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AND-COVER TUNNELING

Vol. 1. Construction Methods, Design, and Activity Variations

G. E. Wickham and H. R. Tiedemann

May 1976
Final Report

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16. Abstract

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

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

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

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PREFACE

The results, conclusions and recommendations of a study to optimize cut-and-cover tunneling operations are contained in this two volume report. The study has been conducted by Jacobs Associates in accordance with the terms of Contract No. D.O.T.-FH-11-8513, dated June 28, 1974, with the Department of Transportation. The Contracting Officer is Mr. H. G. Gale; the Project Manager is Mr. J. R. Sallberg.

Volume I of the report covers Task A of the contract requirements to develop a method for analyzing the efficiency of cut-and-cover tunnel construction systems. Volume II contains the results of an extensive series of comparative multiple estimates, a suggested method for quantifying construction disruption, and a comparison of traditional and under-the-roof construction sequences.

The major portion of the work has been conducted within the Jacobs Associates organization. Information for the study has been drawn from job files of cut-and-cover projects maintained by Jacobs Associates as well as research of previously published texts, reports and papers. The report is the result of the effort of many people of various disciplines including design, estimating, research, cost accounting, and construction. J. L. Wilton of Jacobs Associates wrote Section 4 of Volume 1, Selection and Application of Design Guidelines.

Moretrench American Corporation acting as consultants advised on methods and procedures of dewatering and prepared estimates of dewatering costs for varying conditions. In addition, the authors would like to express their appreciation to the many organizations and construction companies that gave so generously of their time, information and advice.

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INTRODUCTION

It has become increasingly necessary in urban areas to utilize the space above and below the surface as well as the area at ground level in order to serve man's needs. The more concentrated the population becomes the more necessary it is to build expensive structures such as elevated highways and tunnels to handle traffic that can no longer be accommodated on the ground. As the conflict for space becomes critical, it is man who must occupy the space at ground level and transportation and other utilities that serve man relegated to areas below the surface. While it has been common practice to place water, gas and electric service below the surface, only a few of the larger, more congested cities have moved transportation facilities below ground.

In 1965 San Francisco started construction on the first new underground rapid transit system in the United States in fifty years. This was followed a few years later by a similar system in Washington, D.C. Others are being designed for Atlanta, Baltimore and Los Angeles. In addition to rapid transit, many cities are considering increased use of depressed or tunneled highways.

Where possible, economy dictates that these transportation facilities are placed as near the surface as possible and are constructed by cut-and-cover techniques rather than by tunneling methods. In addition to construction economics, shallow cut-and-cover tunnels have several other advantages such as easy access from street level. In addition, except under conditions of exceptionally competent rock, ground support problems make construction of large span openings, such as those required for three lanes of traffic, extremely difficult using tunneling methods. These are comparatively simple to construct in open cut.

The main disadvantages of a cut-and-cover tunnel are the highly disruptive effects of such operations in congested urban areas. Problems of construction include relocating or maintaining existing structures and utilities and maintaining access to the area for pedestrians, emergency vehicles and at least limited traffic.

The purpose of this study is to address itself to these and related problems in order to develop a method for optimizing cut-and-cover tunneling operations for the varied practical considerations of present and future construction. While each individual contractor attempts to optimize his operations for an instant project, he can only do so within the confines of the pre-determined physical size and location of the project and the time frame allowed for completion of the work. The construction is usually scheduled to begin within days or weeks of the award of contract. The individual designer likewise is confined by pre-determined criteria, and while he has more time before construction to consider alternatives, he is usually limited to design of the completed structure in order to guarantee the contractor the flexibility of choosing construction methods suitable to his organization, equipment and past experience.

It is the aim of this study to provide these men with a model of varying site and construction conditions which are used to determine the most advantageous methods of performing each construction activity for optimum efficiency of the entire design-construct operation.

SUMMARY

Cut-and-cover tunneling has, in common with all of man's other endeavors, certain limitations, advantages, drawbacks and efficiencies. Although it has been used and improved for almost one hundred years, there has been relatively little comprehensive study of the overall design-construct process to use as a guide for future methods of improvement. The current program of the Department of Transportation is aimed at improving techniques for several important aspects of cut-and-cover construction: ground support, underpinning and grouting. This portion of the program is devoted to providing an overview of the design-construct process by investigating the various activities involved, and the methods by which they can be combined for optimum results.

In order that the results of this study be applicable to a wide range of alternate situations, various site conditions and construction methods have been specified. The study considers two urban sites with low and high water table with adjacent structures supported on alluvial soil by spread footings. Three types of ground support systems are included, soldier piles and lagging, cast-in-place concrete walls and precast concrete panel walls. The construction methods used must be applicable to three types of structures: highway tunnels, rapid transit line tunnels and rapid transit stations. Each of these structures is considered for three excavated depths of 30 feet (9.1 m), 50 feet (15.2 m) and 70 feet (21.3 m).

The construction process is divided into eleven major activities common to all conditions. Each activity is discussed with respect to alternate methods of performance, efficiencies and drawbacks, and compatibility with other activities in the overall construction systems. For each activity, factors that must be considered for a choice between alternate methods are reviewed, including limitations imposed by the type of structure. Methods that are particularly appropriate, or conversely inappropriate for urban site

conditions are commented on. All discussions are oriented toward reasonable solutions or needed improvement applicable to achievement of maximum efficiency.

A subjective method of evaluation of alternative activity methods is presented for sample activities. While not a quantitative approach, it is possible, using a reasonable set of factors to present a qualitative evaluation of alternative solutions. Where one method obviously rates higher than the others there is no problem. Likewise, if two methods of similar costs rate about the same, using either will not significantly affect the overall project costs. This evaluation process is used to reduce the number of variables to be considered to a manageable number.

Design criteria used for permanent structures, ground support, bracing and decking is discussed. A set of composite design guidelines representing the best features of current design practice is presented and applied to design the several options of ground support and bracing required by the specified site conditions. A background description of cut-and-cover design practice is given. Use of a common practical set of design requirements assures a degree of uniformity not found in projects constructed in different areas and helps to assure that various estimated costs are not affected by the design mode being subjected to individual interpretation. The same set of design guidelines has been used for all conditions.

The results of site conditions, construction methods, activity alternates and design are summarized in a table presenting a unique analytical expression for each of 96 possible sets of conditions. Methods of estimating total construction costs through the use of proprietary computer estimating program are discussed. Three basic estimates, encompassing a maximum number of variables are described and illustrated through the use of analytical activity-condition descriptions and graphic activity precedence networks. Methods of varying the basic estimates to provide the total number of required cost studies are reviewed.

SECTION I

1.0 STUDY CRITERIA

The objective of the present research effort is to develop a flexible analytical method for evaluating and optimizing the entire cut-and-cover tunnel design-construction operation. Since the subject involves the consideration of unlimited possibilities, conditions and situations, it is apparent that certain limits and assumptions must be established to arrive at meaningful results. This is not to say that the overall effectiveness of the developed methodology should be compromised, but rather to indicate the need for an approach which is adaptable to practical usage.

The intent is to consider, compare and evaluate all factors and subsequently delineate and optimize those which would affect significantly a particular cut-and-cover tunneling situation. Each situation being defined by: 1) site conditions; 2) type of structure, and 3) method of construction. Since results must be flexible enough to accommodate the inclusion of many different variables it will be necessary to make certain generalizations in evaluating some of the factors. All assumptions will be defined so that individual projects can be compared, and adjustments made for those details which differ significantly from typical situations used in this study. Although this iterim report deals primarily with construction cost, time and design requirements, the format will be such that environmental considerations can be included at a later time. A portion of the next phase of this study will deal specifically with disruption caused by construction.

The stipulated general study criteria are discussed in the following sections.

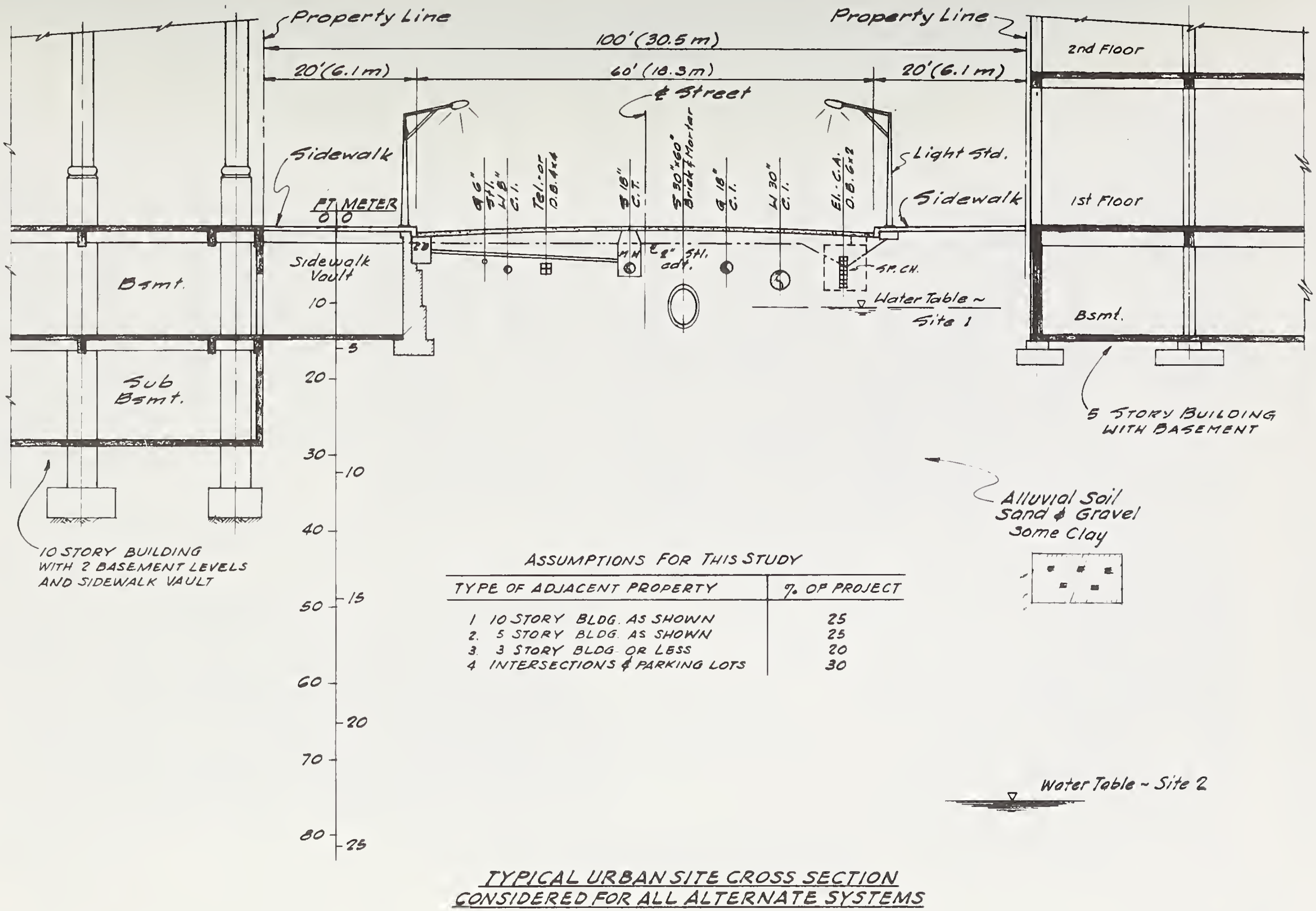
1.1 URBAN SITES

Cut-and-cover tunneling techniques can be used for construction in a variety of locations. The most complex and critical occur in an urban environment. Most urban sites, in addition to normal technical problems of ground support and construction, also require traffic control, utility maintenance, and protection of nearby structures be considered in project planning. By defining this urban site we will be in a position to compare other sites which vary in particulars, or eliminate entirely those activities not required for a project in a less complex environment.

Two urban sites have been suggested for this study, varying only in respect to groundwater conditions. The first site has a high water table meaning that each tunneling situation considered at this site must include the problem of handling groundwater. The second site is similar in all other respects except that the water table is low enough to preclude the necessity of dewatering. These sites typify a fairly high density, downtown commercial district with buildings supported on spread footings on alluvial soils. A cross section of these sites is shown in Figure 1. This cross section gives additional details that are typical of such an area. Two adjacent buildings are shown, one a five story building with a single basement level, the other a ten story building with two basement levels and a sidewalk vault. A percentage of the site will be considered occupied by each of these types and by some smaller buildings. The street width shown is typical of many downtown areas; a wider street would oversimplify the traffic and decking problems. The utilities shown have been chosen to represent a maximum variety, requiring differing solutions of maintenance, support or relocation. By using these same conditions for all ground support and depth variables it will be possible to judge their effect on the construction process.

