22 k/ft

(iii) From Boundary Wall = ((19.375x2x320x0.15)/(62x102))x20 = 5.88 k/ft

(iv) From Partition Wall = (((80+4x2x15)x19.375x0.42x0.12)/(62x102)))x20 = 0.62 k/ft

Total Load, w = (19.61+1.4x(0.22+5.88+0.62) = 29.02 k/ft

For LE : w = 14.51 k/ft, For TI : w =24.2+1.4x(0.276+7.353+0.7725) = 35.96 k/ft,

For TE : w = 17.98 k/ft

Design of Column C1 :

Total Load acting on Column, P = 17.98x10+14.51x12.5 = 361.175 kips

P_c = 0.85x0.25x4x24x24 = 489.6 kips

Here, P< P_c, So, Minimum Reinforcement will be apply. Assume, ρ_g = 0.01

A_{st} = 0.01x24x24 = 5.76 in^2 Use 8 Nos #8.

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x8/8 = 16″, S_2 = 48x3/8 = 18″, S_3 = Least dimention of column = 24″

Hence, Use #3 No bar @ 16″ c/c.

Design of Column C2 :

Total Load acting on Column, P_1 = 35.9621x10+14.51x25 = 722.371 kips, P_2 = 17.98x20+29.02x12.5 = 722.35 kips, i.e, P = 722.371 kips.

P_c = 0.85x0.25x4x24x24 = 489.6 kips

Here, P_s = 722.371-489.6 = 232.75 kips

A_{st} = (232.75/(0.85x24) = 11.409 in^2 Use 14 Nos #8 & 2 Nos #5

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x8/8 = 16″, S_2 = 48x3/8 = 18″, S_3 = Least dimention of column = 24″

Hence, Use #3 No bar @ 16″ c/c.

Design of Column C3 :

Total Load acting on Column, P = 35.9621x20+29.02x25 = 1444.742 kips

P_c = 0.85x0.25x4x36x36 = 1101 kips

Here, P_s = 1444.742-1101 = 343.142 kips

A_{st} = (343.142/(0.85x24) = 16.82 in^2 Use 22 Nos #8

Column C1	Column C2	Column C3

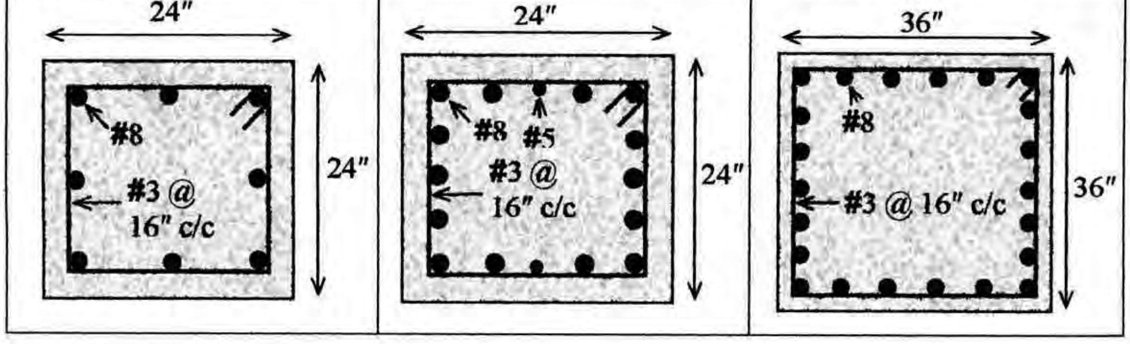


Figure II-37 : Cross Section of Column on G.F. Floor.

Tie Rod :

Use #3 No bar,

Spacing, S_1 = 16x8/8 = 16", S_2 = 48x3/8 = 18", S_3 = Least dimention of column = 36"

Hence, Use #3 No bar @ 16" c/c.

<u>Vertical load Analysis for Ground Floor :</u>

1. **Weight of Slab :**

Roof Slab :	Dead Load	: (5.5/12)x102x62x0.15 = 434.78x1.4 = 608.7 kip
	Wt. of L.C.	: (4/12)x102x62x0.15 = 316.20x1.4 = 442.6 kip
	Wt. of Plaster	: (0.5/12)x102x62x0.15 = 39.025x1.4 = 55.36 kip
	Live Load	: 30x102x62 = 189.72x1.7 = 322.5 kip
2nd Floor :	Dead Load	: (7.5/12)x102x62x0.15 = 592.88x1.4 = 830.0 kip
	Wt. of F.F.	: (1/12)x102x62x0.15 = 79.05x1.4 = 110.67 kip
	Wt. of Plaster	: (0.5/12)x102x62x0.15 = 39.525x1.4 = 55.36 kip
	Live Load	: 100x102x62 = 632.4x1.7 = 1075.1kip
1st Floor :	Dead Load	: (9.5/12)x102x62x0.15 = 750.98x1.4 = 1051.4 kip
	Wt. of F.F.	: (1/12)x102x62x0.15 = 79.05x1.4 = 110.67 kip
	Wt. of Plaster	: (0.5/12)x102x62x0.15 = 39.525x1.4 = 55.36 kip
	Live Load	: 100x102x62 = 632.4x1.7 = 1075.1kip

Total wt. from Slab = 5792.8 kip

2. **Weight of Beam :**

Roof Beam	:((12x18.5/144)x(95x2+96.6x2+57.8x2+56.5x3)x0.15)	x1.4 = 216.37 kip
2nd Floor	:((15x19.5/144)x(92x4+54x5)x0.15)	x1.4 = 272.15 kip
1st Floor	:((18x20.5/144)x(89x2+92x2+54x2+52x3)x0.15)	x1.4 = 336.87 kip

Total wt. from Beam = 825.39 kip

3. **Weight of Column :**

2nd Floor	:((4x10x10+10x15x15+6x18x18)x14.542x0.15)/144)	x1.4 = 97.43 kip
1st Floor	:((20x24x24x19.375x0.15)/144)	x1.4 = 325.50 kip
G.F. Floor	:((6x36x36+14x24x24)21.21x0.15)/144)	x1.4 = 489.91 kip

Total wt. from Column = 912.84 kip

4. **Weight of Boundary Wall:**

From all Floor : (21.21x2x300+19.375x2x300+5x0.83x312.17)x0.15x1.4 = 5386.9 k

5. **Weight of Partition Wall:**

From all Floor : (200x(21.21+19.38+9.5)+312.17x9.5)x0.42x0.12x1.4 = 917.57 k

Grand Total = 13835.5 k

So, Vertical Invert reaction = (13835.5/(102x62)) = 2.19 ksf

Horizontal Pressure at Long-term condition :

At elevation a, P_1 = 132.98x 5.3125x1.3 = 0.9184 ksf

At elevation f, P_2 = 132.98 x 25.4 x1.3 = 4.3903 ksf

At elevation k, P_3 = 132.98 x 48.5 x1.3 = 8.384 ksf

Moment Calculation :

For Wall ak & eo :

$M_a = M_e = (1/16)x((0.9184+4.3903)/2)x20.083^2 = 66.911$ k-ft

$M_{a\text{-}f} = M_{e\text{-}j} = (1/14)x((0.9184+4.3903)/2)x20.083^2 = 67.47$ k-ft

$M_{f1} = M_{j1} = (1/16)x((0.9184+4.3903)/2)x20.083^2 = 66.911$ k-ft

$M_{f2} = M_{j2} = (1/16)x((4.3903+8.384)/2)x23.104^2 = 213.09$ k-ft

$M_f = M_j = (M_{f1} + M_{f2})/2 = 140$ k-ft

$M_{f\text{-}k} = M_{j\text{-}o} = (1/14)x((4.3903+8.384)/2)x23.104^2 = 243.53$ k-ft

$M_{k1} = M_{o1} = (1/16)x((4.3903+8.384)/2)x23.104^2 = 213.09$ k-ft $= M_{I1} = M_{L1}$

For G.F. : (along Longitudinal Direction)

$M_{k2} = M_{o2} = (1/16)x2.19x25^2 = 85.55$ k-ft

$M_k = M_o = (213.09 + 85.55) = 298.64$ k-ft

$M_l = M_n = (1/10)x2.19x25^2 = 136.88$ k-ft

$M_m = M_{o2} = (1/11)x2.19x25^2 = 124.432$ k-ft

$M_{k\text{-}l} = M_{n\text{-}o} = (1/14)x2.19x25^2 = 97.77$ k-ft

$M_{l\text{-}m} = M_{m\text{-}n} = (1/16)x2.19x25^2 = 85.55$ k-ft

For G.F. : (along Short Direction)

$M_{I2} = M_{L2} = (1/16)x2.19x20^2 = 54.75$ k-ft

$M_I = M_L = (213.09 + 54.75) = 267.84$ k-ft

$M_J = M_K = (1/10)x2.19x20^2 = 87.6$ k-ft

$M_{I\text{-}J} = M_{K\text{-}L} = (1/14)x2.19x20^2 = 62.6$ k-ft

$M_{J\text{-}K} = (1/16)x2.19x20^2 = 54.75$ k-ft

Shear Check :

(i) **For G.F. :** $V_u = 1.15x2.19x(25/2) - 2.19x27/2 = 26.55$ kip

$\varphi V_c = 0.85x2x(4000)^{1/2}x12x33 \quad = 42.58$ kip$>V_u$ (ok)

(ii) **For Wall ak & eo :** $V_u = 1.15x6.39x(23.1/2) - 6.39x21/2 = 73.67$ kip

$\varphi V_c = 0.85x2x(4000)^{1/2}x12x21 \quad = 27.09$ kip$>V_u$ (ok)

Spacing, S = (0.85x0.22x60x21)/(73.67-27.09) = 664.02/46.6 = 5.06"≈ 5"

S_{max} = (0.31x2x60000/(50x12)) = 62", S_{max} = 24", S_{max} = (21/2)" = 10.5"

Use #3 @ (5 +10.5)/2 = 7.75" c/c.

Depth Check :

d^2 = (298.64x12)/(0.9x0.0214x12x60x(1-0.59x0.0214x60/4)) = 318.81,

d = 17.9 inch< 21" (ok)

Reinforcement Calculation :

For Wall :

M_a = 66.911 k-ft ; A_s = 0.7265 inch^2 Use #5 @ 5"c/c.

M_{a-f} = 67.47 k-ft ; A_s = 0.8335 inch^2 Use #5 @ 4.25"c/c.

M_f = 140 k-ft ; A_s = (1.57-0.7265) = 0.8435 inch^2 Use #8 @ 11"c/c.

M_{f-k} = 243.53 k-ft ; A_s = (2.8643-0.8335) = 2.031 inch^2 Use #8 @ 4.5"c/c.

M_{k1} = 298.64 k-ft ; A_s = (3.62- 0.7265-0.8435) = 2.05 inch^2 Use #8 @ 4.5"c/c. (for Wall)

For G.F.(Long Direction) :

M_l = 136.88 k-ft ; A_s = 0.942 inch^2 Use #8 @ 10"c/c.

M_m = 124.432 k-ft ; A_s = 0.855 inch^2 Use #8 @ 11"c/c.

M_{k-l} = 97.77 k-ft ; A_s = 0.67 inch^2 Use #5 @ 5.5"c/c.

M_{l-m} = 85.55 k-ft ; A_s = 0.584 inch^2 Use #5 @ 6.25"c/c.

M_{k2} = 298.64 k-ft ; A_s = (0.2.11-0.942) = 1.168 inch^2 Use #8 @ 8"c/c.(for G.F.)

For G.F.(Short Direction) :

M_I = 267.84 k-ft ; A_s = 1.883 inch^2 Use #8 @ 5"c/c. = M_L

M_J = 87.6 k-ft ; A_s = 0.6 inch^2 Use #5 @ 6"c/c.= M_K

M_{I-J} = 62.6 k-ft ; A_s = 0.43 inch^2 Use #5 @ 8.5"c/c. = M_{k-l}

M_{J-K} = 54.75 k-ft ; A_s = 0.372 inch^2 Use #5 @ 10"c/c.