1.2 TYPE OF STRUCTURE

The optimization study is to be applicable to three types of transportation structures: 1) a highway tunnel, 2) a rapid transit tunnel and 3) a rapid transit station. Typical cross sections of these structures shown at the urban site are given



ASSUMPTIONS FOR THIS STUDY

TYPE OF ADJACENT PROPERTY	% OF PROJECT
1 10 STORY BLDG. AS SHOWN	25
2. 5 STORY BLDG. AS SHOWN	25
3. 3 STORY BLDG. OR LESS	20
4 INTERSECTIONS & PARKING LOTS	30

Figure 1

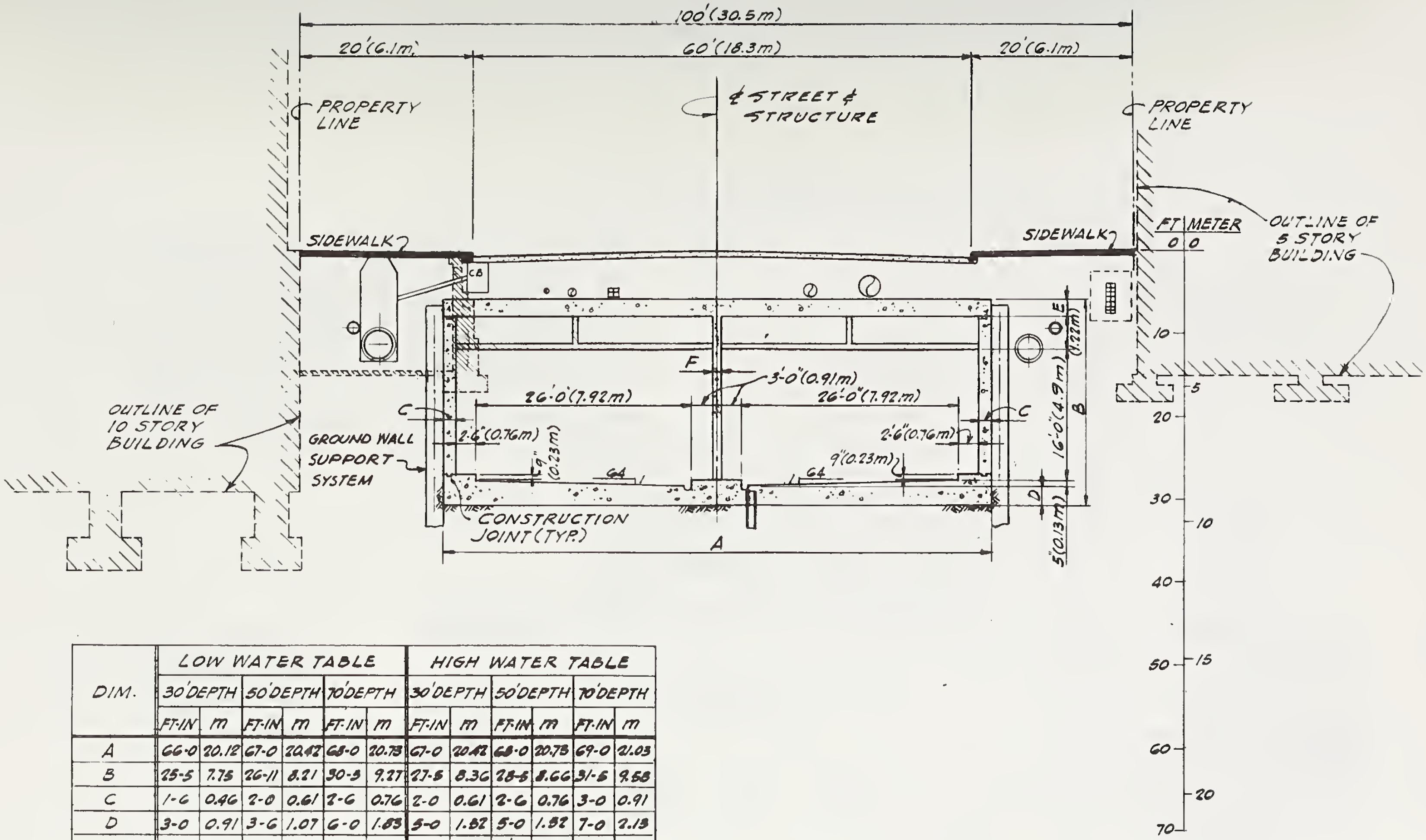
in Figures 2, 3, and 4, each shown at a different depth. Variation in depth below the surface is considered in later discussions.

The wall and slab thickness vary for variations of depth and water pressure. The sections underwater have been checked for uplift, which in some cases proved to be the controlling factor in determining concrete thickness.

For this study the typical sections will be used to develop comparative costs of the basic structural shell including allowances for buried electrical conduit and mechanical ducts appropriate for each structure. Unusual sections, structure end or transition sections, and appendage structures, such as shafts and entrances will be treated in a separate section of the final report. Structure finish, track work, electrification, fans, escalators and other equipment are usually supplied and installed separately after completion of the shell. Since these costs would not vary for any particular structure because of depth, groundwater or ground support considerations, they need not be included in this study.

1.2.1 Highway Tunnel - The highway tunnel shown in Figure 2 consists of a reinforced concrete box structure of four traffic lanes with two lanes on either side of a center wall. A divided plenum chamber above the roadway is sufficiently large for forced air intake and exhaust as well as utility ducts and conduits. Lane widths and vertical clearances are consistent with current highway tunnel standards. For the purpose of this study a 2000 foot length of highway tunnel will be considered as a separate construction contract. Although most direct costs could be computed on any convenient length of tunnel and converted to a cost per linear foot, it is necessary to consider a typical size of contract for determining contract completion time by C.P.M. methods. This contract duration will be used for computing time oriented plant and overhead costs.

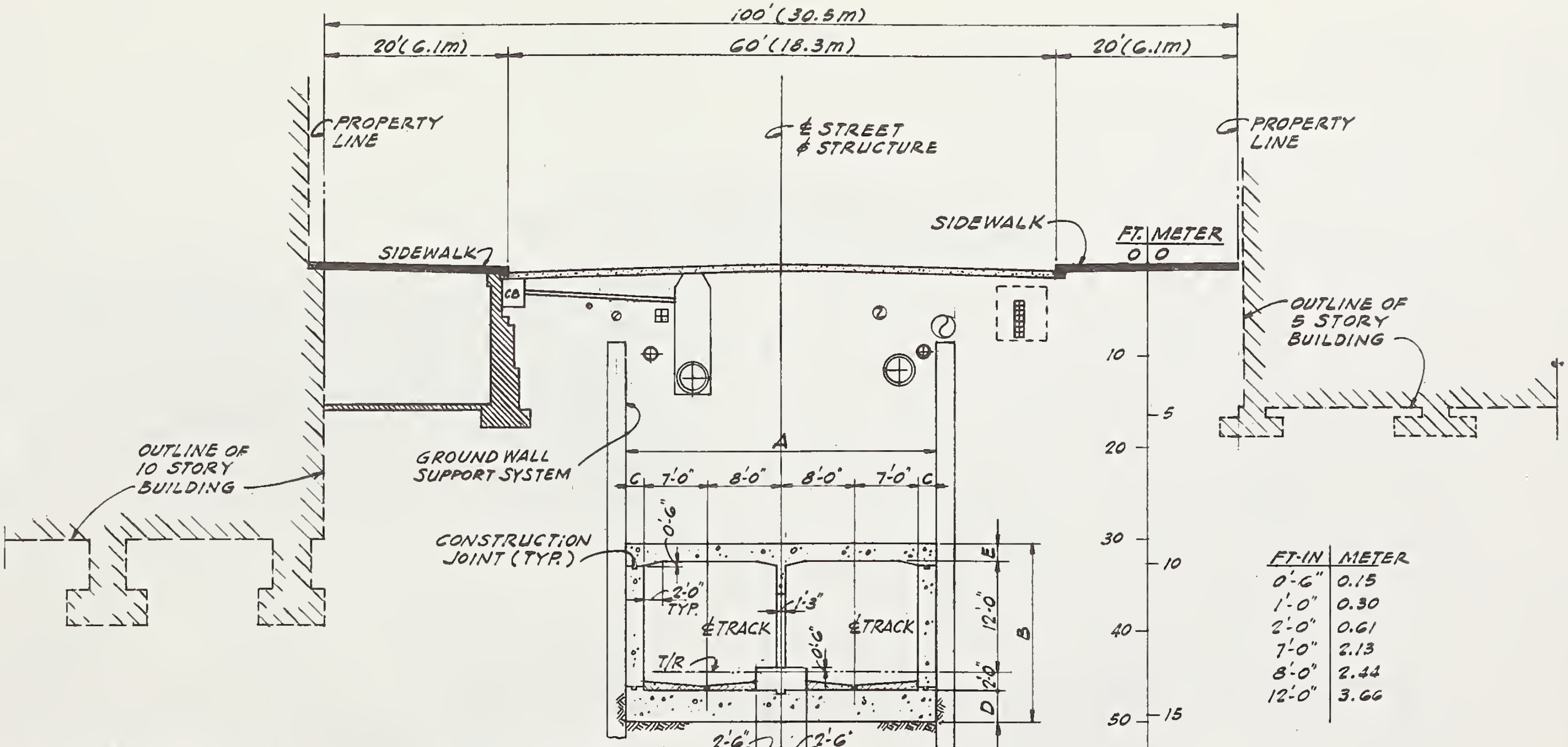
1.2.2 Rapid Transit Line Tunnel - The tunnel section shown in Figure 3 consists of a twin tube reinforced concrete box section. The inside dimensions are similar to those used on



DIM.	LOW WATER TABLE						HIGH WATER TABLE					
	30' DEPTH		50' DEPTH		70' DEPTH		30' DEPTH		50' DEPTH		70' DEPTH	
	FT-IN	m	FT-IN	m	FT-IN	m	FT-IN	m	FT-IN	m	FT-IN	m
A	66-0	20.12	67-0	20.42	68-0	20.73	67-0	20.42	68-0	20.73	69-0	21.03
B	25-5	7.75	26-11	8.21	30-5	9.27	27-5	8.36	28-5	8.66	31-5	9.58
C	1-6	0.46	2-0	0.61	2-6	0.76	2-0	0.61	2-6	0.76	3-0	0.91
D	3-0	0.91	3-6	1.07	6-0	1.83	5-0	1.52	5-0	1.52	7-0	2.13
E	2-0	0.61	3-0	0.91	4-0	1.22	2-0	0.61	3-0	0.91	4-0	1.22
F	1-3	0.38	1-6	0.46	2-0	0.61	1-3	0.38	1-6	0.46	2-0	0.61

DETAILS OF FOUR-LANE HIGHWAY TUNNEL
(SHOWN AT 30' DEPTH)

Figure 2

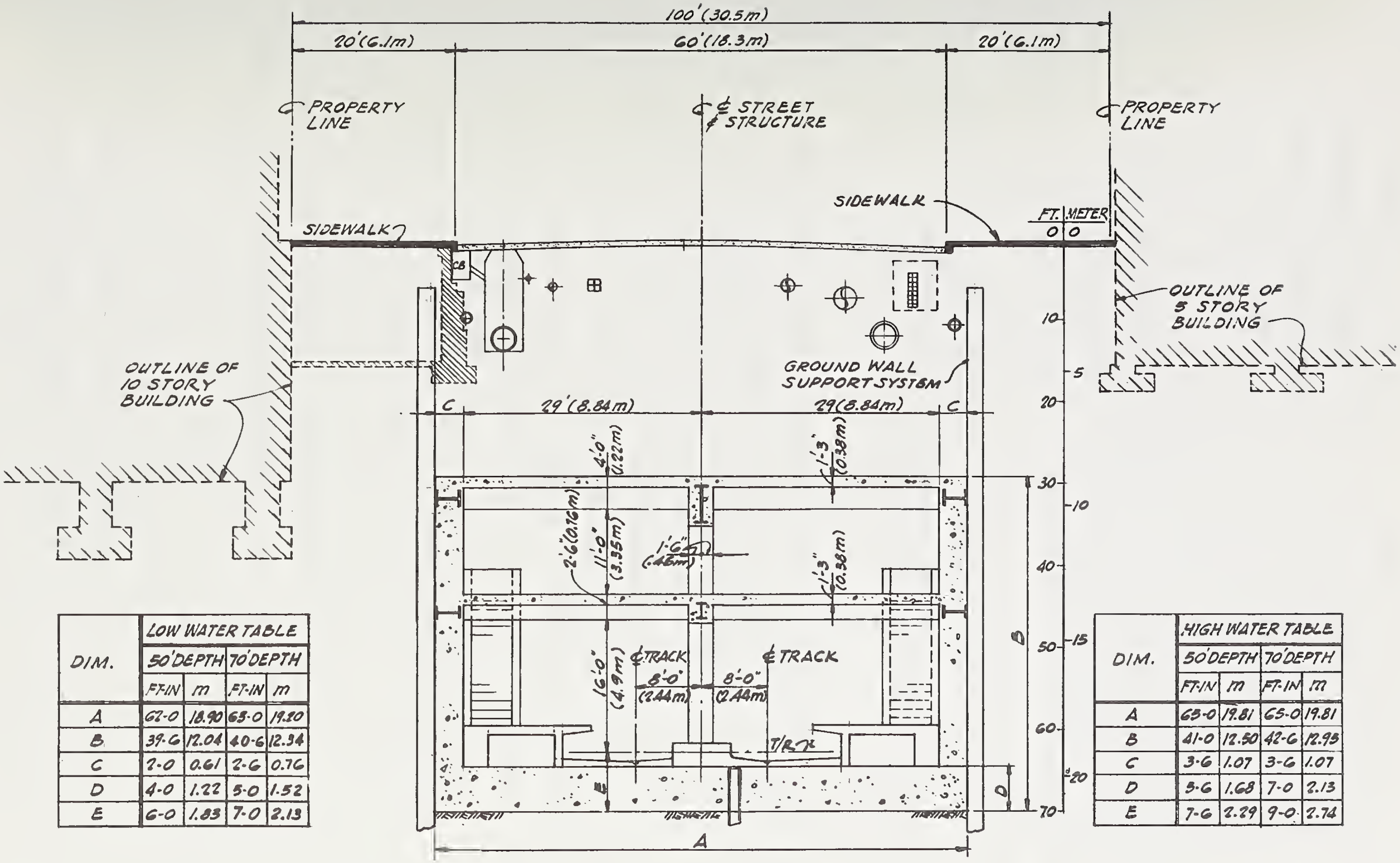


FT-IN	METER
0'-6"	0.15
1'-0"	0.30
2'-0"	0.61
7'-0"	2.13
8'-0"	2.44
12'-0"	3.66

DIM.	LOW WATER TABLE						HIGH WATER TABLE					
	30' DEPTH		50' DEPTH		70' DEPTH		30' DEPTH		50' DEPTH		70' DEPTH	
	FT-IN	m	FT-IN	m	FT-IN	m	FT-IN	m	FT-IN	m	FT-IN	m
A	33-0	10.06	34-0	10.36	35-0	10.67	34-0	10.36	35-0	10.67	36-0	10.97
B	18-0	5.49	20-0	6.10	21-6	6.55	19-6	5.94	21-6	6.55	23-0	7.01
C	1-6	0.46	2-0	0.61	2-6	0.76	2-0	0.61	2-6	0.76	3-0	0.91
D	2-6	0.76	3-6	1.07	4-6	1.37	3-0	0.91	4-0	1.22	5-0	1.52
E	1-6	0.46	2-6	0.76	3-0	0.91	2-6	0.76	3-6	1.07	4-0	1.22

DETAILS OF DOUBLE-BOX RAPID TRANSIT LINE SECTION
(SHOWN AT 50' DEPTH)

Figure 3



DIM.	LOW WATER TABLE			
	50'DEPTH		70'DEPTH	
	FT-IN	m	FT-IN	m
A	62-0	18.90	63-0	19.20
B	39-6	12.04	40-6	12.34
C	2-0	0.61	2-6	0.76
D	4-0	1.22	5-0	1.52
E	6-0	1.83	7-0	2.13

DIM.	HIGH WATER TABLE			
	50'DEPTH		70'DEPTH	
	FT-IN	m	FT-IN	m
A	65-0	19.81	65-0	19.81
B	41-0	12.50	42-6	12.95
C	3-6	1.07	3-6	1.07
D	5-6	1.68	7-0	2.13
E	7-6	2.29	9-0	2.74

DETAILS OF RAPID TRANSIT STATION
(SHOWN AT 70'DEPTH)

Figure 4

the San Francisco Bay Area Rapid Transit District (BARTD) system and the Washingtom Metropolitan Area Transit Authority (WMATA) system. The center wall contains frequent openings allowing track workers a place to stand while trains are passing, and to facilitate air flow around high speed trains. This section is the most economical and efficient cut-and-cover line section in use today. It is preferable to two individual box sections. For crossover sections the center wall is eliminated and the roof slab thickened or reinforced with steel beams. The study will consider a contract length of 2000 feet as in the case of the highway tunnel. With the 700 foot long station to be described next, this allows for stations approximately one half mile apart, which is reasonable in an urban area.

1.2.3 Rapid Transit Station - The cross section of a rapid transit station shown in Figure 4 is similar to several on the new San Francisco Bay Area Rapid Transit system (BARTD). It has reinforced concrete walls and composite structural steel and concrete slabs. The upper mezzanine level contains public areas for ticket booths, turnstiles and walkways as well as work rooms for storage, equipment, pumps, transformers, etc. The lower level contains trackway and platform loading areas.

The station is consistent with the line section shown in Figure 3, with tracks on either side of the center wall and platforms on the outside. When the line sections are driven tunnels instead of being constructed by cut-and-cover, they are usually constructed as twin circular tubes. To minimize mutual interference and ground disturbance between the tubes, they are driven with as much room between as the construction easement and station width permit. They then enter the station along the outside with a center common loading platform area between. There is no single "best" design or layout for stations and they will vary considerably more than any other underground structure. All underground stations on the BARTD system were deliberately made distinctive and designed by different architectural firms. Even where a system attempts to standardize their stations, as in Washington, D.C., varying site conditions preclude exact duplication in most cases. The section shown in Figure 3 is an economical, functional design which can be enhanced by an attractive architectural finish. Although a wide, high arch is more pleasing aesthetically, the arch concept must be weighed against the increased costs involved. The choice between beauty and economics is always difficult. The economics of structural design are discussed further in Section 2.10. The station length of 700 feet used in this study is similar to station lengths of both BARTD and WMATA.

1.3 DEPTH OF STRUCTURES

This is an important parameter in any discussion of cost of cut-and-cover construction. The designer will try to keep a structure as close to the surface as possible. There are times when the natural grade of the ground surface exceeds the allowable grade of road or track, and the designer is forced to locate the structure farther from the surface. Cost of construction increases sharply with increased depth. In addition, considerations of access from the surface, and difficulties of ventilation increase with depth. Tunnel driving costs are usually higher per foot of tunnel than the average shallow cut-and-cover tunnel. At some increased depth, depending on the nature and size of the structure, the costs are similar, and the minimal disruption of the surface by tunneling construction methods then dictate use of this method. This study will help in determining how varying depth affects the cost of cut-and-cover tunnels for comparison with construction by tunneling methods.

In this study, three excavated depths will be considered, 30 foot (91.1 m), 50 foot (15.2 m) and 70 foot (21.3 m), in order to judge the effect of depth on each type of structure. The individual structures in Figures 2, 3 and 4 are shown at a different depth for illustrative purposes, but each type of structure will be considered at all three depths. Because the station structure is about 40 feet (12.2 m) high, it will not be considered for the 30 foot depth. As described in Section 1.1, each of these will be considered for both wet and dry conditions.

1.4 METHOD OF CONSTRUCTION

The most important construction variable to be considered in this first phase of the study is that of ground support. While there are variations of under-the-roof methods of construction of the permanent structure that affect many of the other basic construction activities, they will be considered after the optimization of construction procedures based on conventional construction of the structure.

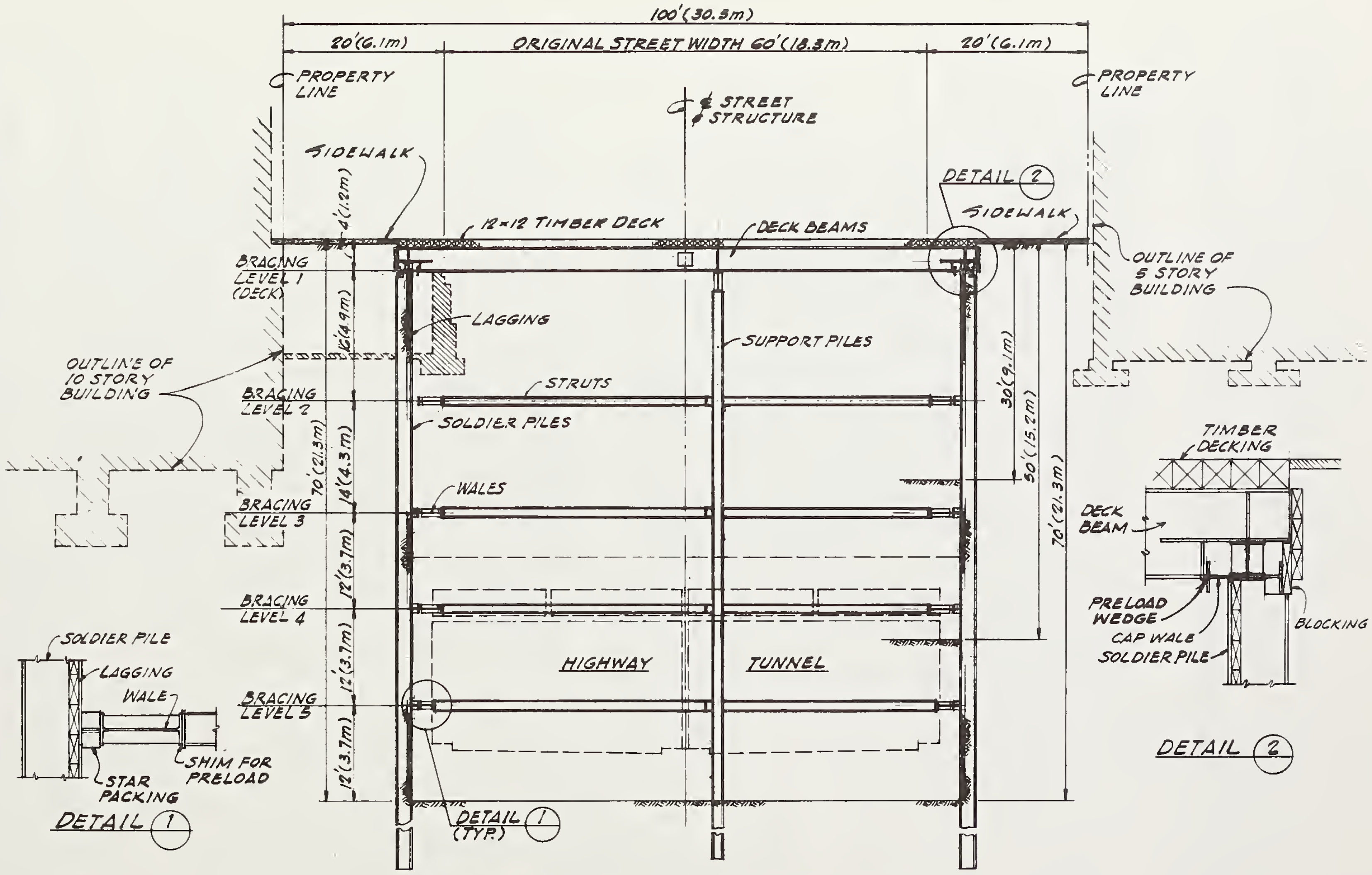
Eight methods of ground support will be discussed in Section 2.7. Several are applicable to special conditions only and therefore are not commonly used. Three types of ground support are specified for this study; 1) Soldier piles and lagging, 2) Cast-in-slurry concrete diaphragm walls, and 3) Precast concrete panel diaphragm walls. A fourth method, interlocking steel sheet piling, has been, and still is an acceptable ground support for cut-and-cover work, and is particularly competitive in wet ground. New high strength steels and fabrication methods have increased the applicability of steel sheeting to cuts of greater depth. Efforts to curb noise pollution, however, have led to restrictions on pile driving in most cities for the type of urban environment used in this study. These restrictions will seriously limit the use of this type of ground support in the future.

1.4.1 Soldier piles and lagging - This is the most common method of ground support used today in cut-and-cover construction in the United States. Of approximately one hundred cut-and-cover contracts completed or in progress on the BARTD and WMATA, four utilized soldier piles and tremie concrete (SPTC) walls entirely, and a few had portions of SPTC walls, or steel sheeted walls. The remainder of ground support walls were soldier piles and lagging. This is not necessarily a sign that contractors are reluctant to change. Although there is a natural unwillingness to experiment with new methods where the contractor is completely liable for failure, contractors do use other methods when the situation calls for it and economics permit.

Soldier piles and lagging are the least expensive method of ground support for normal cut-and-cover work in the United States with its current ratio of material to labor costs. Although there are several variations, the most common consists of vertical steel beams placed 6 feet to 10 feet (1.8 m to 3 m) apart with horizontal timber lagging bearing on the inside flanges. This is illustrated in Figure 5. Although other ground support methods may be competitive in particular situations with special considerations, to replace soldier piles and lagging on a large scale, innovative techniques will be needed to provide economies that cannot be achieved by this method. A more complete description of the soldier pile and lagging method of support with variations and alternative methods is given in Section 2.7.

1.4.2 Cast-in-slurry concrete diaphragm - Two general variations of this type of ground support are currently used. One method, developed in Europe, consists of excavating full depth alternate slots about 20 feet long (6.1 m) using a bentonite slurry to support the sides. A reinforcing cage is placed in the slot which is then filled with tremie concrete. The process is repeated for in-between slots. The method developed in the United States and most used in this country is the soldier pile and tremie concrete (SPTC) wall, shown in Figure 6. Soldier piles placed about 6 feet to 10 feet (1.8 m to 3 m) apart provide the primary ground support as in the case of soldier piles and lagging. Tremie concrete, usually unreinforced is placed in a slurry filled slot between piles to transfer ground load to the piles.

Advantages of this type of ground support include water cutoff and minimal wall deflection. Under certain conditions