Minimum reinforcement for control Shrinkage & Temperature Crack :

For Wall : A_s = 0.0018x12x21 = 0.4536 inch^2 Use #5 @ 16"c/c. (per 12" strip, both upper & lower section)

The AZ Sheet Pile Range :

Section	Width (w) in	Height (h) in	Flange Thickness (t_f) in	Web Thickness (t_w) in	Cross Sectional Area in^2/ft	Weight Pile lb/ft	Weight Wall lb/ft^2	Section Modulus in^3/ft	Moment Inertia in^4/ft
AZ 12	**26.38**	11.89	0.335	0.335	5.94	44.42	20.22	**22.32**	132.84
		302.0	8.50	8.50	125.7	66.10	98.70		
AZ 13	**26.38**	11.93	0.375	0.375	6.47	48.38	21.92	**24.18**	144.26
		303.0	9.50	9.50	137	72.00	107.00		
AZ 14	**26.38**	11.97	0.413	0.413	7.03	52.62	23.94	**26.04**	155.98
		304.0	10.50	10.50	148.9	78.30	116.90		
AZ 17	**24.80**	14.92	0.335	0.335	6.53	45.96	22.24	**30.97**	231.26
		379.0	8.50	8.50	138.3	68.40	108.60		
AZ 18	**24.80**	14.96	0.375	0.375	7.09	49.99	24.17	**33.48**	250.45
		380.0	9.50	9.50	150	74.40	118.00		
AZ 19	**24.80**	15.00	0.413	0.413	7.74	54.43	26.34	**36.08**	270.80
		381.0	10.50	10.50	163.8	81.00	128.60		
AZ 25	**24.80**	16.77	0.472	0.441	8.74	61.49	29.74	**45.66**	382.63
		426.0	12.00	11.20	185	91.50	145.20		
AZ 26	**24.80**	16.81	0.512	0.480	9.35	65.72	31.75	**48.36**	406.50
		427.0	13.00	12.20	198	97.80	155.00		
AZ 28	**24.80**	16.85	0.551	0.520	9.97	70.15	33.94	**51.24**	431.62
		428.0	14.00	13.20	211.1	104.40	165.70		
AZ 34	**24.80**	18.07	0.669	0.512	11.03	77.61	37.54	**63.80**	576.32
		459.0	17.00	13.00	233.5	115.50	183.30		
AZ 36	**24.80**	18.11	0.709	0.551	11.67	82.11	39.73	**66.96**	606.34
		460.0	18.00	14.00	247	122.20	194.00		
AZ 38	**24.80**	18.15	0.748	0.591	12.33	86.75	41.97	**70.31**	637.69
		461.0	19.00	15.00	261	129.10	204.90		
AZ 46	**22.83**	18.94	0.709	0.551	13.76	89.10	46.82	**85.47**	808.83
		481.0	18.00	14.00	291.2	132.60	228.60		
AZ 48	**22.83**	18.98	0.748	0.591	14.48	93.81	49.28	**89.28**	847.05
		482.0	19.00	15.00	306.5	139.60	240.60		
AZ 50	**22.83**	19.02	0.787	0.630	15.22	98.58	51.80	**93.28**	886.52
		483.0	20.00	16.00	322.2	146.70	252.90		

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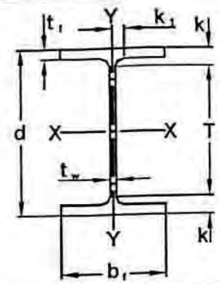

Designation	Area	Depth	Web		Flange		Distance			Nominal Weight per foot	Compact Section Criteria			X_1	$X_2 \times 10^6$	Elastic Properties						Plastic Modulus	
																Axis X-X			Axis Y-Y				
	A	D	Thickness t_w	$t_w/2$	Width b_f	Thickness t_f	T	k	k_1		$b_f/2t_f$	h/t_w	F_y'''	X_1	$X_2 \times 10^6$	I	S	r	I	S	r	Z_x	Z_y
	in^2	in	in	in	in	in	in	in	in	lbs			ksi	ksi	$(1/ksi)^2$	in^4	in^3	in	in^4	in^3	in	in^3	in^3
W 44 x 335	98.3	44.02	1.020	1/2	15.950	1.770	38 7/16	2 9/16	1 5/16	335	4.5	38.1	44	2430	5110	31100	1410	17.8	1200	150	3.49	1620	236
x 290	85.8	43.62	0.870	7/16	15.830	1.580	38 7/16	2 3/8	1 1/4	290	5.0	44.7	32	2140	8220	27100	1240	17.8	1050	133	3.50	1420	206
x 262	77.2	43.31	0.790	3/8	15.750	1.420	38 7/16	2 3/16	1 3/16	262	5.5	49.2	26	1930	12300	24200	1120	17.7	927	118	3.46	1270	183
x 230	67.7	42.91	0.710	3/8	15.750	1.220	38 7/16	2	1 1/8	230	6.5	54.8	21	1690	21200	20800	969	17.5	796	101	3.43	1100	157
W 40 x 593*	174	42.99	1.790	1	16.690	3.230	34 3/16	4 7/16	2 1/16	593	2.6	19.1	-	4790	337	50400	2340	17.0	2520	302	3.81	2760	481
x 503*	148	42.05	1.540	3/4	16.420	2.760	34 3/16	3 15/16	1 15/16	503	3.0	22.2	-	4110	620	41700	1980	16.8	2050	250	3.72	2300	394
x 431	127	41.26	1.340	11/16	16.220	2.360	34 3/16	3 9/16	1 7/8	431	3.4	25.5	-	3550	1100	34800	1690	16.6	1690	208	3.65	1950	327
x 372	109	40.63	1.160	9/16	16.060	2.050	34 3/16	3 1/4	1 3/4	372	3.9	29.5	-	3100	1860	29600	1460	16.4	1420	177	3.60	1670	277
x 321	94.1	40.08	1.000	1/2	15.910	1.770	34 3/16	2 15/16	1 11/16	321	4.5	34.2	55	2690	3240	25100	1250	16.3	1190	150	3.56	1420	234
x 297	87.4	39.84	0.930	1/2	15.825	1.650	34 3/16	3 1/16	1 11/16	297	4.8	36.8	47	2500	4240	23200	1170	16.3	1090	138	3.54	1330	215
x 277	81.3	39.69	0.830	7/16	15.830	1.575	34 3/16	2 3/4	1 5/8	277	5.0	41.2	38	2350	5370	21900	1100	16.4	1040	132	3.58	1250	204
x 249	73.3	39.38	0.750	3/8	15.750	1.420	34 3/16	2 5/8	1 9/16	249	5.5	45.6	31	2120	7940	19500	992	16.3	926	118	3.56	1120	182
x 215	63.3	38.98	0.650	5/16	15.750	1.220	34 3/16	2 3/8	1 1/2	215	6.5	52.6	23	1830	14000	16700	858	16.2	796	101	3.54	963	156
x 199	58.4	38.67	0.650	5/16	15.750	1.065	34 3/16	2 1/4	1 1/2	199	7.4	52.6	23	1690	20300	14900	769	16.0	695	88.2	3.45	868	137
x 174	51.1	38.20	0.650	5/16	15.750	0.830	34 3/16	2	1 1/2	174	9.5	52.6	23	1500	36000	12200	639	15.5	541	68.8	3.26	715	107
W 40 x 466*	137	42.44	1.67	13/16	12.640	2.950	34 3/16	4 1/8	2	466	2.1	20.5	-	4560	473	36300	1710	16.3	1010	160	2.72	2050	262
x 392*	115	41.57	1.42	11/16	12.360	2.520	34 3/16	3 11/16	1 7/8	392	2.5	24.1	-	3920	851	29900	1440	16.1	803	130	2.64	1710	212
x 331	97.6	40.79	1.22	5/8	12.170	2.130	34 3/16	3 5/16	1 13/16	331	2.9	28.0	-	3360	1560	24700	1210	15.9	646	106	2.57	1430	172
x 278	81.8	40.16	1.02	1/2	11.970	1.810	34 3/16	3	1 11/16	278	3.3	33.5	57	2860	2910	20500	1020	15.8	521	87.1	2.52	1190	140
x 264	77.6	40.00	0.960	1/2	11.930	1.730	34 3/16	2 15/16	1 11/16	264	3.4	35.6	50	2720	3510	19400	971	15.8	493	82.6	2.52	1130	132
x 235	68.9	39.69	0.830	7/16	11.890	1.575	34 3/16	2 3/4	1 5/8	235	3.8	41.2	38	2430	5310	17400	874	15.9	444	74.6	2.54	1010	118
x 211	62.0	39.37	0.750	3/8	11.810	1.415	34 3/16	2 5/8	1 9/16	211	4.2	45.6	31	2200	7890	15500	785	15.8	390	66.1	2.51	905	105
x 183	53.7	38.98	0.650	5/16	11.810	1.220	34 3/16	2 3/8	1 1/2	183	4.8	52.6	23	1900	13700	13300	682	15.7	336	56.9	2.50	781	89.6
x 167	49.1	38.59	0.650	5/16	11.810	1.025	34 3/16	2 3/16	1 1/2	167	5.8	52.6	23	1750	20500	11600	599	15.3	283	47.9	2.40	692	76.0
x 149	43.8	38.20	0.630	5/16	11.810	0.830	34 3/16	2	1 1/2	149	7.1	54.3	22	1610	31400	9780	512	14.9	229	38.8	2.29	597	62.2
W 36 x 848*	249	42.45	2.520	1 1/4	18.130	4.530	31 1/8	5 11/16	2 1/4	848	2.0	12.5	-	7100	71	67400	3170	16.4	4550	501	4.27	3830	799
x 798*	234	41.97	2.380	1 3/16	17.990	4.290	31 1/8	5 7/16	2 3/16	798	2.1	13.2	-	6720	87	62600	2980	16.4	4200	467	4.24	3570	743
x 650*	190	40.47	1.970	1	17.575	3.540	31 1/8	4 11/16	2	650	2.5	16.0	-	5590	175	48900	2420	16.0	3230	367	4.12	2840	580
x 527*	154	39.21	1.610	13/16	17.220	2.910	31 1/8	4 1/16	1 3/4	527	3.0	19.6	-	4630	365	38300	1950	15.8	2490	289	4.02	2270	454
x 439*	128	38.26	1.360	11/16	16.965	2.440	31 1/8	3 9/16	1 5/8	439	3.5	23.1	-	3900	704	31000	1620	15.6	1990	235	3.95	1860	367
x 393*	115	37.80	1.220	5/8	16.830	2.200	31 1/8	3 5/16	1 5/8	393	3.8	25.8	-	3540	1040	27500	1450	15.5	1750	208	3.90	1660	325
x 359*	105	37.40	1.120	9/16	16.730	2.010	31 1/8	3 1/8	1 9/16	359	4.2	28.1	-	3240	1470	24800	1320	15.4	1570	188	3.87	1510	292
x 328*	96.4	37.09	1.020	1/2	16.630	1.850	31 1/8	3	1 1/2	328	4.5	30.9	-	2980	2040	22500	1210	15.3	1420	171	3.84	1380	265
x 300	88.3	36.74	0.945	1/2	16.655	1.680	31 1/8	2 13/16	1 1/2	300	5.0	33.3	58	2720	2930	20300	1110	15.2	1300	156	3.83	1260	241
x 280	82.4	36.52	0.885	7/16	16.595	1.570	31 1/8	2 11/16	1 1/2	280	5.3	35.6	51	2560	3730	18900	1030	15.1	1200	144	3.81	1170	223
x 260	76.5	36.26	0.840	7/16	16.550	1.440	31 1/8	2 9/16	1 1/2	260	5.7	37.5	46	2370	5100	17300	953	15.0	1090	132	3.78	1080	204
x 245	72.1	36.08	0.800	7/16	16.510	1.350	31 1/8	2 1/2	1 7/16	245	6.1	39.4	41	2230	6430	16100	895	15.0	1010	123	3.75	1010	190
x 230	67.6	35.90	0.760	3/8	16.470	1.260	31 1/8	2 3/8	1 7/16	230	6.5	41.4	37	2100	8190	15000	837	14.9	940	114	3.73	943	176

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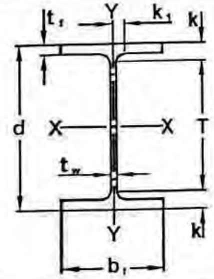