SOLDIER PILES AND LAGGING GROUND SUPPORT AND INTERNAL BRACING ALTERNATE DETAILS

Figure 5

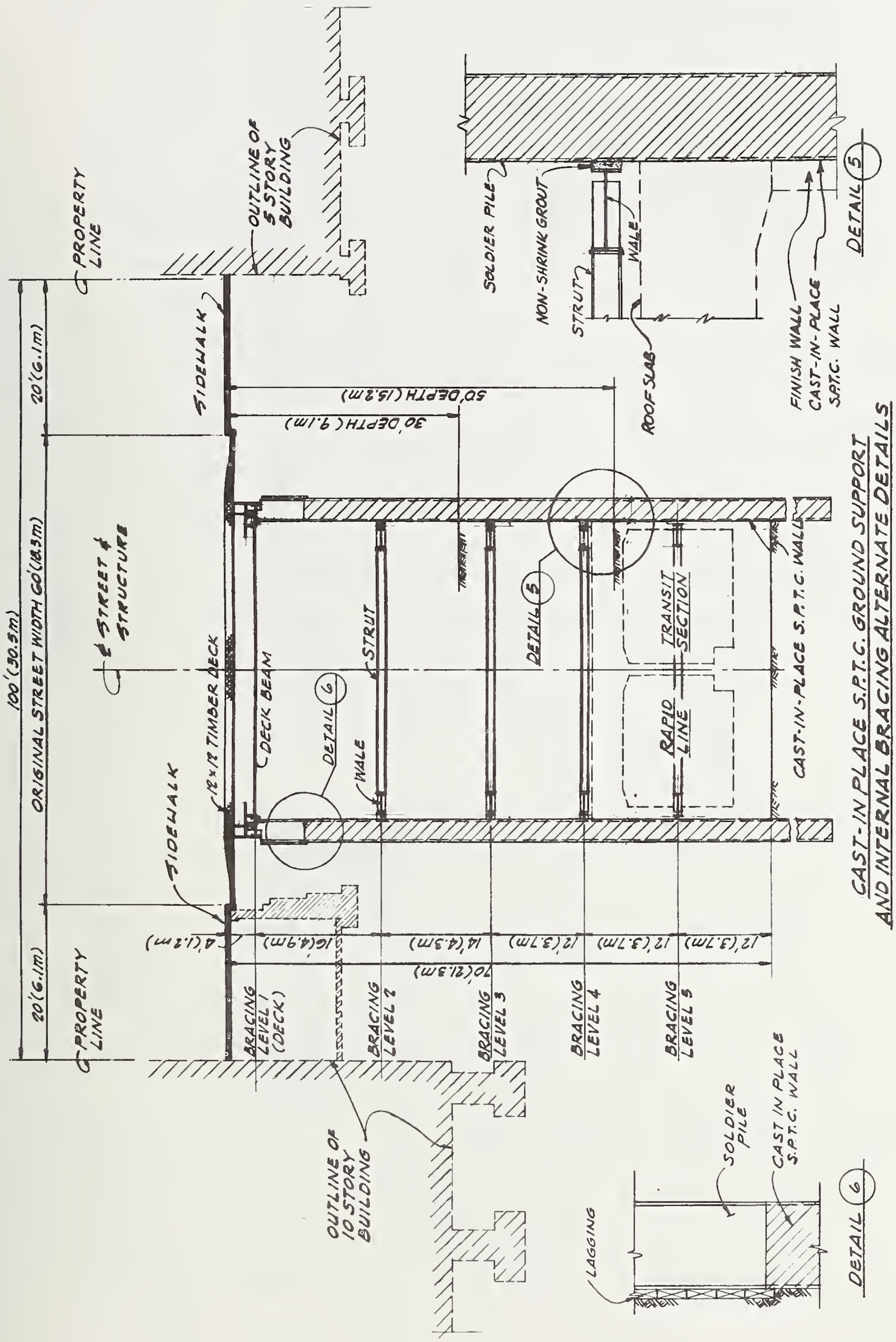

CAST-IN PLACE S.P.T.C. GROUND SUPPORT
AND INTERNAL BRACING ALTERNATE DETAILS
100'(30.5m)
ORIGINAL STREET WIDTH 60'(18.3m)
20'(6.1m)
20'(6.1m)
PROPERTY LINE
PROPERTY LINE
OUTLINE OF 5 STORY BUILDING
OUTLINE OF 10 STORY BUILDING
SIDEWALK
SIDEWALK
12"x12" TIMBER DECK
DECK BEAM
STREET & STRUCTURE
STRUT
WALE
DETAIL 6
DETAIL 5
50'DEPTH (15.2m)
30'DEPTH (9.1m)
70'(21.3m)
4'(1.2m)
16'(4.9m)
14'(4.3m)
12'(3.7m)
12'(3.7m)
12'(3.7m)
BRACING LEVEL 1 (DECK)
BRACING LEVEL 2
BRACING LEVEL 3
BRACING LEVEL 4
BRACING LEVEL 5
TRANSIT SECTION
RAPID LINE
CAST-IN-PLACE S.P.T.C. WALL
SOLDIER PILE
NON-SHRINK GROUT
STRUT
WALE
ROOF SLAB
FINISH WALL
CAST-IN-PLACE S.P.T.C. WALL
DETAIL 5
LAGGING
SOLDIER PILE
CAST IN PLACE S.P.T.C. WALL
DETAIL 6

Figure 6

the diaphragm wall can be incorporated in the permanent structure. Because these cast-in-place walls have a very rough surface they cannot be used without a thin concrete finish wall, or other type of architectural finish, where they are exposed to public view, as in subway stations. In addition, keying the slabs to the completed wall is a problem. Slurry concrete walls have been used on the BART system for Civic Center Station, Powell Street Station and Embarcadero Station; on the WMATA system they were used on the Voice of America Station; in Toronto they were used (unfinished) on a test line section of the University Line. The first four described were SPTC walls; the last reinforced, cast-in-slurry concrete.

1.4.3 Precast concrete diaphragm - This type of ground support is a recent European development and has not been used in the United States. The method developed by Soletanche (Ref. 8) uses vertical precast concrete panels placed in a cement slurry slot. The cement slurry hardens to provide contact with the outside soil and minimize ground movement. This type of ground support is shown in Figure 7. Various configurations of panels have been used. A study is in progress by another contractor to the Department of Transportation to explore the capabilities of this method.

Advantages include the ability to cast in architectural and structural features as needed and to coat the exterior surface with waterproofing. The wall has the same water cutoff capability of cast-in-slurry walls and greater potential for use in the permanent structure. Since the concrete is cast above ground greater quality control is possible. This method of ground support appears to have the greatest potential for improvement of cut-and-cover construction techniques.

1.4.4 Bracing of ground support wall - No discussion of ground support would be complete without mentioning the methods used to brace the basic ground support system. Two methods are in general use, internal cross-bracing and tieback earth anchors. Internal bracing is shown with the ground wall supports in

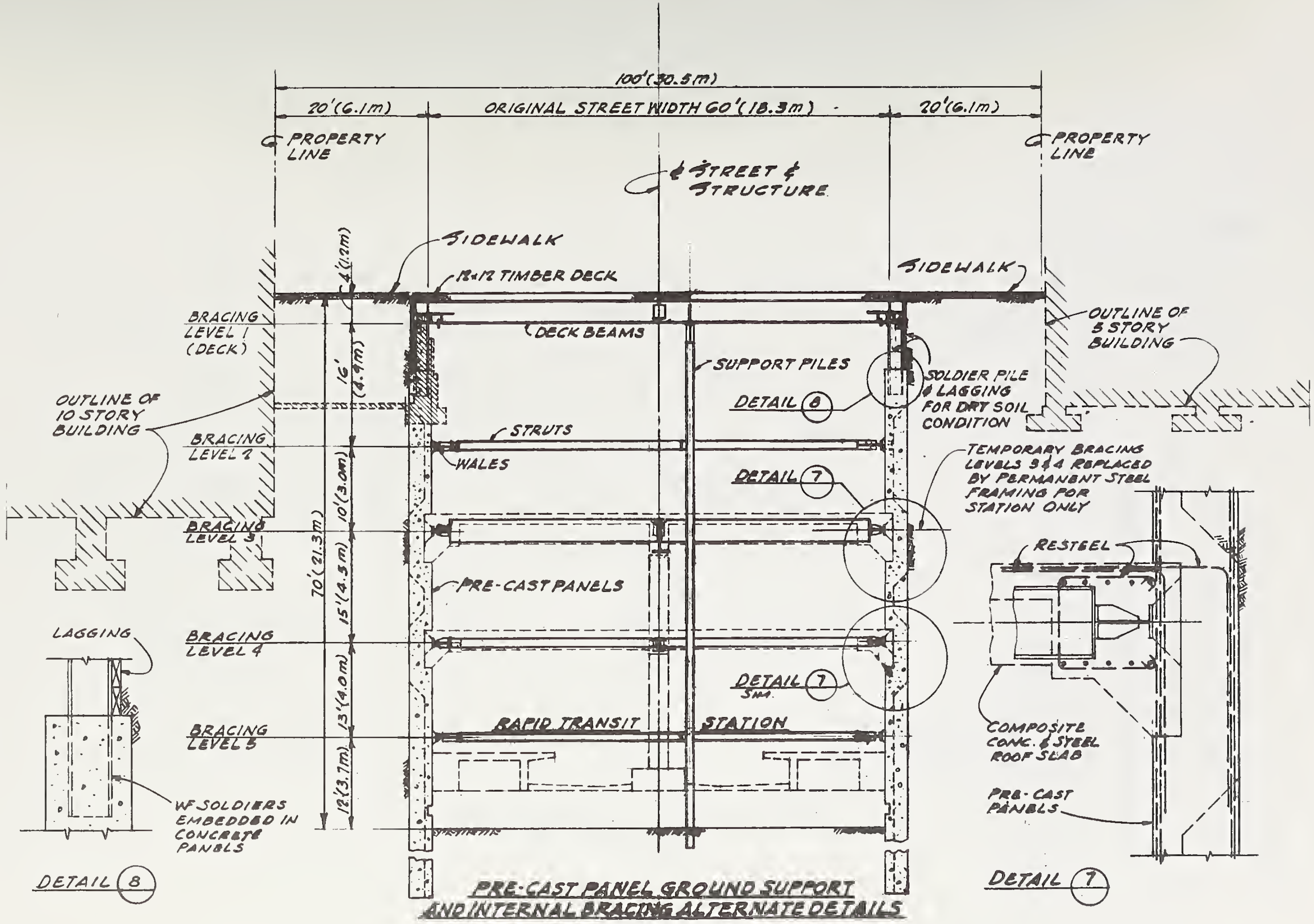

100'(30.5m)
20'(6.1m)
ORIGINAL STREET WIDTH 60'(18.3m)
20'(6.1m)
PROPERTY LINE
PROPERTY LINE
STREET & STRUCTURE
SIDEWALK
12x12 TIMBER DECK
SIDEWALK
DECK BEAMS
OUTLINE OF 5 STORY BUILDING
SUPPORT PILES
SOLDIER PILE & LAGGING FOR DRY SOIL CONDITION
DETAIL 8
OUTLINE OF 10 STORY BUILDING
BRACING LEVEL 1 (DECK)
BRACING LEVEL 2
BRACING LEVEL 3
BRACING LEVEL 4
BRACING LEVEL 5
STRUTS
WALES
DETAIL 7
TEMPORARY BRACING LEVELS 3 & 4 REPLACED BY PERMANENT STEEL FRAMING FOR STATION ONLY
RESTEEL
PRE-CAST PANELS
4'(1.2m)
16' (4.9m)
10'(3.0m)
15'(4.5m)
13'(4.0m)
12'(3.7m)
70'(21.3m)
LAGGING
DETAIL 7 SIM.
COMPOSITE CONC. & STEEL ROOF SLAB
RAPID TRANSIT STATION
WF SOLDIERS EMBEDDED IN CONCRETE PANELS
PRE-CAST PANELS
DETAIL 8
DETAIL 7
PRE-CAST PANEL GROUND SUPPORT AND INTERNAL BRACING ALTERNATE DETAILS

Figure 7

Figures 5, 6 and 7. Figure 8 shows typical tieback earth anchor configuration for all three depths, 30 foot, 50 foot and 70 foot. Either bracing method can be used, provided other factors are favorable, for each of the ground supports discussed above. Section 2.8 is devoted to a more complete discussion of these bracing systems. Although cross-bracing is more common in cut-and-cover tunnel construction, tiebacks have the advantage of providing better access for excavation, construction and backfill.

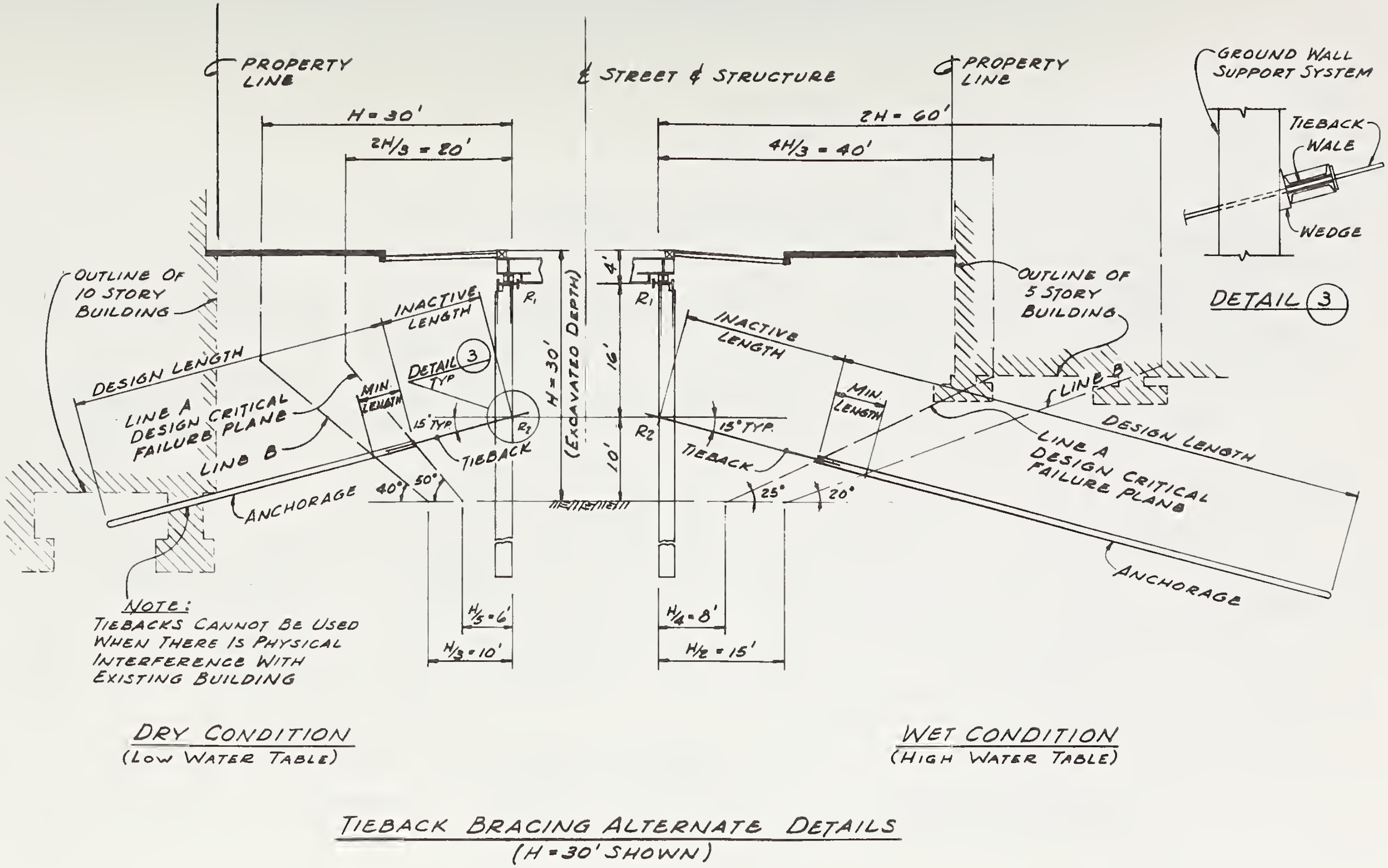
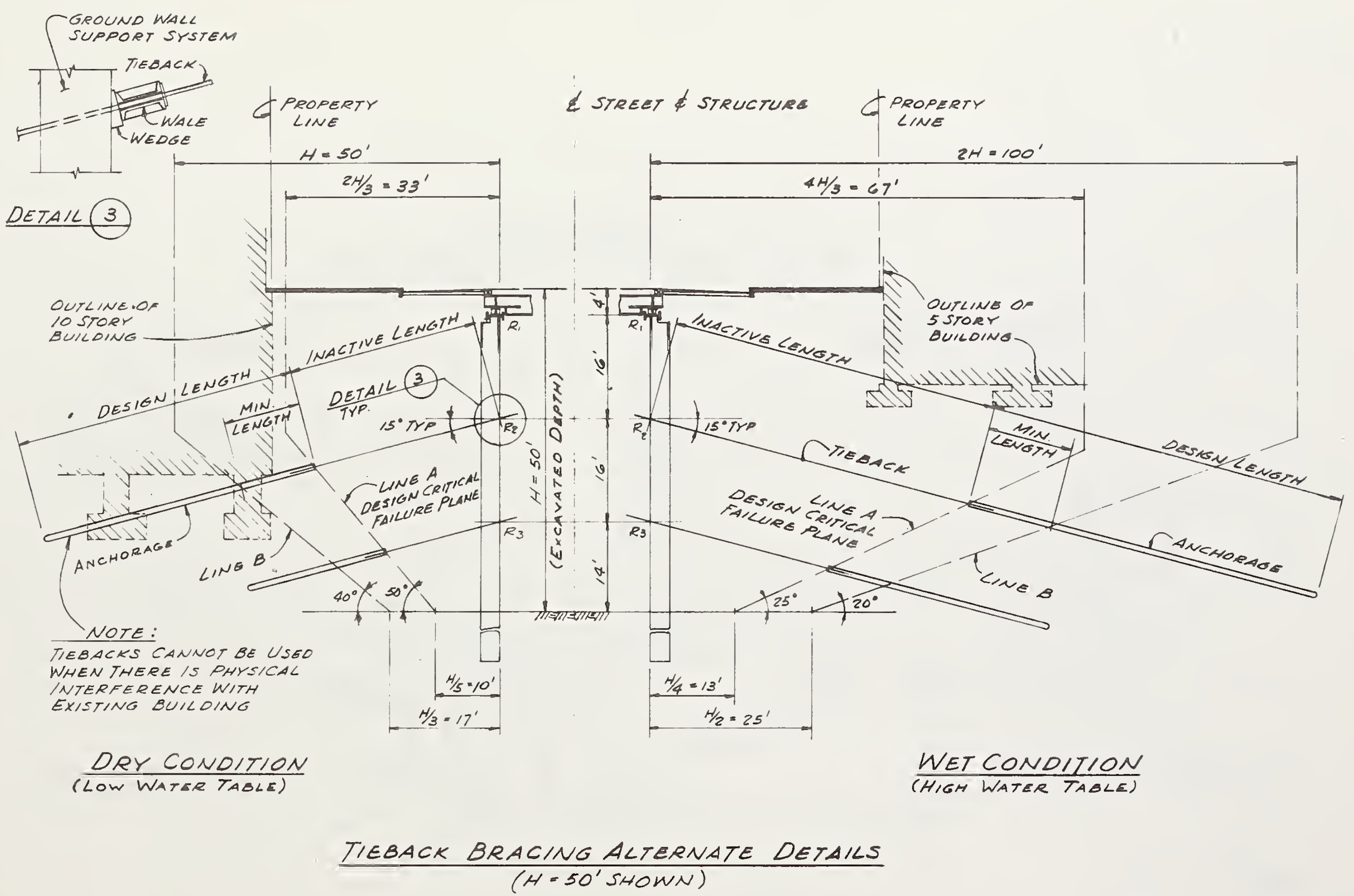


Figure 8 - Sheet 1



TIEBACK BRACING ALTERNATE DETAILS
(H = 50' SHOWN)

Figure 8 - Sheet 2

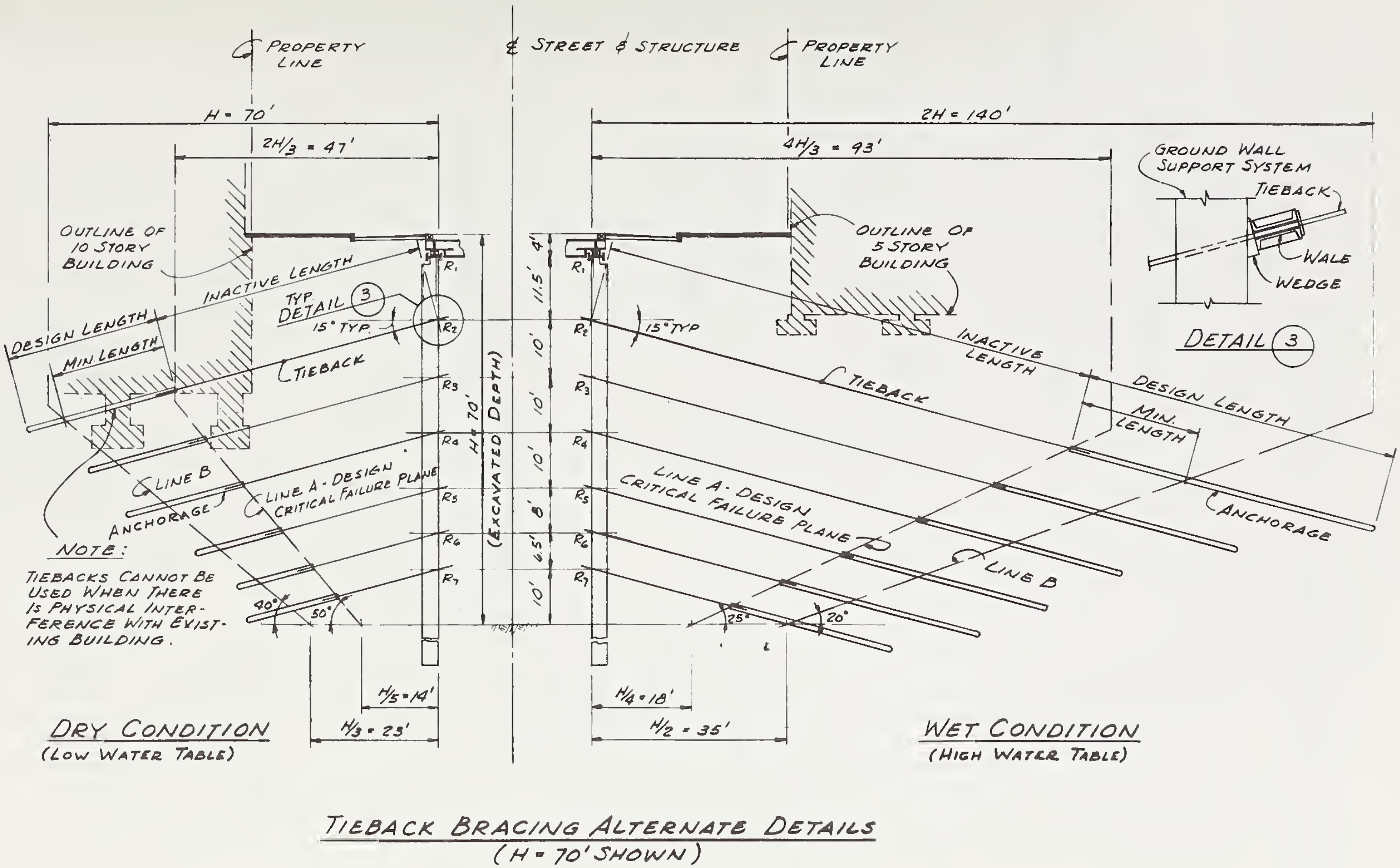


Figure 8 - Sheet 3

SECTION 2

2.0 CONSTRUCTION ACTIVITIES AND METHODS

It is safe to say that no two heavy construction projects are ever alike. Even adjoining sections of the same transportation system will have enough distinguishing characteristics to make each unique. Varying surface and sub-surface conditions, starting at a different time of the year, or an individual contractor's preference of construction methods can alter the project characteristics. To place things in proper perspective we must consider first those things that are average, typical or common for most situations, and then investigate how varying situations and methods affect not only individual activities but the overall construction process. The successful project is one that is completed with a minimum of wasted effort and time. The successful contractor is one who knows how to balance, coordinate and, if necessary, change his construction methods to achieve this result.

2.1 CONSTRUCTION ACTIVITIES

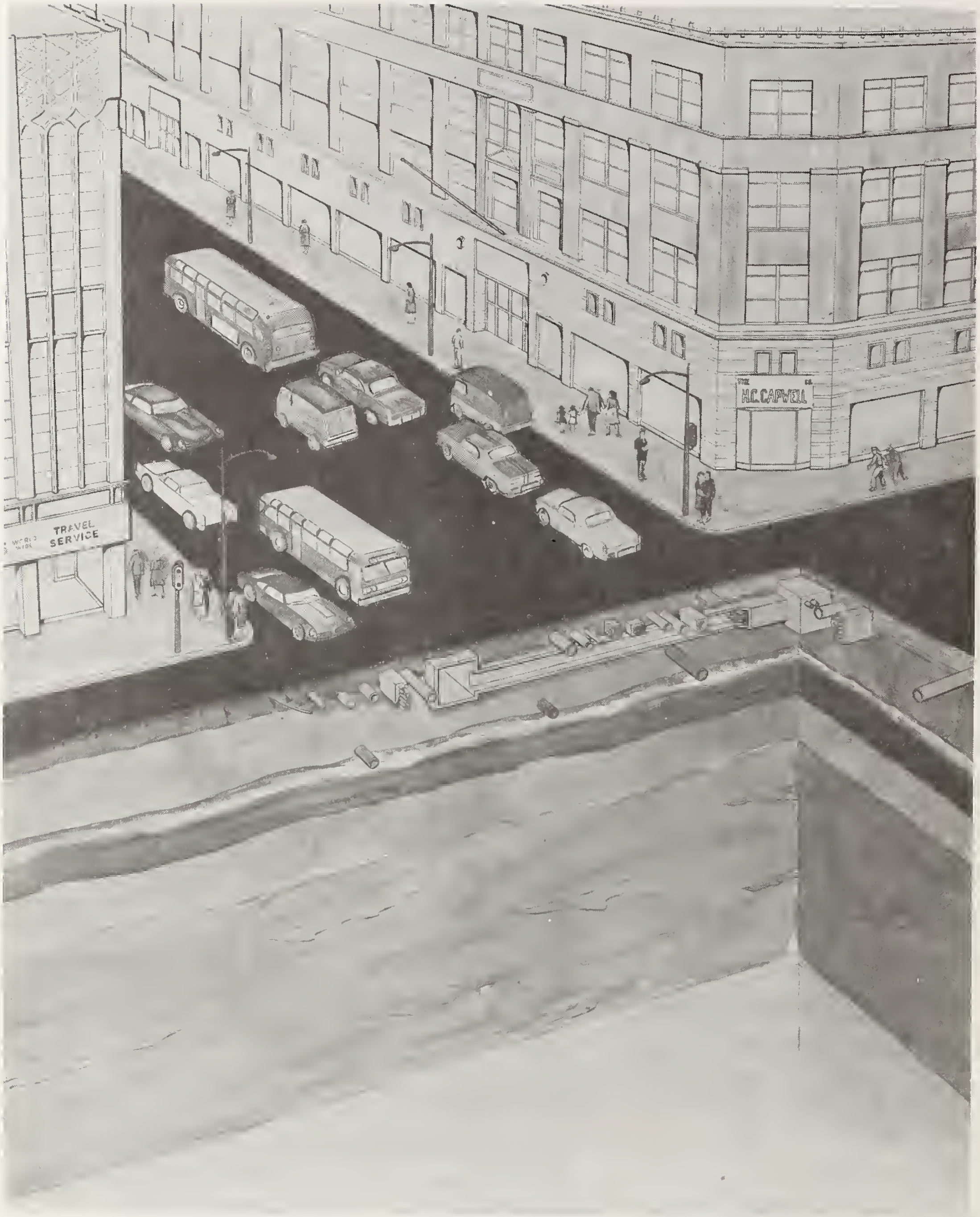
There are a number of basic activities that are common to construction projects in the same field whether that field is highway construction, building construction or tunneling. The same holds true for construction of cut-and-cover highway tunnels, rapid transit tunnels or rapid transit stations. The following list of major activities may be considered basic for all situations to be considered in this study. It will provide a common basis of comparison for varying situations and construction methods. While some of these are closely related they are nevertheless distinct activities. Some, as utility maintenance and restoration, contain a multiple variety of possible sub-activities; others such as ground water control are dependent on site conditions and may not be required on a particular individual project.

These basic activities, in approximate order of appearance on most projects, are;

A. Traffic Control
B. Maintain, Replace or Relocate Utilities
C. Protection of Adjacent Structures
D. Ground water Control
E. Installation of Decking
F. Installation of Ground-wall Support
G. Bracing of Ground-wall Support
H. Excavation
I. Construction of Permanent Structure
J. Backfill
K. Restoration

While these general activities may be considered basic, the sub-activities within each may vary considerably on individual projects. At times these sub-activities are dependent on the construction method chosen, as in the case of sub-activities to be followed when a particular ground support system has been chosen. In other cases the method employed may be restricted by other factors inherent in the project environment and accepted as intrinsic to a particular sub-activity. Hauling of excavated muck from the job site, for instance, is handled by dump trucks on more than 95% of urban projects, due to traffic and street conditions encountered. Most activities can be accomplished by alternate methods and therefore more flexible. Figure 9 illustrates typical work activities used in constructing a station on the BART system. These drawings are reproduced by permission of the Perini Corporation.