Designation	Area A (in^2)	Depth D (in)	Web Thickness t_w (in)	Web $t_w/2$ (in)	Flange Width b_f (in)	Flange Thickness t_f (in)	Distance T (in)	Distance k (in)	Distance k_1 (in)	Nominal Weight per foot (lbs)	Compact Section Criteria $b_f/2t_f$	Compact Section Criteria h/t_w	Compact Section Criteria F_y''' (ksi)	X_1 (ksi)	$X_2 \times 10^6$ $(1/ksi)^2$	Elastic Axis X-X I (in^4)	Elastic Axis X-X S (in^3)	Elastic Axis X-X r (in)	Elastic Axis Y-Y I (in^4)	Elastic Axis Y-Y S (in^3)	Elastic Axis Y-Y r (in)	Plastic Modulus Z_x (in^3)	Plastic Modulus Z_y (in^3)
W36x256	75.4	37.43	0.960	1/2	12.215	1.730	32 1/8	2 5/8	1 5/16	256	3.5	33.8	56	2840	2870	16800	895	14.9	528	86.5	2.65	1040	137
x232	68.1	37.12	0.870	7/16	12.120	1.570	32 1/8	2 1/2	1 1/4	232	3.9	37.3	46	2580	4160	15000	809	14.8	468	77.2	2.62	936	122
x210	61.8	36.69	0.830	7/16	12.180	1.360	32 1/8	2 5/16	1 1/4	210	4.5	39.1	42	2320	6560	13200	719	14.6	411	67.5	2.58	833	107
x194	57.0	36.49	0.765	3/8	12.115	1.260	32 1/8	2 3/16	1 3/16	194	4.8	42.4	36	2140	8850	12100	664	14.6	375	61.9	2.56	767	97.7
x182	53.6	36.33	0.725	3/8	12.075	1.180	32 1/8	2 1/8	1 3/16	182	5.1	44.8	32	2020	11300	11300	623	14.5	347	57.6	2.55	718	90.7
x170	50.0	36.17	0.680	3/8	12.030	1.100	32 1/8	2	1 3/16	170	5.5	47.8	28	1900	14500	10500	580	14.5	320	53.2	2.53	668	83.8
x160	47.0	36.01	0.650	5/16	12.000	1.020	32 1/8	1 15/16	1 1/8	160	5.9	50.0	26	1780	18600	9750	542	14.4	295	49.1	2.50	624	77.3
x150	44.2	35.85	0.625	5/16	11.975	0.940	32 1/8	1 7/8	1 1/8	150	6.4	52.0	24	1680	24200	9040	504	14.3	270	45.1	2.47	581	70.9
x135	39.7	35.55	0.600	5/16	11.950	0.790	32 1/8	1 11/16	1 1/8	135	7.6	54.1	22	1520	38000	7800	439	14.0	225	37.7	2.38	509	59.7
W33x354*	104.0	35.55	1.160	5/8	16.100	2.090	29 3/4	2 7/8	1 3/8	354	3.8	25.8	-	3540	1030	21900	1230	14.5	1460	181	3.74	1420	282
x318*	93.5	35.16	1.040	9/16	15.985	1.890	29 3/4	2 11/16	1 5/16	318	4.2	29.9	-	3200	1530	19500	1110	14.4	1290	161	3.71	1270	250
x291*	85.6	34.84	0.960	1/2	15.905	1.730	29 3/4	2 9/16	1 1/4	291	4.6	31.2	-	2940	2130	17700	1010	14.4	1160	146	3.69	1150	226
x263*	77.4	34.53	0.870	7/16	15.805	1.570	29 3/4	2 3/8	1 3/16	263	5.0	34.5	54	2670	3100	15800	917	14.3	1030	131	3.66	1040	202
x241	70.9	34.18	0.830	7/16	15.860	1.400	29 3/4	2 3/16	1 3/16	241	5.7	36.1	49	2430	4590	14200	829	14.1	932	118	3.63	939	182
x221	65.0	33.93	0.775	3/8	15.805	1.275	29 3/4	2 1/16	1 3/16	221	6.2	38.7	43	2240	6440	12800	757	14.1	840	106	3.59	855	164
x201	59.1	33.68	0.715	3/8	15.745	1.150	29 3/4	1 15/16	1 1/8	201	6.8	41.9	36	2040	9390	11500	684	14.0	749	95.2	3.56	772	147
W33x169	49.5	33.82	0.670	3/8	11.500	1.220	29 3/4	2 1/16	1 1/8	169	4.7	44.7	32	2160	8150	9290	549	13.7	310	53.9	2.50	629	84.4
x152	44.7	33.49	0.635	5/16	11.565	1.055	29 3/4	1 7/8	1 1/8	152	5.5	47.2	29	1940	12900	8160	487	13.5	273	47.2	2.47	559	73.9
x141	41.6	33.30	0.605	5/16	11.535	0.960	29 3/4	1 3/4	1 1/16	141	6.0	49.6	26	1800	17800	7450	448	13.4	246	42.7	2.43	514	66.9
x130	38.3	33.09	0.580	5/16	11.510	0.855	29 3/4	1 11/16	1 1/16	130	6.7	51.7	24	1660	25100	6710	406	13.2	218	37.9	2.39	467	59.5
x118	34.7	32.86	0.550	5/16	11.480	0.740	29 3/4	1 9/16	1 1/16	118	7.8	54.5	22	1510	37700	5900	359	13.0	187	32.6	2.32	415	51.3
W30x477*	140.0	34.21	1.630	13/16	15.865	2.950	26 3/4	3 3/4	1 9/16	477	2.7	16.6	-	5420	193,	26100	1530	13.7	1970	249	3.75	1790	390
x391*	114.0	33.19	1.360	11/16	15.590	2.440	26 3/4	3 1/4	1 7/16	391	3.2	19.9	-	4510	386	20700	1250	13.5	1550	198	3.68	1430	310
x326*	95.7	32.40	1.140	9/16	15.370	2.050	26 3/4	2 13/16	1 5/16	326	3.7	23.7	-	3860	735	16800	1030	13.2	1240	162	3.61	1190	252
x292*	85.7	32.01	1.020	1/2	15.255	1.850	26 3/4	2 5/8	1 1/4	292	4.1	26.5	-	3460	1110	14900	928	13.2	1100	144	3.58	1060	223
x261	76.7	31.61	0.930	1/2	15.155	1.650	26 3/4	2 7/16	1 3/16	261	4.6	29.0	-	3110	1690	13100	827	13.1	959	127	3.54	941	195
x235	69.0	31.30	0.830	7/16	15.055	1.500	26 3/4	2 1/4	1 1/8	235	5.0	32.5	61	2820	2460	11700	746	13.0	855	114	3.52	845	175
x211	62.0	30.94	0.775	3/8	15.105	1.315	26 3/4	2 1/8	1 1/8	211	5.7	34.9	53	2510	3950	10300	663	12.9	757	100	3.49	749	154
x191	56.1	30.68	0.710	3/8	15.040	1.185	26 3/4	1 15/16	1 1/16	191	6.3	38.0	44	2280	5840	9170	598	12.8	673	89.5	3.46	673	138
x173	50.8	30.44	0.655	5/16	14.985	1.065	26 3/4	1 7/8	1 1/16	173	7.0	41.2	38	2070	8540	8200	539	12.7	598	79.8	3.43	605	123
W30x148	43.5	30.67	0.650	5/16	10.480	1.180	26 3/4	2	1	148	4.4	41.5	37	2310	6180	6680	436	12.4	227	43.3	2.28	500	68.0
x132	38.9	30.31	0.615	5/16	10.545	1.000	26 3/4	1 3/4	1 1/16	132	5.3	43.9	33	2050	10500	5770	380	12.2	196	37.2	2.25	437	58.4
x124	36.5	30.17	0.585	5/16	10.515	0.930	26 3/4	1 11/16	1	124	5.7	46.2	30	1930	13500	5360	355	12.1	181	34.4	2.23	408	54.0
x116	34.2	30.01	0.565	5/16	10.495	0.850	26 3/4	1 5/8	1	116	6.2	48.8	28	1800	17700	4930	329	12.0	164	31.3	2.19	378	49.2
x108	31.7	29.83	0.545	5/16	10.475	0.760	26 3/4	1 9/16	1	108	6.9	49.6	26	1680	24200	4470	299	11.9	146	27.9	2.15	346	43.9
x99	29.1	29.65	0.520	1/4	10.450	0.670	26 3/4	1 7/16	1	99	7.8	51.9	24	1560	34100	3990	269	11.7	128	24.5	2.10	312	38.6
x90	26.4	29.53	0.470	1/4	10.400	0.610	26 3/4	1 5/16	1	90	8.5	57.5	19	1430	47000	3620	245	11.7	115	22.1	2.09	283	34.7

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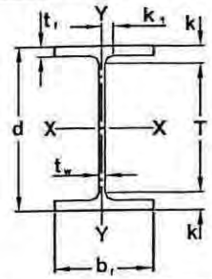

Designation	Area	Depth	Web		Flange		Distance			Nominal Weight per foot	Compact Section Criteria					Elastic Properties						Plastic Modulus	
			Thickness	$t_w/2$	Width	Thickness										Axis X-X			Axis Y-Y				
	A	D	t_w		b_f	t_f	T	k	k_1		$b_f/2t_f$	h/t_w	F_y'''	X_1	$X_2 \times 10^6$	I	S	r	I	S	r	Z_x	Z_y
	in^2	in	in	in	in	in	in	in	in	lbs			ksi	ksi	$(1/ksi)^2$	in^4	in^3	in	in^4	in^3	in	in^3	in^3
W 27 x 539*	158	32.52	1.970	1	15.255	3.540	24	4 1/4	1 5/8	539	2.2	12.3	-	7160	66	25500	1570	12.7	2110	277	3.66	1880	437
x 448*	131	31.42	1.650	13/16	14.940	2.990	24	3 11/16	1 1/2	448	2.5	14.7	-	6070	123	20400	1300	12.5	1670	224	3.57	1530	351
x 368*	108	30.39	1.380	11/16	14.665	2.480	24	3 3/16	1 5/16	368	3.0	17.6	-	5100	243	16100	1060	12.2	1310	179	3.48	1240	279
x 307*	90.2	29.61	1.160	5/8	14.445	2.090	24	2 13/16	1 1/4	307	3.5	20.9	-	4320	463	13100	884	12.0	1050	146	3.42	1020	227
x 258	75.7	28.98	0.980	1/2	14.270	1.770	24	2 1/2	1 1/8	258	4.0	24.7	-	3670	873	10800	742	11.9	859	120	3.37	850	187
x 235	69.1	28.66	0.910	1/2	14.190	1.610	24	2 5/16	1 1/8	235	4.4	26.6	-	3360	1230	9660	674	11.8	768	108	3.33	769	168
x 217	63.8	28.43	0.830	7/16	14.115	1.500	24	2 3/16	1 1/16	217	4.7	29.2	-	3120	1640	8870	624	11.8	704	99.8	3.32	708	154
x 194	57.0	28.11	0.750	3/8	14.035	1.340	24	2 1/16	1	194	5.2	32.3	61	2800	2520	7820	556	11.7	618	88.1	3.29	628	136
x 178	52.3	27.81	0.725	3/8	14.085	1.190	24	1 7/8	1 1/16	178	5.9	33.4	57	2550	3740	6990	502	11.6	555	78.8	3.26	567	122
x 161	47.4	27.59	0.660	3/8	14.020	1.080	24	1 13/16	1	161	6.5	36.7	47	2320	5370	6280	455	11.5	497	70.9	3.24	512	109
x 146	42.9	27.38	0.605	5/16	13.965	0.975	24	1 11/16	1	146	7.2	40.0	40	2110	7900	5630	411	11.4	443	63.5	3.21	461	97.5
W 27 x 129	37.8	27.63	0.610	5/16	10.010	1.100	24	1 13/16	15/16	129	4.5	39.7	41	2390	5340	4760	345	11.2	184	36.8	2.21	395	57.6
x 114	33.5	27.29	0.570	5/16	10.070	0.930	24	1 5/8	15/16	114	5.4	42.5	35	2100	9220	4090	299	11.0	159	31.5	2.18	343	49.3
x 102	30.0	27.09	0.515	1/4	10.015	0.830	24	1 9/16	15/16	102	6.0	47.0	29	1890	14000	3620	267	11.0	139	27.8	2.15	305	43.4
x 94	27.7	26.92	0.490	1/4	9.990	0.745	24	1 7/16	15/16	94	6.7	49.4	26	1740	19900	3270	243	10.9	124	24.8	2.12	278	38.8
x 84	24.8	26.71	0.460	1/4	9.960	0.640	24	1 3/8	15/16	84	7.8	52.7	23	1570	31100	2850	213	10.7	106	21.2	2.07	244	33.2
W 24 x 492*	114	29.65	1.970	1	14.115	3.540	21	4 5/16	1 9/16	492	2.0	10.9	-	7950	43	19100	1290	11.5	1670	237	3.41	1550	375
x 408*	119	28.54	1.650	13/16	13.800	2.990	21	3 3/4	1 3/8	408	2.3	13.1	-	6780	79	15100	1060	11.3	1320	191	3.33	1250	300
x 335*	98.4	27.52	1.380	11/16	13.520	2.480	21	3 1/4	1 1/4	335	2.7	15.6	-	5700	156	11900	864	11.0	1030	152	3.23	1020	238
x 279*	82.0	26.73	1.160	5/8	13.305	2.090	21	2 7/8	1 1/8	279	3.2	18.6	-	4840	297	9600	718	10.8	823	124	3.17	835	193
x 250*	73.5	26.34	1.040	9/16	13.185	1.890	21	2 11/16	1 1/8	250	3.5	20.7	-	4370	436	8490	644	10.7	724	110	3.14	744	171
x 229	67.2	26.02	0.960	1/2	13.110	1.730	21	2 1/2	1	229	3.8	22.5	-	4020	605	7650	588	10.7	651	99.4	3.11	676	154
x 207	60.7	25.71	0.870	7/16	13.010	1.570	21	2 3/8	1	207	4.1	24.8	-	3650	876	6820	531	10.6	578	88.8	3.08	606	137
x 192	56.3	25.47	0.810	7/16	12.950	1.460	21	2 1/4	1	192	4.4	26.6	-	3410	1150	6260	491	10.5	530	81.8	3.07	559	126
x 176	51.7	25.24	0.750	3/8	12.890	1.340	21	2 1/8	15/16	176	4.8	28.7	-	3140	1590	5680	450	10.5	479	74.3	3.04	511	115
x 162	47.7	25.00	0.705	3/8	12.955	1.220	21	2	1 1/16	162	5.3	30.6	-	2870	2260	5170	414	10.4	443	68.4	3.05	468	105
x 146	43.0	24.74	0.650	5/16	12.900	1.090	21	1 7/8	1 1/16	146	5.9	33.2	58	2590	3420	4580	371	10.3	391	60.5	3.01	418	93.2
x 131	38.5	24.48	0.605	5/16	12.855	0.960	21	1 3/4	1 1/16	131	6.7	35.6	50	2330	5290	4020	329	10.2	340	53.0	2.97	370	81.5
x 117	34.4	24.26	0.550	5/16	12.800	0.850	21	1 5/8	1	117	7.5	39.2	42	2090	8190	3540	291	10.1	297	46.5	2.94	327	71.4
x 104	30.6	24.06	0.500	1/4	12.750	0.750	21	1 1/2	1	104	8.5	43.1	34	1860	12900	3100	258	10.1	259	40.7	2.91	289	62.4
W 24 x 103	30.3	24.53	0.550	5/16	9.000	0.980	21	1 3/4	13/16	103	4.6	39.2	42	2400	5280	3000	245	9.96	119	26.5	1.99	280	41.5
x 94	27.7	24.31	0.515	1/4	9.065	0.875	21	1 5/8	1	94	5.2	41.9	37	2180	7800	2700	222	9.87	109	24.0	1.98	254	37.5
x 84	24.7	24.10	0.470	1/4	9.020	0.770	21	1 9/16	15/16	84	5.9	45.9	30	1950	12200	2370	196	9.79	94.4	20.9	1.95	224	32.6
x 76	22.4	23.92	0.440	1/4	8.990	0.680	21	1 7/16	15/16	76	6.6	49.0	27	1760	18600	2100	176	9.69	82.5	18.4	1.92	200	28.6
x 68	20.1	23.73	0.415	1/4	8.965	0.585	21	1 3/8	15/16	68	7.7	52.0	24	1590	29000	1830	154	9.55	70.4	15.7	1.87	177	24.5
W 24 x 62	18.2	23.74	0.430	1/4	7.040	0.590	21	1 3/8	15/16	62	6.0	50.1	25	1700	25100	1550	131	9.23	34.5	9.80	1.38	153	15.7
x 55	16.2	23.57	0.395	3/16	7.005	0.505	21	1 5/16	15/16	55	6.9	54.6	21	1540	39600	1350	114	9.11	29.1	8.30	1.34	134	13.3

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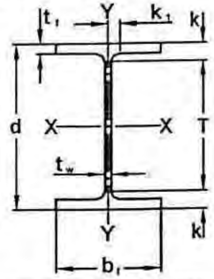

Designation	Area A	Depth D	Web Thickness t_w	Web $t_w/2$	Flange Width b_f	Flange Thickness t_f	Distance T	Distance k	Distance k_1	Nominal Weight per foot	Compact Section Criteria $b_f/2t_f$	Compact Section Criteria h/t_w	Compact Section Criteria F_y'''	X_1	$X_2 \times 10^6$	Elastic Properties Axis X-X I	Axis X-X S	Axis X-X r	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Plastic Modulus Z_x	Plastic Modulus Z_y
	in^2	in	in	in	in	in	in	in	in	lbs			ksi	ksi	$(1/ksi)^2$	in^4	in^3	in	in^4	in^3	in	in^3	in^3
W 21 x 201	59.2	23.03	0.910	1/2	12.575	1.630	18 1/4	2 3/8	1	201	3.9	20.6	-	4290	453	5310	461	9.47	542	86.1	3.02	530	133
x 182	53.6	22.72	0.830	7/16	12.500	1.480	18 1/4	2 1/4	1	182	4.2	22.6	-	3910	649	4730	417	9.40	483	77.2	3.00	476	119
x 166	48.8	22.48	0.750	3/8	12.420	1.360	18 1/4	2 1/8	15/16	166	4.6	24.9	-	3590	904	4280	380	9.36	435	70.1	2.98	432	108
x 147	43.2	22.06	0.720	3/8	12.510	1.150	18 1/4	1 7/8	1 1/16	147	5.4	26.1	-	3140	1590	3630	329	9.17	376	60.1	2.95	373	92.6
x 132	48.8	21.83	0.650	5/16	12.440	1.035	18 1/4	1 13/16	1	132	6.0	28.9	-	2840	2350	3220	295	9.12	333	53.5	2.93	333	82.3
x 122	35.9	21.68	0.600	5/16	12.390	0.960	18 1/4	1 11/16	1	122	6.5	31.3	-	2630	3160	2960	273	9.09	305	49.2	2.92	307	75.6
x 111	32.7	21.51	0.550	5/16	12.340	0.875	18 1/4	1 5/8	15/16	111	7.1	34.1	55	2400	4510	2670	249	9.05	274	44.5	2.90	279	68.2
x 101	29.8	21.36	0.500	1/4	12.290	0.800	18 1/4	1 9/16	15/16	101	7.7	37.5	45	2200	6400	2420	227	9.02	248	40.3	2.89	253	61.7
W 21 x 93	27.3	21.62	0.580	5/16	8.420	0.930	18 1/4	1 11/16	1	93	4.5	32.3	61	2680	3460	2070	192	8.70	92.9	22.1	1.84	221	34.7
x 83	24.3	21.43	0.515	1/4	8.355	0.835	18 1/4	1 9/16	15/16	83	5.0	36.4	48	2400	5250	1830	171	8.67	81.4	19.5	1.83	196	30.5
x 73	21.5	21.24	0.455	1/4	8.295	0.740	18 1/4	1 1/2	15/16	73	5.6	41.2	38	2140	8380	1600	151	8.64	70.6	17.0	1.81	172	26.6
x 68	20.0	21.13	0.430	1/4	8.270	0.685	18 1/4	1 7/16	7/8	68	6.0	43.6	34	2000	10900	1480	140	8.60	64.7	15.7	1.80	160	24.4
x 62	18.3	20.99	0.400	3/16	8.240	0.615	18 1/4	1 3/8	7/8	62	6.7	46.9	29	1820	15900	1330	127	8.54	57.5	13.9	1.77	144	21.7
W 21 x 57	16.7	21.06	0.405	3/16	6.555	0.650	18 1/4	1 3/8	7/8	57	5.0	46.3	30	1960	13100	1170	111	8.36	30.6	9.35	1.35	129	14.8
x 50	14.7	20.83	0.380	3/16	6.530	0.535	18 1/4	1 5/16	7/8	50	6.1	49.4	26	1730	22600	984	94.5	8.18	24.9	7.64	1.30	110	12.2
x 44	13.0	20.66	0.350	3/16	6.500	0.450	18 1/4	1 3/16	7/8	44	7.2	53.6	22	1550	36600	843	81.6	8.06	20.7	6.36	1.26	95.4	10.2
W 18 x 311*	91.5	22.32	1.520	3/4	12.005	2.740	15 1/2	3 7/16	1 3/16	311	2.2	10.6	-	8160	38	6960	624	8.72	795	132	2.95	753	207
x 283*	83.2	21.85	1.400	11/16	11.890	2.500	15 1/2	3 3/16	1 3/16	283	2.4	11.5	-	7520	52	6160	564	8.61	704	118	2.91	676	185
x 258*	75.9	21.46	1.280	5/8	11.770	2.300	15 1/2	3	1 1/8	258	2.6	12.5	-	6920	71	5510	514	8.53	628	107	2.88	611	166
x 234*	68.8	21.06	1.160	5/8	11.650	2.110	15 1/2	2 3/4	1	234	2.8	13.8	-	6360	97	4900	466	8.44	558	95.8	2.85	549	149
x 211*	62.1	20.67	1.060	9/16	11.555	1.910	15 1/2	2 9/16	1	211	3.0	15.1	-	5800	140	4330	419	8.35	493	85.3	2.82	490	132
x 192	56.4	20.35	0.960	1/2	11.455	1.750	15 1/2	2 7/16	15/16	192	3.3	16.7	-	5320	194	3870	380	8.28	440	76.8	2.79	442	119
x 175	51.3	20.04	0.890	7/16	11.375	1.590	15 1/2	2 1/4	7/8	175	3.6	18.0	-	4870	274	3450	344	8.20	391	68.8	1.76	398	106
x 158	46.3	19.72	0.810	7/16	11.300	1.440	15 1/2	2 1/8	7/8	158	3.9	19.8	-	4430	396	3060	310	8.12	347	61.4	2.74	356	94.5
x 143	42.1	19.49	0.730	3/8	11.220	1.320	15 1/2	2	13/16	143	4.2	21.9	-	4060	557	2750	282	8.09	311	55.5	2.72	322	85.4
x 130	38.2	19.25	0.670	3/8	11.160	1.200	15 1/2	1 7/8	13/16	130	4.6	23.9	-	3710	789	2460	256	8.03	278	49.9	2.70	291	76.7
W 18 x 119	35.1	18.97	0.655	5/16	11.265	1.060	15 1/2	1 3/4	15/16	119	5.3	24.5	-	3340	1210	2190	231	7.90	253	44.9	2.69	261	69.1
x 106	31.1	18.73	0.590	5/16	11.200	0.940	15 1/2	1 5/8	15/16	106	6.0	27.2	-	2990	1880	1910	204	7.84	220	39.4	2.66	230	60.5
x 97	28.5	18.59	0.535	5/16	11.145	0.870	15 1/2	1 9/16	7/8	97	6.4	30.0	-	2750	2580	1750	188	7.82	201	36.1	2.65	211	55.3
x 86	25.3	18.39	0.480	1/4	11.090	0.770	15 1/2	1 7/16	7/8	86	7.2	33.4	57	2460	4060	1530	166	7.77	175	31.6	2.63	186	48.4
x 76	22.3	18.21	0.425	1/4	11.035	0.680	15 1/2	1 3/8	13/16	76	8.1	37.8	45	2180	6520	1330	146	7.73	152	27.6	2.61	163	42.2
W 18 x 71	20.8	18.47	0.495	1/4	7.635	0.810	15 1/2	1 1/2	7/8	71	4.7	32.4	61	2680	3310	1170	127	7.50	60.3	15.8	1.70	145	24.7
x 65	19.1	18.35	0.450	1/4	7.590	0.750	15 1/2	1 7/16	7/8	65	5.1	35.7	50	2470	4540	1070	117	7.49	54.8	14.4	1.69	133	22.5
x 60	17.6	18.24	0.415	1/4	7.555	0.695	15 1/2	1 3/8	13/16	60	5.4	38.7	43	2290	6080	984	108	7.47	50.1	13.3	1.69	123	20.6
x 55	16.2	18.11	0.390	3/16	7.530	0.630	15 1/2	1 5/16	13/16	55	6.0	41.2	38	2110	8540	890	98.3	7.41	44.9	11.9	1.67	112	18.5
x 50	14.7	17.99	0.355	3/16	7.495	0.570	15 1/2	1 1/4	13/16	50	6.6	45.2	31	1920	12400	800	88.9	7.38	40.1	10.7	1.65	101	16.6
W 18 x 40	13.5	18.06	0.360	3/16	6.060	0.605	15 1/2	1 1/4	13/16	46	5.0	44.6	32	2060	10100	712	78.8	7.25	22.5	7.43	1.29	90.7	11.7
x 40	11.8	17.90	0.315	3/16	6.015	0.525	15 1/2	1 3/16	13/16	40	5.7	51.0	25	1810	17200	612	68.4	7.21	19.1	6.35	1.27	78.4	9.95
x 35	10.3	17.70	0.300	3/16	6.000	0.425	15 1/2	1 1/8	3/4	35	7.1	53.5	22	1590	30300	510	57.6	7.04	15.3	5.12	1.22	66.5	8.06