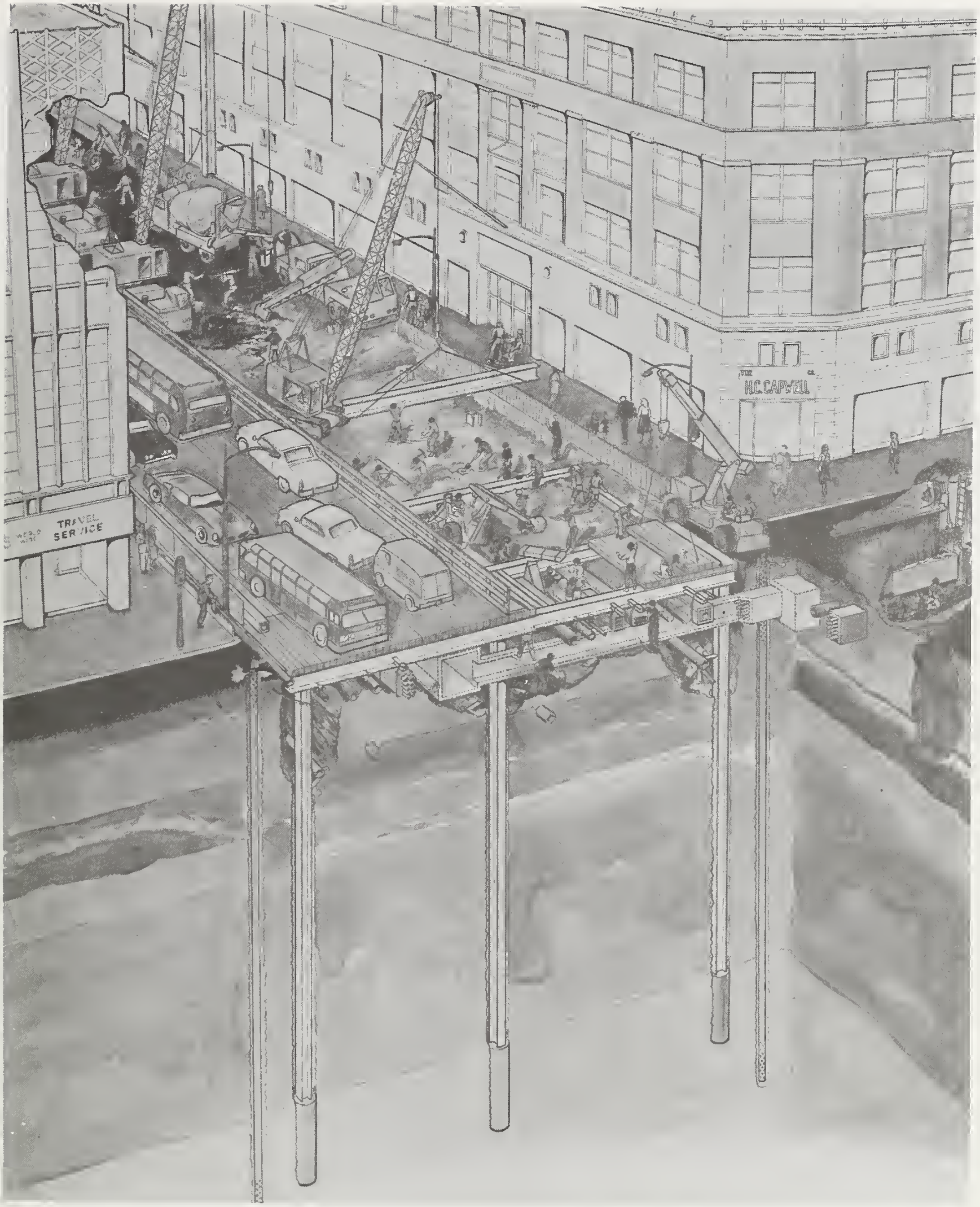
In the remainder of this section of the report, each basic activity is considered individually, describing the alternative sub-activities and construction methods that are available, how they are interrelated, where they complement each other and where they conflict. An attempt is made to show how these various factors affect the different choices that must be made on each project.



TYPICAL CUT-AND-COVER CONSTRUCTION ACTIVITIES

WORKSITE PRIOR TO START OF PROJECT SHOWING EXISTING UTILITIES BELOW STREET SURFACE.
(COURTESY OF THE PERINI CORPORATION)

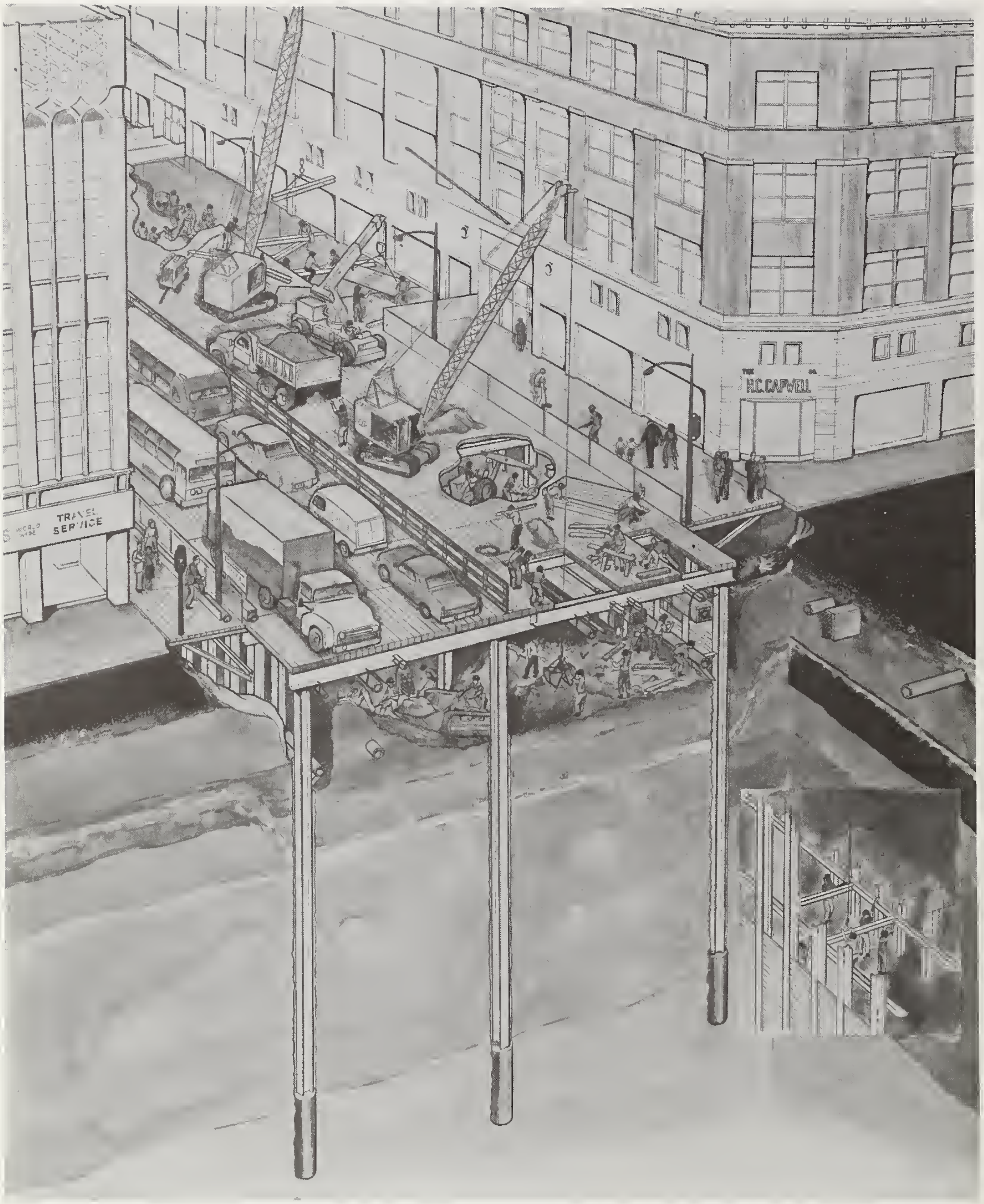
FIGURE 9 - SHEET 1



TYPICAL CUT-AND-COVER CONSTRUCTION ACTIVITIES

SHOWING EARLY ACTIVITIES OF (A) TRAFFIC CONTROL, (B) UTILITY MAINTENANCE, (C) UNDERPINNING (INSERT) (D) DEWATERING WELL, CASINGS, (E) DECKING INSTALLATION, AND (H) FIRST PASS OF EXCAVATION.
(COURTESY OF THE PERINI CORPORATION)

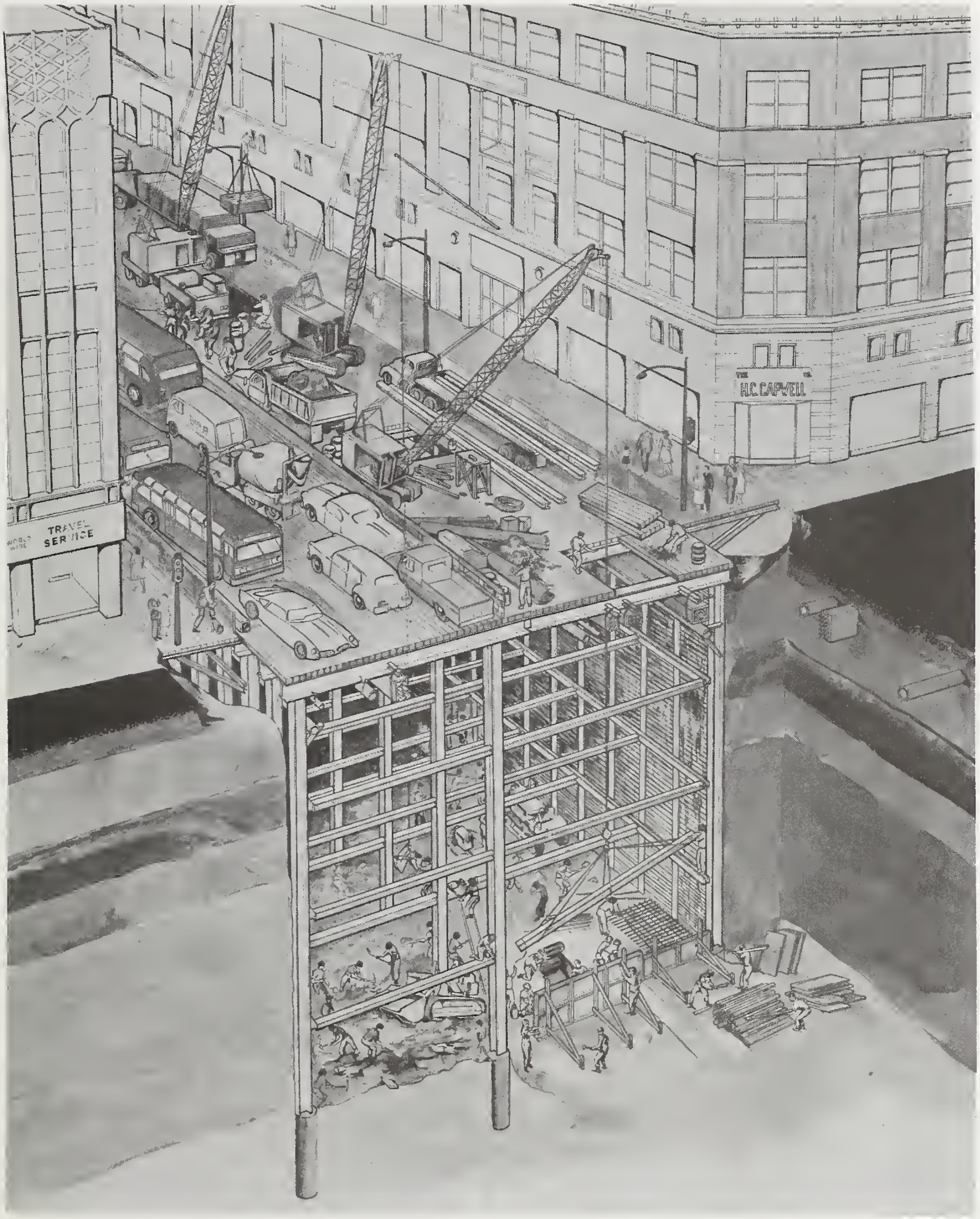
FIGURE 9 - SHEET 2



TYPICAL CUT-AND-COVER CONSTRUCTION ACTIVITIES

SHOWING (C) PROTECTION OF ADJACENT STRUCTURES (INSERT), (E) COMPLETION OF DECKING INSTALLATION, AND (H) SECOND PASS OF EXCAVATION.
(COURTESY OF THE PERINI CORPORATION)

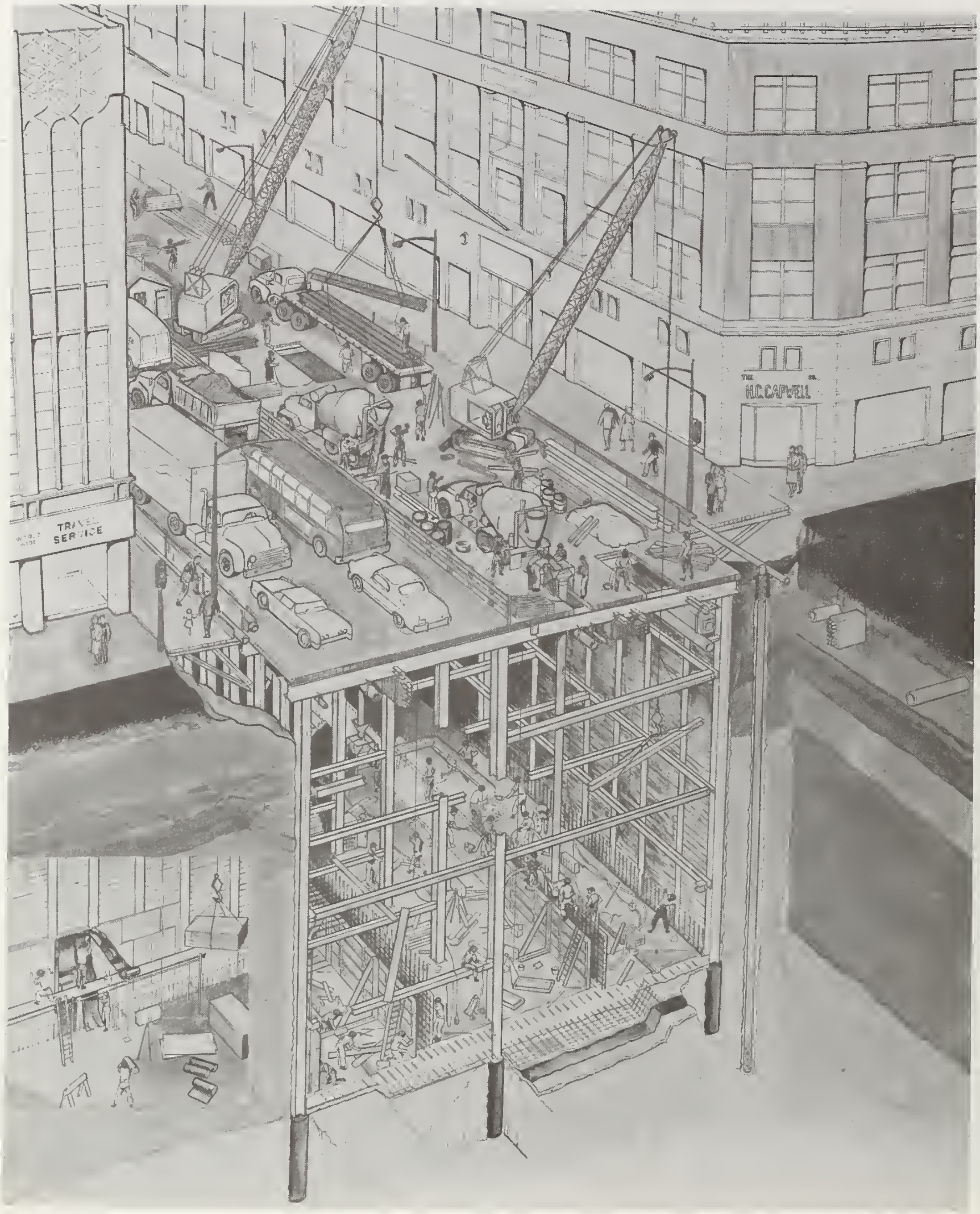
FIGURE 9 - SHEET 3



TYPICAL CUT-AND-COVER CONSTRUCTION ACTIVITIES

SHOWING (F) GROUND WALL SUPPORT, (G) BRACING INSTALLATION, (H) EXCAVATION, AND SETTING FORMS FOR BASE SLAB.
(COURTESY OF THE PERINI CORPORATION)