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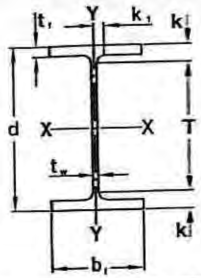

Designation	Area A	Depth D	Web: Thickness t_w	Web: $t_w/2$	Flange: Width b_f	Flange: Thickness t_f	Distance: T	Distance: k	Distance: k_1	Nominal Weight per foot	Compact Section Criteria: $b_f/2t_f$	Compact Section Criteria: h/t_w	Compact Section Criteria: F_y'''	X_1	$X_2 \times 10^6$	Elastic Properties Axis X-X: I	Axis X-X: S	Axis X-X: r	Axis Y-Y: I	Axis Y-Y: S	Axis Y-Y: r	Plastic Modulus Z_x	Plastic Modulus Z_y
	in^2	in	in	in	in	in	in	in	in	lbs			ksi	ksi	$(1/ksi)^2$	in^4	in^3	in	in^4	in^3	in	in^3	in^3
W 16 x 100	29.4	16.97	0.585	5/16	10.425	0.985	13 5/8	1 11/16	15/16	100	5.3	24.3	-	3450	1040	1490	175	7.10	186	35.7	2.51	198	54.9
x 89	26.2	16.75	0.525	1/4	10.365	0.875	13 5/8	1 9/16	7/8	89	5.9	27.0	-	3090	1630	1300	155	7.05	163	31.4	2.49	175	48.1
x 77	22.6	16.52	0.455	1/4	10.295	0.760	13 5/8	1 7/16	7/8	77	6.8	31.2	-	2680	2790	1110	134	7.00	138	26.9	2.47	150	41.1
x 67	19.7	16.33	0.395	3/16	10.235	0.665	13 5/8	1 3/8	13/16	67	7.7	35.9	50	2350	4690	954	117	6.96	119	23.2	2.46	130	35.5
W 16 x 57	16.8	16.43	0.430	1/4	7.120	0.715	13 5/8	1 3/8	7/8	57	5.0	33.0	59	2650	3400	758	92.2	6.72	43.1	12.1	1.60	105	18.9
x 50	14.7	16.26	0.380	3/16	7.070	0.650	13 5/8	1 5/16	13/16	50	5.6	37.4	46	2340	5530	659	81.0	6.68	37.2	10.5	1.59	92.0	16.3
x 45	13.3	16.13	0.345	3/16	7.035	0.565	13 5/8	1 1/4	13/16	45	6.2	41.2	38	2120	8280	586	72.7	6.65	32.8	9.34	1.57	82.3	14.5
x 40	11.8	16.01	0.305	3/16	6.995	0.505	13 5/8	1 3/16	13/16	40	6.9	46.6	30	1890	12900	518	64.7	6.63	28.9	8.25	1.57	72.9	12.7
x 36	10.6	15.86	0.295	3/16	6.985	0.430	13 5/8	1 1/8	3/4	36	8.1	48.1	28	1700	20800	448	56.5	6.51	24.5	7.00	1.52	64.0	10.8
W 16 x 31	9.12	15.88	0.275	1/8	5.525	0.440	13 5/8	1 1/8	3/4	31	6.3	51.6	24	1740	20000	375	47.2	6.41	12.4	4.49	1.17	54.0	7.03
x 26	7.68	15.69	0.250	1/8	5.500	0.345	13 5/8	1 1/16	3/4	26	8.0	56.8	20	1470	409000	301	38.4	6.26	9.59	3.49	1.12	44.2	5.48
W 14 x 808*	237	22.84	3.740	1 7/8	18.560	5.120	11 1/4	5 13/16	2 1/2	808	1.8	3.4	-	18900	1.45	16000	1400	8.21	5510	594	4.82	1834	927
x 730*	215	22.42	3.070	1 9/16	17.890	4.910	11 1/4	5 9/16	2 3/16	730	1.8	3.7	-	17500	1.90	14300	1280	8.17	4720	527	4.69	1660	816
x 665*	196	21.64	2.830	1 7/16	17.650	4.520	11 1/4	5 3/16	2 1/16	665	2.0	4.0	-	16300	2.50	12400	1150	7.98	4170	472	4.62	1480	730
x 605*	178	20.92	2.595	1 5/16	17.415	4.160	11 1/4	4 13/16	1 15/16	605	2.1	4.4	-	15100	3.20	10800	1040	7.80	3680	423	4.55	1320	652
x 550*	162	20.24	2.380	1 3/16	17.200	3.820	11 1/4	4 1/2	1 13/16	550	2.3	4.8	-	14200	4.20	9430	931	7.63	3250	378	4.49	1180	583
x 500*	147	19.60	2.190	1 1/8	17.010	3.500	11 1/4	4 3/16	1 3/4	500	2.4	5.2	-	13100	5.50	8210	838	7.48	2880	339	4.43	1050	522
x 455*	134	19.02	2.015	1	16.835	3.210	11 1/4	3 7/8	1 5/8	455	2.6	5.7	-	12200	7.30	7190	756	7.33	2560	304	4.38	936	468
W 14 x 426*	125	18.67	1.875	15/16	16.695	3.035	11 1/4	3 11/16	1 9/16	426	2.8	6.1	-	11500	8.90	6600	707	7.26	2360	283	4.34	869	434
x 398*	117	18.29	1.770	7/8	16.590	2.845	11 1/4	3 1/2	1 1/2	398	2.9	6.4	-	10900	11.0	6000	656	7.16	2170	262	4.31	801	402
x 370*	109	17.92	1.655	13/16	16.475	2.660	11 1/4	3 5/16	1 7/16	370	3.1	6.9	-	10300	13.9	5440	607	7.07	1990	241	4.27	736	370
x 342*	101	17.54	1.540	13/16	16.360	2.470	11 1/4	3 1/8	1 3/8	342	3.3	7.4	-	9600	17.9	4900	559	6.98	1810	221	4.24	672	338
x 311*	91.4	17.12	1.410	3/4	16.230	2.260	11 1/4	2 15/16	1 5/16	311	3.6	8.1	-	8820	24.4	4330	506	6.88	1610	199	4.20	603	304
x 283*	83.3	16.74	1.290	11/16	16.110	2.070	11 1/4	2 3/4	1 1/4	283	3.9	8.8	-	8120	33.4	3840	459	6.79	1440	179	4.17	542	274
x 257*	75.6	16.38	1.175	5/8	15.995	1.890	11 1/4	2 9/16	1 3/16	257	4.2	9.7	-	7460	46.1	3400	415	6.71	1290	161	4.13	487	246
x 233*	68.5	16.04	1.070	9/16	15.890	1.720	11 1/4	2 3/8	1 3/16	233	4.6	10.7	-	6820	64.9	3010	375	6.63	1150	145	4.10	436	221
x 211	62.0	15.72	0.980	1/2	15.800	1.560	11 1/4	2 1/4	1 1/8	211	5.1	11.6	-	6230	91.8	2660	338	6.55	1030	130	4.07	390	198
x 193	56.8	15.48	0.890	7/16	15.710	1.440	11 1/4	2 1/8	1 1/16	193	5.5	12.8	-	5740	125	2400	310	6.50	931	119	4.05	355	180
x 176	51.8	15.22	0.830	7/16	15.650	1.310	11 1/4	2	1 1/16	176	6.0	13.7	-	5280	173	2140	281	6.43	838	107	4.02	320	163
x 159	46.7	14.98	0.745	3/8	15.565	1.190	11 1/4	1 7/8	1	159	6.5	15.3	-	4790	249	1900	254	6.38	748	96.2	4.00	287	146
x 145	42.7	14.78	0.680	3/8	15.500	1.090	11 1/4	1 3/4	1	145	7.1	16.8	-	4400	348	1710	232	6.33	677	87.3	3.98	260	133
W 14 x 132	38.8	14.66	0.645	5/16	14.725	1.030	11 1/4	1 11/16	15/16	132	7.1	17.7	-	4180	428	1530	209	6.28	548	74.5	3.76	234	113
x 120	35.3	14.48	0.590	5/16	14.670	0.940	11 1/4	1 5/8	15/16	120	7.8	19.3	-	3830	601	1380	190	6.24	495	67.5	3.74	212	102
x 109	32.0	14.32	0.525	1/4	14.605	0.860	11 1/4	1 9/16	7/8	109	8.5	21.7	-	3490	853	1240	173	6.22	447	61.2	3.73	192	92.7
x 99	29.1	14.16	0.485	1/4	14.565	0.780	11 1/4	1 7/16	7/8	99	9.3	23.5	-	3190	1220	1110	157	6.17	402	55.2	3.71	173	83.6
x 90	26.5	14.02	0.440	1/4	14.520	0.710	11 1/4	1 3/8	7/8	90	10.2	25.9	-	2900	1750	999	143	6.14	362	49.9	3.70	157	75.6
W 14 x 82	24.1	14.31	0.510	1/4	10.130	0.855	11	1 5/8	1	82	5.9	22.4	-	3600	846	882	123	6.05	148	29.3	2.48	139	44.8
x 74	21.8	4.17	0.450	1/4	10.070	0.785	11	1 9/16	15/16	74	6.4	25.3	-	3290	1190	796	112	6.04	134	26.6	2.48	126	40.6
x 68	20.0	14.04	0.415	1/4	10.035	0.720	11	1 1/2	15/16	68	7.0	27.5	-	3020	1650	723	103	6.01	121	24.2	2.46	115	36.9
x 61	17.9	13.89	0.375	3/16	9.995	0.645	11	1 7/16	15/16	61	7.7	30.4	-	2720	2460	640	92.2	5.98	107	21.5	2.45	102	32.8

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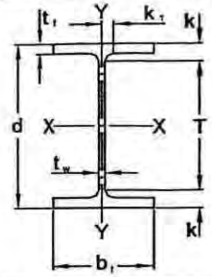

Designation	Area	Depth	Web		Flange		Distance			Nominal Weight per foot	Compact Section Criteria					Elastic Properties						Plastic Modulus	
																Axis X-X			Axis Y-Y				
	A	D	Thickness t_w	$t_w/2$	Width b_f	Thickness t_f	T	k	k_1		$b_f/2t_f$	h/t_w	F_y'''	X_1	$X_2 \times 10^6$	I	S	r	I	S	r	Z_x	Z_y
	in^2	in	in	in	in	in	in	in	in	lbs			ksi	ksi	$(1/ksi)^2$	in^4	in^3	in	in^4	in^3	in	in^3	in^3
W 14 x 53	15.6	13.92	0.370	3/16	8.060	0.660	11	1 7/16	15/16	53	6.1	30.8	-	2830	2250	541	77.8	5.89	57.7	14.3	1.92	87.1	22.0
x 48	14.1	13.79	0.340	3/16	8.030	0.595	11	1 3/8	7/8	48	6.7	33.5	57	2580	3220	485	70.3	5.85	51.4	12.8	1.91	78.4	19.6
x 43	12.6	13.66	0.305	3/16	7.995	0.530	11	1 5/16	7/8	43	7.5	37.4	46	2320	4900	428	62.7	5.82	45.2	11.3	1.89	69.6	17.3
W 14 x 38	11.2	14.10	0.310	3/16	6.770	0.515	12	1 1/16	5/8	38	6.6	39.6	41	2190	6850	385	54.6	5.87	26.7	7.88	1.55	61.5	12.1
x 34	10.0	13.98	0.285	3/16	6.745	0.455	12	1	5/8	34	7.4	43.1	35	1970	10600	340	48.6	5.83	23.3	6.91	1.53	54.6	10.6
x 30	8.85	13.84	0.270	1/8	6.730	0.385	12	15/16	5/8	30	8.7	45.4	31	1750	17600	291	42.0	5.73	19.6	5.82	1.49	47.3	8.99
W 14 x 26	7.69	13.91	0.255	1/8	5.025	0.420	12	15/16	9/16	26	6.0	48.1	28	1890	13900	245	35.3	5.65	8.91	3.54	1.08	40.2	5.54
x 22	6.49	13.74	0.230	1/8	5.000	0.335	12	7/8	9/16	22	7.5	53.3	22	1610	27300	199	29.0	5.54	7.00	2.80	1.04	33.2	4.39
W 12 x 336*	98.8	16.82	1.775	7/8	13.385	2.955	9 1/2	3 11/16	1 1/2	336	2.3	5.5	-	12800	6.05	4060	483	6.41	1190	177	3.47	603	274
x 305*	89.6	16.32	1.625	13/16	13.235	2.705	9 1/2	3 7/196	1 7/16	305	2.4	6.0	-	11800	8.17	3550	435	6.29	1050	159	3.42	537	244
x 279*	81.9	15.85	1.530	3/4	13.140	2.470	9 1/2	3 3/16	1 3/8	279	2.7	6.3	-	11000	10.8	3110	393	6.16	937	143	3.38	481	220
x 252*	74.1	15.41	1.395	11/16	13.005	2.250	9 1/2	2 15/16	1 5/16	252	2.9	7.0	-	10100	14.7	2720	353	6.06	828	127	3.34	428	196
x 230*	67.7	15.05	1.285	11/16	12.895	2.070	9 1/2	2 3/4	1 1/4	230	3.1	7.6	-	9390	19.7	2420	321	5.97	742	115	3.31	386	177
x 210*	61.8	14.71	1.180	5/8	12.790	1.900	9 1/2	2 5/8	1 1/4	210	3.4	8.2	-	8670	26.6	2140	292	5.89	664	104	3.28	348	159
x 190	55.8	14.38	1.060	9/16	12.670	1.735	9 1/2	2 7/16	1 3/16	190	3.7	9.2	-	7940	37.0	1890	263	5.82	589	93.0	3.25	311	143
x 170	50.0	14.03	0.960	1/2	12.570	1.560	9 1/2	2 1/4	1 1/8	170	4.0	10.1	-	7190	54.0	1650	235	5.74	517	82.3	3.22	275	126
x 152	44.7	13.71	0.870	7/16	12.480	1.400	9 1/2	2 1/8	1 1/16	152	4.5	11.2	-	6510	79.3	1430	209	5.66	454	72.8	3.19	243	111
x 136	39.9	13.41	0.790	7/16	12.400	1.250	9 1/2	1 15/16	1	136	5.0	12.3	-	5850	119	1240	186	5.58	398	64.2	3.16	214	98.0
x 120	35.3	13.12	0.710	3/8	12.320	1.105	9 1/2	1 13/16	1	120	5.6	13.7	-	5240	184	1070	163	5.51	345	56.0	3.13	186	85.4
x 106	31.2	12.89	0.610	5/16	12.220	0.990	9 1/2	1 11/16	15/16	106	6.2	15.9	-	4660	285	933	145	5.47	301	49.3	3.11	164	75.1
x 96	28.2	12.71	0.550	5/16	12.160	0.900	9 1/2	1 5/8	7/8	96	6.8	17.7	-	4250	405	833	131	5.44	270	44.4	3.09	147	67.5
x 87	25.6	12.53	0.515	1/4	12.125	0.810	9 1/2	1 1/2	7/8	87	7.5	18.9	-	3880	586	740	118	5.38	241	39.7	3.07	132	60.4
x 79	23.2	12.38	0.470	1/4	12.080	0.735	9 1/2	1 7/16	7/8	79	8.2	20.7	-	3530	839	662	107	5.34	216	35.8	3.05	119	54.3
x 72	21.1	12.25	0.430	1/4	12.040	0.670	9 1/2	1 3/8	7/8	72	9.0	22.6	-	3230	1180	597	97.4	5.31	195	32.4	3.04	108	49.2
x 65	19.1	12.12	0.390	3/16	12.000	0.605	9 1/2	1 5/16	13/16	65	9.9	24.9	-	2940	1720	533	87.9	5.28	174	29.1	3.02	96.8	44.1
W 12 x 58	17.0	12.19	0.360	3/16	10.010	0.640	9 1/2	1 3/8	13/16	58	7.8	27.0	-	3070	1470	475	78.0	5.28	107	21.4	2.51	86.4	32.5
x 53	15.6	12.06	0.345	3/16	9.995	0.575	9 1/2	1 1/4	13/16	53	8.7	28.1	-	2820	2100	425	70.6	5.23	95.8	19.2	2.48	77.9	29.1
W 12 x 50	14.7	12.19	0.370	3/16	8.080	0.640	9 1/2	1 3/8	13/16	50	6.3	26.2	-	3170	1410	394	64.7	5.18	56.3	13.9	1.96	72.4	21.4
x 45	13.2	12.06	0.335	3/16	8.045	0.575	9 1/2	1 1/4	13/16	45	7.0	29.0	-	2870	2070	350	58.1	5.15	50.0	12.4	1.94	64.7	19.0
x 40	11.8	11.94	0.295	3/16	8.005	0.515	9 1/2	1 1/4	3/4	40	7.8	32.9	59	2580	3110	310	51.9	5.13	44.1	11.0	1.93	57.5	16.8
W 12 x 35	10.3	12.50	0.300	3/16	6.560	0.520	10 1/2	1	9/16	35	6.3	36.2	49	2420	4340	285	45.6	5.25	24.5	7.47	1.54	51.2	11.5
x 30	8.79	12.34	0.260	1/8	6.520	0.440	10 1/2	15/16	1/2	30	7.4	41.8	37	2090	7950	238	38.6	5.21	20.3	6.24	1.52	43.1	9.56
x 26	7.65	12.22	0.230	1/8	6.490	0.380	10 1/2	7/8	1/2	26	8.5	47.2	29	1820	13900	204	33.4	5.17	17.3	5.34	1.51	37.2	8.17
W 12 x22	6.48	12.31	0.260	1/8	4.030	0.425	10 1/2	7/8	1/2	22	4.7	41.8	37	2160	8640	156	25.4	4.91	4.66	2.31	0.847	29.3	3.66
x 19	5.57	12.16	0.235	1/8	4.005	0.350	10 1/2	13/16	1/2	19	5.7	46.2	30	1880	15600	130	21.3	4.82	3.76	1.88	0.822	24.7	2.98
x 16	4.71	11.99	0.220	1/8	3.990	0.265	10 1/2	3/4	1/2	16	7.5	49.4	26	1610	32000	103	17.1	4.67	2.82	1.41	0.773	20.1	2.26
x 14	4.16	11.91	0.200	1/8	3.970	0.225	10 1/2	11/16	1/2	14	8.8	54.3	22	1450	49300	88.6	14.9	4.62	2.36	1.19	0.753	17.4	1.90

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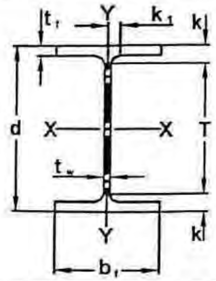