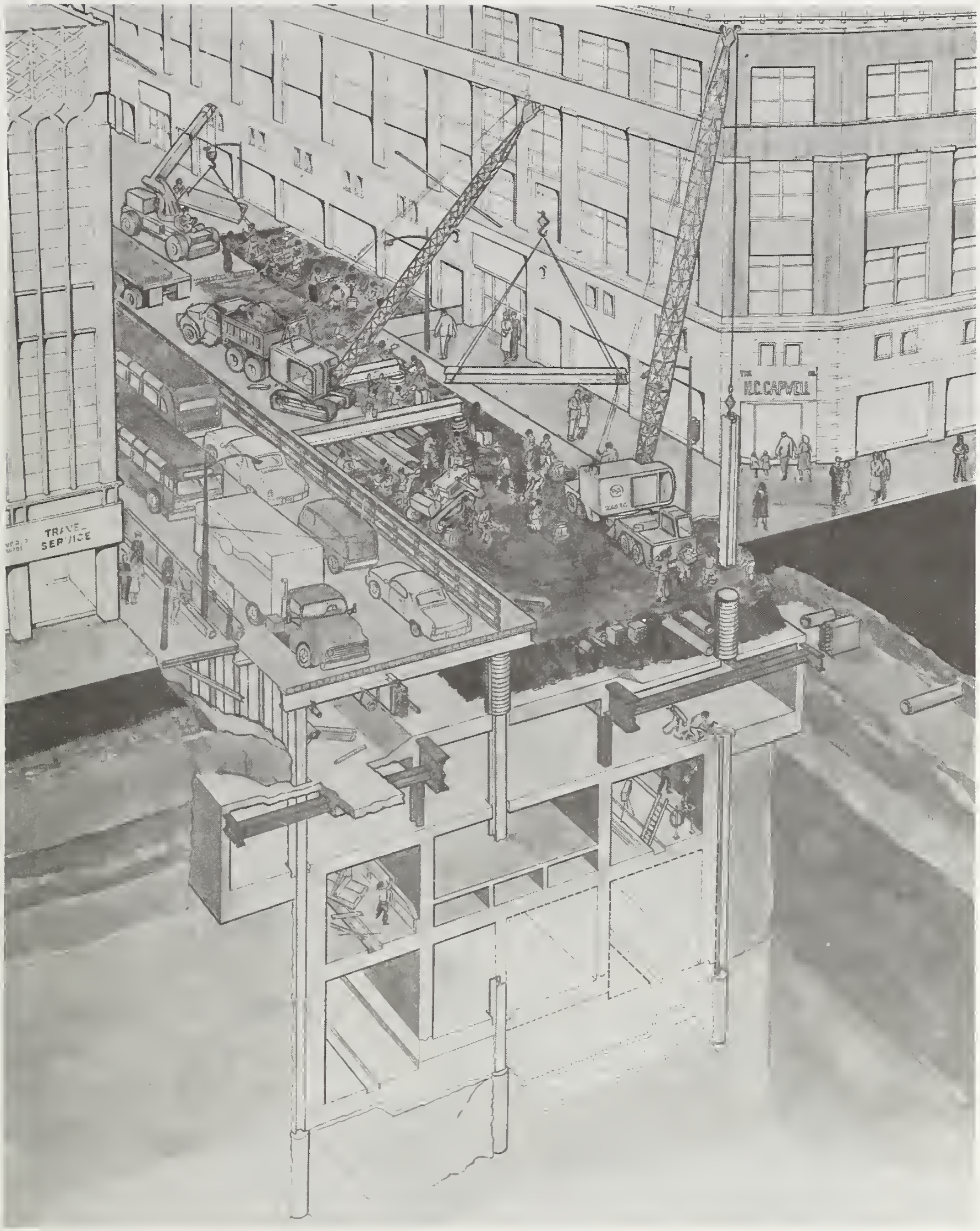
FIGURE 9 - SHEET 4



TYPICAL CUT-AND-COVER CONSTRUCTION ACTIVITIES

SHOWING (G) REMOVAL OF BRACING AND (I) CONSTRUCTION OF THE PERMANENT STRUCTURE. INSERT SHOWS INSTALLATION OF OUTER WALL WATERPROOFING.
(COURTESY OF THE PERINI CORPORATION)

FIGURE 9 - SHEET 5



TYPICAL CUT-AND-COVER CONSTRUCTION ACTIVITIES

SHOWING (I) COMPLETION OF PERMANENT STRUCTURE, (J) BACKFILL, REMOVAL OF STREET DECKING, AND (K) SURFACE RESTORATION.
(COURTESY OF THE PERINI CORPORATION)

FIGURE 9 · SHEET 6

2.2 TRAFFIC CONTROL

Disruption of traffic in an urban area is the plague of cut-and-cover construction. Noise and dust receive their share of complaints, but these can be controlled to some extent to minimize nuisance. It is the day-to-day rerouted obstacle course of construction equipment, barricades, flagmen, and rattling deck beams that create an impression of confusion and personal affront to the daily commuter or casual visitor. While this problem is inherent in street excavation of any sort in an urban environment, it is the long duration of large projects such as highway tunnels and mass transit systems that aggravate the situation.

In a downtown area a large cut-and-cover project is normally constructed in congested public street areas. Privately owned and developed property is usually too expensive to purchase and demolish in order to construct the project off the street, unless the work is done in conjunction with a redevelopment program. It is the congestion of the downtown area that creates the need for the tunnel in the first place. Though not the most important, nor the first consideration in planning, traffic control is one of the first activities the contractor is faced with at the beginning of a project and one which continues to the final stages of restoration.

There are situations of disruptions to traffic other than normal urban pedestrian and vehicular traffic, but these are unusual situations that should be treated individually. These include cut-and-cover crossing of existing highways, railroads, vehicular or transit tunnels, or even aircraft runways or canals. As an example, a cut-and-cover section of the Washington subway was constructed partly under the existing Army-Navy Club building.

2.2.1 Relative needs of traffic access - While it might be ideal from the point of view of construction operations to ban or reroute all traffic from the job site, this is hardly ever practical. Of prime importance is the need for continuous access of pedestrian traffic to adjacent buildings. Even under conditions where all vehicular traffic can be bypassed to adjacent streets, the need of people to enter and leave nearby buildings when they choose is considered basic and fundamental.

Equally important is the need of access for emergency vehicles such as fire trucks and ambulances, though these may sometimes have access through side streets or back alleys. Next in order of priority are service vehicles and delivery trucks for commercial businesses in the area, then transportation vehicles such as busses and trolleys. Last, but certainly not least, is the ever abundant private car. Though private cars may be the first to be routed to adjacent streets, their very numbers may make this an unattractive option to be used only temporarily or in cases of emergency.

It can be generally assumed that the traffic flow on the street being considered is in a major direction in the downtown area. This is based on the assumption that the purpose of the proposed tunnel is to move people and vehicles into and out of the central part of the city. There is always traffic movement crosswise to this direction that must be considered at intersections. This cross traffic introduces additional complications to the traffic flow problem.

During construction, one of the major considerations in the competition for space at the street level is the location of a construction plant and working room for construction equipment. Where possible, an area should be provided adjacent to the excavation site for the contractor's plant to minimize the use of on-site space. This plant consists of temporary buildings, electrical substation, equipment storage, material storage, dewatering equipment, slurry plants, and other maintenance facilities. If vacant lots are not available for this purpose it may be possible to utilize portions of a side street. Construction equipment that must occupy street (or deck) space at various times includes cranes, backhoes and dump trucks for excavation, concrete pumps and concrete batch trucks, fork lifts and flatbed trailers for delivery of needed materials directly to the jobsite, etc. Because of heavy traffic, contractor use of street (or deck) is often restricted during rush hours.

2.2.2 <u>Traffic diversion options available</u> - As mentioned above the complete diversion of traffic from the job site is generally impractical in an urban area except on a short term basis. Partial diversion by one or more methods is sometimes available to the contractor. If the structure is narrow in comparison to the width of the street it is possible to maintain one or more traffic lanes outside the net line of excavation. Usually a ten foot (3 m) wide sidewalk is considered adequate for temporary pedestrian walkways, so it may be possible to utilize part of the original sidewalk for the temporary peripheral roads.

If traffic on adjacent streets is not already saturated, part of the traffic may be diverted away from the jobsite. One way to do this is to limit traffic to one direction and divert traffic in the opposite direction to parallel streets. Another possibility is to divert automobiles while maintaining emergency, service and mass transportation vehicles. In any event, it is common practice to eliminate curb parking or limit it to deliveries during non-rush hours only.

In many urban areas it is impractical, or insufficient, to reduce traffic for the life of the project, and a temporary roadway or decking is constructed to carry most or all traffic during the excavation, construction and backfill operations. While this limits major disruption of traffic to the decking installation and removal stages there will still be limited mutual interferences of traffic and construction equipment. It will be assumed for the purpose of this study that decking is required at the urban sites under construction and various decking installation options will be described in Section 2.6. The analysis process resulting from this study will be flexible enough to eliminate the decking (or any other item) for a situation where it is not required.

2.2.3 Traffic flow - While a number of traffic diversion options are indicated above, the actual plan used will not necessarily be decided by the contractor, but will probably be the result of compromises with the local traffic division, private property owners, civic groups, the owner's A & E representatives and local mass transportation interests. The contractor will probably be requested to submit detailed plans of traffic diversion during various stages of the operation, for review and approval by all concerned, before proceeding with the work. It will be the contractor's responsibility to implement and coordinate the traffic control. The plans should identify all traffic flow aids: warning and directional sign locations, traffic signal locations, flashing warning lights at obstacles and lane changes, fixed and movable barricades, location of deck openings, material storage and stages of construction. Traffic signals should be utilized to the fullest extent possible; flagmen should be reserved only for helping to move slow equipment or to aid in unusual operations, rather than serving day-to-day traffic officer duties.

Though it is not typical, some communities will make policemen available on a contract basis to direct traffic when needed rather than on a discretionary or emergency basis. Under this type of system, policemen assigned by the police department, are available when needed. They do not work directly for the contractor, but the contractor reimburses the city for their time and is assured of trained traffic officers when required. This can be incorporated into the contract as a pay item. The contractor must pay flagmen in any event, and experienced policemen are preferable to willing but untrained workers who are sometimes ignored by irate drivers.

Whenever possible major changes to traffic flow such as those involved in the construction of decking in stages, should be made on weekends with sufficient advance notice given to all concerned. In particular, work involving intersections should be performed on a weekend. This will be discussed more fully in the section on decking.

2.2.4 Restrictions to traffic control imposed by type of structure - In general the methods used for flow and control of traffic are not directly related to the type of cut-and-cover tunnel to be built. One exception to this is the width of the structure in relation to the existing street width. The narrower the structure the less the interference with normal street activities and in particular, pedestrian traffic. Structures with column free arches spanning the width of a street can create difficulties in constructing decking with resulting increased problems of traffic flow. Structures with appendages in the sidewalk area such as entrances, shafts and ventilators, increase the problems of traffic flow at the street level. Conversely, structures with shafts to the surface on community owned property off the street, reduce the need for contractor's equipment to occupy vital street areas.

2.2.5 Compatibility of traffic control methods to other construction operations - All methods of ground support installation depend on large equipment: cranes, backhoes, piledrivers, etc. to occupy portions of the street, resulting in a disruption to normal traffic volume. The initial trenching needed to expose utilities precludes limiting this interference to non-rush hour periods. Ground support systems involving slurry trenches require recirculating systems and settling tanks, but these can usually be placed off the street or on less active side streets. Deliveries of long steel piles or precast concrete sections must be planned to meet construction schedules as job site storage space is usually at a premium. During excavation, access must be provided to accommodate maximum production. This usually means shafts or deck openings for clamshell operation or ramps for direct truck or front end loader access. If these ramps cannot enter from a side street or adjacent vacant lot, they provide continuous interference to traffic during this portion of the work. Deck openings can be covered when not in use. In some types of soil, novel material handling methods utilizing conveyors or piping can reduce the interference because of reduced access requirements.

During construction of the permanent structure interference with traffic can be minimized by using a concrete pump on a side street or limiting gravity pours through the decking to non-rush hour periods.