Designation	Area A	Depth D	Web Thickness t_w	$t_w/2$	Flange Width b_f	Flange Thickness t_f	Distance T	Distance k	Distance k_1	Nominal Weight per foot	Compact Section Criteria $b_f/2t_f$	h/t_w	F_y'''	X_1	$X_2 \times 10^6$	Axis X-X I	Axis X-X S	Axis X-X r	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r	Plastic Modulus Z_x	Z_y
	in^2	in	in	in	in	in	in	in	in	lbs			ksi	ksi	$(1/ksi)^2$	in^4	in^3	in	in^4	in^3	in	in^3	in^3
W 10 x 112	32.9	11.36	0.755	3/8	10.415	1.250	7 5/8	1 7/8	15/16	112	4.2	10.4	-	7080	56.7	716	126	4.66	236	45.3	2.68	147	69.2
x 100	29.4	11.10	0.680	3/8	10.340	1.120	7 5/8	1 3/4	7/8	100	4.6	11.6	-	6400	83.8	623	112	4.60	207	40.0	2.65	130	61.0
x 88	25.9	10.84	0.605	5/16	10.265	0.990	7 5/8	1 5/8	13/16	88	5.2	13.0	-	5680	132	534	98.5	4.54	179	34.8	2.63	113	53.1
x 77	22.6	10.60	0.530	1/4	10.190	0.870	7 5/8	1 1/2	13/16	77	5.9	14.8	-	5010	213	455	85.9	4.49	154	30.1	2.60	97.6	45.9
x 68	20.0	10.40	0.470	1/4	10.130	0.770	7 5/8	1 3/8	3/4	68	6.6	16.7	-	4460	334	394	75.7	4.44	134	26.4	2.59	85.3	40.1
x 60	17.6	10.22	0.420	1/4	10.080	0.680	7 5/8	1 5/16	3/4	60	7.4	18.7	-	3970	525	341	66.7	4.39	116	23.0	2.57	74.6	35.0
x 54	15.8	10.09	0.370	3/16	10.030	0.615	7 5/8	1 1/4	11/16	54	8.2	21.2	-	3580	778	303	60.0	4.37	103	20.6	2.56	66.6	31.3
x 49	14.4	9.98	0.340	3/16	10.000	0.560	7 5/8	1 3/16	11/16	49	8.9	23.1	-	3280	1090	272	54.6	4.35	93.4	18.7	2.54	60.4	28.3
W 10 x 45	13.3	10.10	0.350	3/16	8.020	0.620	7 5/8	1 1/4	11/16	45	6.5	22.5	-	3650	758	248	49.1	4.32	53.4	13.3	2.01	54.9	20.3
x 39	11.5	9.92	0.315	3/16	7.985	0.530	7 5/8	1 1/8	11/16	39	7.5	25.0	-	3190	1300	209	42.1	4.27	45.0	11.3	1.98	46.8	17.2
x 33	9.71	9.73	0.290	3/16	7.960	0.435	7 5/8	1 1/16	11/16	33	9.1	27.1	-	2710	2510	170	35.0	4.19	36.6	9.20	1.94	38.8	14.0
W 10x 30	8.84	10.47	0.300	3/16	5.810	0.510	8 5/8	15/16	1/2	30	5.7	29.5	-	2890	2160	170	32.4	4.38	16.7	5.75	1.37	36.6	8.84
x 26	7.61	10.33	0.260	1/8	5.770	0.440	8 5/8	7/8	1/2	26	6.6	34.0	55	2500	3790	144	27.9	4.35	14.1	4.89	1.36	31.3	7.50
x 22	6.49	10.17	0.240	1/8	5.750	0.360	8 5/8	3/4	1/2	22	8.0	36.9	47	2150	7170	118	23.2	4.27	11.4	3.97	1.33	26.0	6.10
W 10 x 19	5.62	10.24	0.250	1/8	4.020	0.395	8 5/8	13/16	1/2	19	5.1	35.4	51	2420	5160	96.3	18.8	4.14	4.29	2.14	0.874	21.6	3.35
x 17	4.99	10.11	0.240	1/8	4.010	0.330	8 5/8	3/4	1/2	17	6.1	36.9	47	2210	7820	81.9	16.2	4.05	3.56	1.78	0.844	18.7	2.80
x 15	4.41	9.99	0.230	1/8	4.000	0.270	8 5/8	11/16	7/16	15	7.4	38.5	43	1930	14300	68.9	13.8	3.95	2.89	1.45	0.810	16.0	2.30
x 12	3.54	9.87	0.190	1/8	3.960	0.210	8 5/8	5/8	7/16	12	9.4	46.6	30	1550	35400	53.8	10.9	3.90	2.18	1.10	0.785	12.6	1.74
W 8 x 67	19.7	9.00	0.570	5/16	8.280	0.935	6 1/8	1 7/16	11/16	67	4.4	11.1	-	6620	73.9	272	60.4	3.72	88.6	21.4	2.12	70.2	32.7
x 58	17.1	8.75	0.510	1/4	8.220	0.810	6 1/8	1 5/16	11/16	58	5.1	12.4	-	5820	122	228	52.0	3.65	75.1	18.3	2.10	59.8	27.9
x 48	1431	8.50	0.400	3/16	8.110	0.685	6 1/8	1 3/16	5/8	48	5.9	15.8	-	4860	238	184	43.3	3.61	60.9	15.0	2.08	49.0	22.9
x 40	11.7	8.25	0.360	3/16	8.070	0.560	6 1/8	1 1/16	5/8	40	7.2	17.6	-	4080	474	146	35.5	3.53	49.1	12.2	2.04	39.8	18.5
x 35	10.3	8.12	0.310	3/16	8.020	0.495	6 1/8	1	9/16	35	8.1	20.4	-	3610	761	127	31.2	3.51	42.6	10.9	2.03	34.7	16.1
x 31	9.13	8.00	0.285	3/16	7.995	0.435	6 1/8	15/16	9/16	31	9.2	22.2	-	3230	1180	110	27.5	3.47	37.1	9.27	2.02	30.4	14.1
W 8 x 28	8.25	8.06	0.285	3/16	6.535	0.465	6 1/8	15/16	9/16	28	7.0	22.2	-	3480	931	98.0	24.3	3.45	21.7	6.63	1.62	27.2	10.1
x 24	7.08	7.93	0.245	1/8	6.495	0.400	6 1/8	7/8	9/16	24	8.1	25.8	-	3020	1610	82.8	20.9	3.42	18.3	5.63	1.61	23.2	8.57
W 8 x 21	6.16	8.28	0.250	1/8	5.270	0.400	6 5/8	13/16	1/2	21	6.6	27.5	-	2890	2090	75.3	18.2	3.49	9.77	3.71	1.26	20.4	5.69
x 18	5.26	8.14	0.230	1/8	5.250	0.330	6 5/8	3/4	7/16	18	8.0	29.9	-	2490	3890	61.9	15.2	3.43	7.97	3.04	1.23	17.0	4.66
W 8 x 15	4.44	8.11	0.245	1/8	4.015	0.315	6 5/8	3/4	1/2	15	6.4	28.1	-	2670	3440	48.0	11.8	3.29	3.41	1.70	0.876	13.6	2.67
x 13	3.84	7.99	0.230	1/8	4.000	0.255	6 5/8	11/16	7/16	13	7.8	29.9	-	2370	5780	39.6	9.91	3.21	2.73	1.37	0.843	11.4	2.15
x 10	2.96	7.89	0.170	1/8	3.940	0.205	6 5/8	5/8	7/16	10	9.6	40.5	39	1760	17900	30.8	7.81	3.22	2.09	1.06	0.841	8.87	1.66
W 6 x 25	7.34	6.38	0.320	3/16	6.080	0.455	4 3/4	13/16	7/16	25	6.7	15.5	-	4410	369	53.4	16.7	2.70	17.1	5.61	1.52	18.9	8.56
x 20	5.87	6.20	0.260	1/8	6.020	0.365	4 3/4	3/4	7/16	20	8.2	19.1	-	3550	846	41.4	13.4	2.66	13.3	4.41	1.50	14.9	6.72
x 15	4.43	5.99	0.230	1/8	5.990	0.260	4 3/4	5/8	3/8	15	11.5	21.6	-	2740	2470	29.1	9.72	2.56	9.32	3.11	1.46	10.8	4.75
W 6 x 16	4.74	6.28	0.260	1/8	4.030	0.405	4 3/4	3/4	7/16	16	5.0	19.1	-	4010	591	32.1	10.2	2.60	4.43	2.20	0.966	11.7	3.39
x 12	3.55	6.03	0.230	1/8	4.000	0.280	4 3/4	5/8	3/8	12	7.1	21.6	-	3100	1740	22.1	7.31	2.49	2.99	1.50	0.918	8.30	2.32
x 9	2.68	5.90	0.170	1/8	3.940	0.215	4 3/4	9/16	3/8	9	9.2	29.2	-	2360	4980	16.4	5.56	2.47	2.19	1.11	0.905	6.23	1.72
W 5 x 19	5.54	5.15	0.270	1/8	5.030	0.430	3 1/2	13/16	7/16	19	5.8	14.0	-	5140	192	26.2	10.2	2.17	9.13	3.63	1.28	11.6	5.53
x 16	4.68	5.01	0.240	1/8	5.000	0.360	3 1/2	3/4	7/16	16	6.9	15.8	-	4440	346	21.3	8.51	2.13	7.51	3.00	1.27	9.59	4.57
W 4 x 13	3.83	4.16	0.280	1/8	4.060	0.345	2 3/4	11/16	7/16	13	5.9	10.6	-	5560	154	11.3	5.46	1.72	3.86	1.90	1.00	6.28	2.92

Appendix –III

Cost Estimation

APPENDIX – III

Estimation of cost for different cut and cover tunneling methods :

(01) Bill of quantities and estimated cost for total route (20 km) of earth retention method using by Sheet Pile :

(a) For 77 ft upper Clay layer depth:

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Supplying, fabrication & fixing of Sheet Pile	Quintal	7359	661826	4870.38
3	Supplying, fabrication & fixing of Wales	Quintal	7359	8927	65.7
4	Supplying, fabrication & fixing of Tie Rods	Quintal	5400	9002	48.6
5	Drilling & Concrete Works for Tiebacks	Cum	7668	42	0.32
Total Cost for 77 ft upper Clay layer depth					4986.28

Cost per kilometer = 249.31 million Tk

(02) Bill of quantities and estimated cost for total route (20 km) of earth retention method using by Braced-Cut :

(a) For 20 ft upper Clay layer depth :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Supplying, fabrication & fixing of Sheet Piles	Quintal	7359	661653	4869.10
3	Supplying, fabrication & fixing of Wales	Quintal	7359	213610	1571.96
4	Supplying, fabrication & fixing of Struts	Quintal	7359	138362	1018.20
Total Cost for 20 ft upper Clay layer depth					7460.55

Cost per kilometer = 373.03 million Tk.

(b) For 77 ft upper Clay layer depth:

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Supplying, fabrication & fixing of Sheet Pile	Quintal	7359	661653	4869.10
3	Supplying, fabrication & fixing of Wales	Quintal	7359	206469	1519.40
4	Supplying, fabrication & fixing of Struts	Quintal	7359	128543	945.95
Total Cost for 77 ft upper Clay layer depth					7335.74

Cost per kilometer = 366.79 million Tk.

(c) For 65 ft upper Clay layer depth:

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Supplying, fabrication & fixing of Sheet Pile	Quintal	7359	661653	4869.10
3	Supplying, fabrication & fixing of Wales	Quintal	7359	188024	1383.67
4	Supplying, fabrication & fixing of Struts	Quintal	7359	115153	847.41
Total Cost for 65 ft upper Soft Clay layer					7101.46

Cost per kilometer = 355.07 million Tk.

(d) Building Structures beside the Route:

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Supplying, fabrication & fixing of Sheet Pile	Quintal	7359	1009735	7430.64
3	Supplying, fabrication & fixing of Wales	Quintal	7359	357602	2631.59
4	Supplying, fabrication & fixing of Struts	Quintal	7359	220486	1622.56
Total Cost for Building Structures beside the Route					11686.07

Cost per kilometer = 584.30 million Tk.

(03) Bill of quantities and estimated cost for total route (20 km) of earth retention method using by Bored Pile :

(a) For 20 ft upper Clay layer depth :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Boring for cast-in-situ pile	Rm	513	1333364	684.01
3	Cast-in-situ pile with R.C.C. works	Cum	7668	173052	1326.96
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	320666	1731.59
5	Fabrication of 60 grade 10 mm deformed bar	Quintal	5400	228686	1234.90
6	Providing and making Point Welding	Point	2.38	210452820	500.87
7	Providing and making Welded Splice	Inch	3.57	9446760	33.72
8	Supplying, fabrication & fixing of Wales	Quintal	7359	213610	1571.95
9	Supplying, fabrication & fixing of Struts	Quintal	7359	138362	1018.20
Total Cost for 20 ft upper Clay layer depth					8103.52

Cost per kilometer = 405.17 million Tk.

(b) For 77 ft upper Clay layer depth :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.280489
2	Boring for cast-in-situ pile	Rm	513	1333364	684.015732
3	Cast-in-situ pile with R.C.C. works	Cum	7668	173052	1326.962736
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	320666	1731.5964
5	Fabrication of 60 grade 10mm deformed bar	Quintal	5400	228686	1234.9044
6	Providing and making Point Welding	Point	2.38	210452820	500.8777116
7	Providing and making Welded Splice	Inch	3.57	9446760	33.7249332
8	Supplying, fabrication & fixing of Wales	Quintal	7359	206469	1519.405371
9	Supplying, fabrication & fixing of Struts	Quintal	7359	128543	945.947937
Total Cost for 77 ft upper Clay layer depth					7978.72

Cost per kilometer = 398.94 million Tk.

(c) For 65 ft upper Clay layer depth :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.280489
2	Boring for cast-in-situ pile	Rm	513	1333364	684.015732
3	Cast-in-situ pile with R.C.C. works	Cum	7668	173052	1326.962736
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	320666	1731.5964
5	Fabrication of 60 grade 10mm deformed bar	Quintal	5400	228686	1234.9044
6	Providing and making Point Welding	Point	2.38	210452820	500.8777116
7	Providing and making Welded Splice	Inch	3.57	9446760	33.7249332
8	Supplying, fabrication & fixing of Wales	Quintal	7359	188024	1383.668616
9	Supplying, fabrication & fixing of Struts	Quintal	7359	115153	847.410927
Total Cost for 65 ft upper Clay layer depth					7744.44

Cost per kilometer = 387.22 million Tk.

(d) Building Structures beside the Route :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Boring for cast-in-situ pile	Rm	661	1000031	661.02
3	Cast-in-situ pile with R.C.C. works	Cum	7668	245380	1881.57
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	400829	2164.47
5	Fabrication of 60 grade 12mm deformed bar	Quintal	5400	248569	1342.27
6	Providing and making Point Welding	Point	2.38	197462020	469.96
7	Providing and making Welded Splice	Inch	3.57	11808360	42.16
8	Supplying, fabrication & fixing of Wales	Quintal	7359	357602	2631.59
9	Supplying, fabrication & fixing of Struts	Quintal	7359	220486	1622.56
Total Cost for Building Structures beside the Route					10816.89

Cost per kilometer = 540.84 million Tk.

(04) Bill of quantities and estimated cost for total route (20 km) of earth retention method using by Diaphragm Wall :

(a) For 20 ft upper Clay layer depth:

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Boring for cast-in Diaphragm Wall	Rm	441	1503796	663.17
3	Cast-in-Diaphragm Wall with R.C.C. works	Cum	7668	185902	1425.49
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	304621	1644.95
5	Fabrication of 60 grade 10mm deformed bar	Quintal	5400	27647	149.29
6	Providing and making Point Welding	Point	2.38	25426560	60.51
7	Providing and making Welded Splice	Inch	3.57	8974080	32.04
8	Supplying, fabrication & fixing of Wales	Quintal	7359	213610	1571.96
9	Supplying, fabrication & fixing of Struts	Quintal	7359	138362	1018.20
Total Cost for 20 ft upper Clay layer depth					6566.91

Cost per kilometer = 328.34 million Tk.

(b) For 77 ft upper Clay layer depth:

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Boring for cast-in Diaphragm Wall	Rm	441	1503796	663.17
3	Cast-in-Diaphragm Wall with R.C.C. works	Cum	7668	185902	1425.49
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	304621	1644.95
5	Fabrication of 60 grade 10mm deformed bar	Quintal	5400	27647	149.29
6	Providing and making Point Welding	Point	2.38	25426560	60.51
7	Providing and making Welded Splice	Inch	3.57	8974080	32.04
8	Supplying, fabrication & fixing of Wales	Quintal	7359	206469	1519.40
9	Supplying, fabrication & fixing of Struts	Quintal	7359	128543	945.94
Total Cost for 77 ft upper Clay layer depth					6442.10

Cost per kilometer = 322.10 million Tk.

(c) For 65 ft upper Soft Clay layer depth:

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Boring for cast-in Diaphragm Wall	Rm	441	1503796	663.17
3	Cast-in-Diaphragm Wall with R.C.C. works	Cum	7668	170410	1306.70
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	272555	1471.79
5	Fabrication of 60 grade 10mm deformed bar	Quintal	5400	30127	162.68
6	Providing and making Point Welding	Point	2.38	24980480	59.45
7	Providing and making Welded Splice	Inch	3.57	8029440	28.66
8	Supplying, fabrication & fixing of Wales	Quintal	7359	188024	1383.66
9	Supplying, fabrication & fixing of Struts	Quintal	7359	115153	847.41
Total Cost for 65 ft upper Clay layer depth					5924.84

Cost per kilometer = 296.24 million Tk.

(d) Building Structures beside the Route :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill. Tk.)
1	Layout and marking	Sqm	7	182927	1.28
2	Boring for cast-in Diaphragm Wall	Rm	441	1503796	663.17
3	Cast-in-Diaphragm Wall with R.C.C. works	Cum	7668	247869	1900.65
4	Fabrication of 60 grade 25mm deformed bar	Quintal	5400	400816	2164.40
5	Fabrication of 60 grade 10mm deformed bar	Quintal	5400	22103	119.35
6	Providing and making Point Welding	Point	2.38	25584000	60.88
7	Providing and making Welded Splice	Inch	3.57	11808000	42.15
8	Supplying, fabrication & fixing of Wales	Quintal	7359	357602	2631.59
9	Supplying, fabrication & fixing of Struts	Quintal	7359	220486	1622.55
Total Cost for Building Structures beside the Route					9206.07

Cost per kilometer = 460.30 million Tk.

Bill of quanitites and estimated cost of Tunnel Box for total route (20km) construction of Metro-Rail Tunnel in Dhaka City :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill.Taka)
1	Earthwork in Excavation	Cum	32	2788519	89.24
2	Extra rate for exceeding 1.5 m depth	Cum	7	2514129	17.6
3	Extra rate for exceeding 10 m depth	Cum	1.06	958786	1.02
4	Extra rate for protected by palisading and de-watering	Cum	244	2788519	680.4
5	Extra rate for de-watering exceeding 1.5 m depth	Cum	425	2514129	1068.51
6	Sand filling (18″) by Coarse Sand (F.M. 1.2)	Cum	652	1003867	654.53
7	3″ Soling	Sqm	162	182927	29.64
8	Supplying and laying of single layer polythene sheet	Sqm	22	585366	12.88
9	3″ thick damp proof course (1:1.5:3)	Sqm	676	182927	123.66
10	R.C.C. works (1:1.5:3) with Stone chips				
	(i) In Floor Slab Concrete	Cum	6126	139426	854.13
	(ii) In Roof Slab Concrete	Cum	6126	139426	854.13
	(iii) In Side Wall Concrete	Cum	6195	96669	598.87
	(iv) In Middle Column Concrete	Cum	6195	9667	59.89
	(v) In Wall and Column Capital Concrete	Cum	6195	29745	184.28
11	Shuttering, prop and necessary supports				
	(i) In Roof Slab Shuttering	Sqm	488	182927	89.27
	(ii) In Side Wall Shuttering	Sqm	319	158537	50.58
	(iii) In Middle Column Shuttering	Sqm	319	18000	5.75
	(iv) In Wall and Column Capital Shuttering	Sqm	319	60976	19.46
12	(i) Fabrication of 60 grade 25mm deformed bar	Quintal	5400	255726	1380.93
	(ii) Fabrication of 60 grade 16mm deformed bar	Quintal	5400	68309	368.87
	(iii) Fabrication of 60 grade 12mm deformed bar	Quintal	5400	25729	138.94
	(iv) Fabrication of 60 grade 10mm deformed bar	Quintal	5400	35385	191.08
13	Minimum 6 mm thick cement plaster (1:4)	Sqm	99	640927	63.46
14	Plastic emulsion paint to Wall and ceiling	Sqm	89	496342	44.18
	Total Civil Cost for Construction				7581.3
15	Electrification Cost	Sqm	967	182927	176.9
16	Ventilation Cost (4 % of Civil Cost)				303.252
	Total Cost for Construction of Tunnel Box				8061.452

Bill of quanitites and estimated cost of Metro R.S. in Dhaka City :

Sl. No.	Description of Items	Unit	Unit Rate	Quantity	Total Cost (mill.Taka)
1	Layout and Marking	Sqm	7	1700	0.01
2	Shore protection work	Sqm	579	5300	3.07
3	Earthwork in Excavation	Cum	32	25650	0.82
4	Extra rate for exceeding 1.5 m depth	Cum	7	23150	0.16
5	Extra rate for exceeding 10 m depth	Cum	1.06	9100	0.01
6	Extra rate for protected by palisading and de-watering	Cum	244	25650	6.26
7	Extra rate for de-watering exceeding 1.5 m depth	Cum	425	23150	9.84
8	Sand filling (18″) by Coarse Sand (F.M. 1.2)	Cum	652	700	0.46
9	3″ Soling	Sqm	162	1530	0.25
10	Supplying and laying of single layer polythene sheet	Sqm	22	1530	0.03
11	3″ thick damp proof course (1:1.5:3)	Sqm	676	1530	1.03
12	R.C.C. works (1:1.5:3) with Stone chips				
	(i) In Ground Floor Concrete	Cum	6126	1615	9.89
	(ii) In Slab (Roof, 2nd and 1st Floor) Concrete	Cum	6126	927	5.67
	(iii) Extra Rate in Slab (Roof, 2nd and 1st Floor) Concrete	Cum	94	927	0.09
	(iii) In Side Wall Concrete	Cum	6195	1274	7.89
	(iv) Extra Rate in Side Wall (2nd and 1st Floor) Concrete	Cum	92	1068	0.09
	(v) In Column (2nd,1st G.F. Floor) Concrete	Cum	6195	101	0.63
	(vi) Extra Rate in Columns (2nd and 1st and G.F.) Con.	Cum	92	86	0.01
	(vii) In Beam (Roof, 2nd and 1st Floor) Concrete	Cum	6126	1214	7.44
	(viii) Extra Rate in Beams (Roof, 2nd and 1st Floor) Con.	Cum	94	910	0.09
	(ix) In Staircase (2nd,1st G.F. Floor) Concrete	Cum	6309	27	0.17
	(x) Extra Rate in Staircase (2nd,1st and G.F.) Concrete	Cum	94	21	0.002
	(xi) In Lintle (2nd Floor) Concrete	Cum	6101	2.5	0.015
	(xii) Extra Rate in Lintle (2nd Floor) Concrete	Cum	94	2.5	0.0002
13	Shuttering, prop and necessary supports				
	(i) In Ground Floor Shuttering	Sqm	306	1362	0.42
	(ii) In Slab (Roof, 2nd and 1st Floor) Shuttering	Sqm	488	2438	1.19
	(iii) Extra Rate in Slab (Roof, 2nd and 1st Floor) Shutt.	Sqm	23	2022	0.05

	(iv) In Side Wall Shuttering	Sqm	319	2183	0.70
	(v) Extra Rate in Side Wall (2nd and 1st Floor) Shutt.	Sqm	16	1719	0.03
	(vi) In Column (2nd,1st G.F. Floor) Shuttering	Sqm	319	539	0.17
	(vii) Extra Rate in Columns (2nd and 1st and G.F.) Shutt.	Sqm	16	472	0.007
	(viii) In Beam (Roof, 2nd and 1st Floor) Shuttering	Sqm	386	500	0.19
	(ix) Extra Rate in Beams (Roof, 2nd and 1st Floor) Shutt.	Sqm	39	375	0.015
	(x) In Staircase (2nd,1st G.F. Floor) Shuttering	Sqm	390	100	0.04
	(xi) Extra Rate in Staircase (2nd,1st and G.F.) Shuttering	Sqm	39	75	0.003
14	(i) Fabrication of 60 grade 25mm deformed bar	Quintal	5400	2177	11.76
15	(ii) Fabrication of 60 grade 16mm deformed bar	Quintal	5400	1141	6.16
16	(iii) Fabrication of 60 grade 12mm deformed bar	Quintal	5400	302	1.63
17	(iv) Fabrication of 60 grade 10mm deformed bar	Quintal	5400	499	2.69
18	125 mm thick Brick Wall	Sqm	423	13005	5.50
19	Minimum 12 mm thick cement plaster (1:6)	Sqm	109	305	0.03
20	Minimum 6 mm thick cement plaster (1:4)	Sqm	99	6718	0.66
21	10 mm thick color situ mosaic	Sqm	921	2884	2.65
22	Plastic emulsion paint to inner Wall and ceiling	Sqm	89	5170	0.46
23	Cement paint to outer Wall	Sqm	62	305	0.02
24	Supplying, fitting and fixing Stair railing	Sqm	3710	35	0.13
25	Supplying, fitting and fixing Collapsible Gate 2 Nos	Sqm	2892	13	0.038
26	Supplying, fitting and fixing window grills	Sqm	1167	168	0.20
27	Supplying, fitting and fixing Steel glazed window Shutter	Sqm	3297	168	0.55
28	Supplying, fitting and fixing M.S. Door Shutter	Sqm	3400	65	0.22
29	Painting to Door and window Shutter	Sqm	101	110	0.01
30	Providing drip course	Rm	47	116	0.005
31	Roof top parapet	Sqm	953	92	0.09
	Total Civil Cost for Construction				89.56
32	Cost of Lighting and Escalators (From Kolkata Metro)	Per Station			30
33	Air Conditioning and Ventilation Cost (From Kolkata Met.)	Per Station			64.4
	Total Cost for Construction of Metro Railway Station				183.96

When it passing the 21 Nos. Station Building then total Construction cost of Tunnel Box becomes (8061315974 -112368508) = 7948.95 million Tk.

Cost of Tunnel Box per km = 397.45 million Tk.

(01) If Bored pile is used, then total cost for construction of Tunnel Box is (7948.95-1453.89) = 6495.06 million Tk. Cost of Tunnel Box per km = 324.8 million Tk.

(02) If Dia. Wall is used, then Total cost for construction of Tunnel Box is (7948.95-1374.95) = 6574 million Tk. Cost of Tunnel Box per km = 328.7 million Tk.

Total construction cost for 20 km Tunnel Box including Diaphragm wall and Underground Station Building is (9206.07+6574+21x183.96) = 19643.23 million Tk. ≈ 19644 million. Cost per kilometer = (19644/20) = 982.2 million Tk.≈ 983 million Taka.