2.2.6 Summary

A. Traffic Control

1. Need for access to site
    a. Pedestrians - Type and size of buildings
    b. Emergency and service vehicles
    c. Transportation vehicles (buses, trolleys)
    d. Private cars - normal volume: daytime; rush hour
    e. One way traffic or two way
    f. Cross traffic at intersections
    g. Contractors equipment
2. Traffic diversion options available
    a. Traffic routed to side streets: all; part
    b. Two way traffic reduced temporarily to one way, with other diverted
    c. Traffic on temporary side roads
    d. Decking installed for public and/or contractor use
3. Constraints imposed by type of structure
    a. Width of general excavation
    b. Flat roof structure with columns or arched roof (decking) supported on center posts permits side to side traffic diversion)
    c. Openings required outside general excavation (stairways, vent structures, etc.)
4. Constraints imposed by construction methods
    a. Space for equipment driving piles or excavating slurry trench

b. Space for auxiliary equipment (slurry recirculation, dewatering equipment, etc.)
c. Space for general excavation equipment (cranes and access shafts, truck ramps, etc.)
d. Space for material handling equipment and trucks delivering materials

2.3 MAINTENANCE, SUPPORT, REPLACEMENT AND/OR RELOCATION OF UTILITIES

In ancient Rome sewers consisted of ditches in the middle of the streets; water was brought from mountain streams in elevated viaducts; fire provided both heat and light. To conduct water directly into a bath, an engineer conceived the notion of utilizing a cast iron pipe placed carefully out of sight - underground -- That's how it all began.

In the ever increasing rivalry for space on and above the earth's surface, man has found it more and more expedient to follow the lead of that Roman engineer in placing utilities carefully out of sight - underground. This is particularly true in large cities, where one utility after another is relegated to the nether world, and except for necessary repairs, is virtually forgotten. In the case of utilities no longer useful and abandoned, this is often quite literally true. Over the years our downtown urban streets have been filled with a myriad of criss-crossing lines of varying shapes and sizes, many of which are not accurately located. It is a confusing and amazing sight, even to those who helped create it, when a large cut-and-cover excavation exposes this labyrinth to the light of day. While traffic disruption is the most likely problem to raise the ire of the commuting public, and ground support and bracing problems most likely to test the technical skill of the contractor, the problems connected with maintaining and handling utilities usually result in the biggest headaches.

2.3.1 Types of utilities - Generally the utilities in major streets will include among others, large sewer mains and local sewer lines including house connections, catch basins and manholes. Local sewer lines may be 8" to 24" in diameter, mains may be anywhere up to 8' or more. Those greater than 4' in diameter should be treated separately, and if possible, rerouted prior to the time the general construction contract is let. Water lines may consist of mains 12" to 36" in diameter and local lines 6" or 8" with 1" to 6" lateral connections to buildings. Gas lines run about the same size as water lines except that laterals are smaller, rarely exceeding 2". Electrical cable and telephone lines may be found in individual metal, fiber, plastic or asbestos conduits or in banks of ducts of terra cotta, vitrified clay or concrete. Precast or cast-in-place pull boxes and splicing chambers may in some cases reach the size of a large room.

Special utilities may be found in some urban areas that would not normally be found elsewhere. Insulated steam lines may connect a group of related buildings or apartment houses using a common heating plant. Local industrial buildings may have a variety of special connecting lines. One utility tunnel near Ruhr University in Bochum, Germany contains, in addition to water, waste, electrical and telephone lines, the following: a steam line, remote cooling lines, remote heating lines, condensed water line, a pneumatic mail tube and a pipe line for beer. Most special utilities, particularly those subject to thermal losses, are relatively short, usually running across streets rather than longitudinally along the street.

Not all utilities are located below ground. Among those utilities to be maintained above ground are traffic signals and light standards. In some cities electric bus or trolley wires hang suspended from poles and must be continuously supported.

2.3.2 Optional methods of handling utilities during construction - Although a variety of methods may be employed for maintaining continuous service of the utilities affected by cut-and-cover construction, not all are applicable or desirable for each type of utility. While the method used is often the option of the contractor, certain restrictions limit the final choice. These will be discussed individually as they affect each type of utility. The options include the following:

a. Support and maintain in place. If the utility is in good condition and it is not necessary to relocate or replace it prior to construction it is hung from the deck beams and maintained in place. Some utilities such as duct banks may have to be stiffened with steel beams or concrete encasement, or protected with timber against damage by construction equipment.

b. Replace with temporary utility. In a case where the existing utility is in poor condition but it is not practical to replace it immediately with a permanent replacement, it may be necessary to substitute a temporary line during construction. In the case of heavy concrete pipe or underground chambers, it may be expedient to substitute lightweight corrugated pipe and plywood chambers to minimize the load on the decking.

c. Replace with permanent utility. In a case where the utility position can remain unchanged but the pipe or conduit must be replaced because of age, it is probably best to replace it during excavation and maintain the new pipe in place. Some utilities such as cast iron water or gas lines are highly susceptible to impact damage and are replaced with steel pipe which is more adaptable to being supported from decking subject to traffic impact.

d. Relocate temporarily within site area. In cases where the utility cannot be maintained in place because of conflict with the structure being built, the ground support system, or other construction activities, it may be moved or relocated temporarily and replaced during the restoration period.

e. Relocate temporarily outside site area. Under certain conditions, it may be more desirable to temporarily relocate a utility off site during construction and return it to the site during restoration. This would particularly apply to large water mains and gas lines because of their hazard potential.

f. Relocate permanently within site area. It may be necessary under certain conditions, to install a permanent replacement for an existing utility that must be relocated because of a conflict with the structure or construction activities.

g. Relocate permanently outside site area. This may be the most advantageous alternative from the point of construction convenience but may be the most expensive if long distances are involved in relocating to an adjacent street. Generally only major utilities (water mains, etc.) that do not directly service the buildings on the street can be relocated in this manner. In the case of a large sewer that will conflict with the structure, and cannot be raised because of required flow characteristics, it may be the only solution.

2.3.3 Factors affecting the choice of utility treatment - There are in most cities a multiplicity of agencies, public and private, who own or control the various utilities to be considered. Each is primarily interested in the lines under their jurisdiction and very often have strict rules on how work must be performed and who is permitted to do it. Many utilities insist on doing their own work or having it done by a limited number of qualified contractors. While this may be important from the point of view of the utility company to insure quality of work, it increases the problems of the contractor in coordinating the work of crews over which he has no direct control. It is of the utmost importance to the timely completion of the work that all utility agencies be involved in the pre-construction planning stage. When the requirements of each are known, the contractor will be in a better position to plan his work and estimate costs. Discovery of previously unknown requirements during the course of construction can be disruptive and costly. Where possible, it is preferable to relocate or replace utilities prior to general construction to reduce interference with installation of ground support and decking as well as general excavation. For instance, on the BARTD system, cast iron water lines passing over an area to be excavated were replaced with steel pipes prior to award of the major contracts.

Most communities now have regulations requiring new electric lines be placed underground, with those existing above ground to be relocated below within a given time frame. It is probable, therefore, that in a city where wires are still carried on poles, the contractor will be expected to replace them below ground.

The method of ground support plays an important role in considerations of maintaining utilities. Installation of soldier piles and lagging is the most flexible in avoiding existing utilities. Soldier pile locations can often be adjusted in the field to miss pipe lines. Diaphragm type walls do not possess this ability. They are continuous by definition, and whether placed in connecting sheets or panels or concreted in a continuous slot, all utilities crossing the wall line must be moved at least once. If they cannot be moved outside the site area, this move can only be made after the installation of the wall has progressed to a point where the relocated line can be placed above it, where it need not be disturbed again. In some cases this can be accommodated in the schedule of placing the wall; in others it will necessitate a delay. Since these lines must be moved anyway, the possibility of permanent relocation should be given more consideration where a diaphragm type ground support is planned.

Other construction activities may require relocation of utilities. Excavated material is often lifted to the surface by a large clamshell which can place the material in a truck or loading hopper. A 3 cu. yd. clamshell requires a 14 foot x 14 foot (4.3 m x 4.3 m) access hole. These shafts also serve for lowering materials. While there is some flexibility in locating these shafts, it is usually difficult to do so without interfering with some utilities. The placing of deck beams or bracing may also be in conflict with utilities. Because of long spans and severe loading conditions imposed by heavy construction equipment, deck beams may be 36" deep and covered by 12" or 14" deck timbers. In areas where frost is not a problem some utilities may be less than 4' deep for long stretches, creating a conflict with the deck beams.

The structure to be constructed also affects the decisions of maintaining or relocating utilities. While it is desirable to keep the structure as close to the surface as possible this can result in forcing utilities to be relocated. This is most likely to affect sewers that must maintain gravity flow characteristics. In addition to possible roof interference, most underground structures require access to the street level for a variety of needs. These include entrance stairs and escalators, ventilation structures, equipment access shafts, and elevators for disabled persons. While these needs exist for all rapid transit stations, ventilation shafts and emergency exits may also be required for long sections of highway or rapid transit line tunnels. Where possible, these "appendage" structures should be located to minimize interference with major utilities.

In certain instances the individual utility may present a unique situation that limits possible solution because of size or importance. Examples would be a large trunk sewer or major electrical or telephone duct line. Utilities on one section of the WMATA subway included a telephone trunk line with over 30 ducts containing thousands of telephone lines. This type of situation should be treated separately during the design stage with the utility owner and not left for the general excavation contractor to solve as best he can.

2.3.4 <u>Sewers</u> - Four categories of sewers can be considered: Sanitary Sewers, Storm Sewers, Combined Sewers, and Force mains.

Sewers are generally, with the possible exception of force mains, usually removed or replaced due to the high potential leakage and breakage problem if supported in place. Large diameter trunk sewers present a bulky and excessive weight problem, with extensive support structures required. Junction chambers are even larger and heavier with an added tri-axial restraint problem.

Sanitary sewers of small to medium diameter (6" to 12") are generally made of vitrified clay pipe (VCP) in short sections with relatively flexible joints and the same is true for storm sewers which are medium diameter reinforced concrete pipe (RCP) cast in short sections. Combined sewers can be either VCP or RCP with older large capacity types constructed of brick and mortar.

"Support-in-place" is an expensive and time consuming alternate since the sewer must be supported on closely centered points which must also provide adequate lateral restraint. This combined with the burdensome maintenance factor of constantly sealing leaks caused by vibration from the deck structure favors the use of temporary lightweight steel pipe sewers. Large structures such as manholes, junction chambers, and concrete box culverts also present an excessive weight and bulk support problem which adds extensively to the overall steel requirement in the temporary deck structure if supported. Temporary replacement by lighter structures is preferable.

The laying of a temporary sewer line utilizing large diameter steel pipe appears to be the preferred method of approach to this problem in the projects reviewed in this study. Connections are made by welding sections of steel pipe to provide the bends and junctions required. Sewer manholes can generally be eliminated by providing an opening in the top in the temporary sewer for maintenance during the construction period.

A unique method was employed in a shaft in the Washington, D.C. Metro System, for maintaining a large sewer line. Welded wire fabric was installed around in existing 36" RCP pipe and a 3"to 4" shotcrete layer applied around the pipe. This was supported from steel beams in the normal manner. This was only a short section of sewer, but application of this support method to longer lengths might be advantageous.

The replaced temporary sewers within the cut-and-cover excavation are supported and maintained during the course of construction and permanently restored with new material during the final backfill stages toward the end of the contract.

Several segments on the BARTD System in San Francisco along Market Street required sewers be relocated adjacent to the underground subway structures. The major factor contributing to this approach was that the gradient of the sewers was generally below the projected roof of the station structure. In addition, the maze of other existing utilities together with the economic and practical considerations of relocating to another street precluded any other solution.

2.3.5 Water lines - Water lines can be divided into high pressure mains and low pressure local lines.

Cast iron mains and secondary lines have been used for decades. As long as such a line remains in place, with the surrounding ground supporting it, its functional purpose is not impaired. Exposing the utility and removing the supporting ground however, introduces problems of support and leakage. In cut-and-cover work, the danger of a break in the supported water line presents a danger of flooding and damage caused by washouts. The presence of power lines within the work area adds to the problem if they become submerged.

Several alternatives have been utilized to minimize the dangers mentioned above.

a. On several sections of the BARTD system, in Oakland, California, cast iron mains were replaced with cement lined and coated welded steel pipe prior to contract award. This work was done by the regional utility agency and the mains were replaced only within the limits of anticipated cut-and-cover work. The

30" water main feeding the city of Alameda was temporarily bypassed under a sidewalk adjacent to a cut-and-cover section in Oakland until work was completed.

b. In San Francisco on the BARTD system, high pressure fire lines (400 psi) 18 to 24 inches in diameter were temporarily shut down within the limits of individual contract sections, for the duration of the contract and restored at the end of the contract. Continuous service was maintained through cross-connections to adjacent streets.

c. Contracts on WMATA, the Washington Metro System, generally include early permanent replacement of water lines with cement lined and coated ductile iron mains with mechanical joints which are supported and maintained during the course of construction.

d. Valves, hydrants and other appurtenances are maintained in place or replaced in kind during the course of construction.

e. Manholes are generally broken out, a temporary wood structure substituted during the course of construction and replaced by a new, permanent structure at the end of the contract.

PVC (polyvinyl-chloride) and asbestos cement pipe (ACP) are sometimes used as water lines, but these materials present the same problems as cast iron pipe. There is a degree of brittleness in these materials compared to steel pipe which substantially increases the possibility of damage during the construction stage. The increased liability does not justify the economic benefits of their use under these conditions. Permanent relocation adjacent to the construction area would be an alternate, but a more expensive solution. Large mains (24" and larger) should be relocated offsite if at all possible. The potential danger of breaking such a line due to accidental mishandling of construction equipment, and the resulting damage that could occur before the line could be shut down, far outweigh the cost of relocation.

When water lines are maintained within the cut-and-cover area, provisions must be made for lateral support particularly where corners and bends are subject to water hammer.

2.3.6 Electric power facilities - These facilities can be categorized into the following: 1) underground primary and secondary feeder ducts and conduits, 2) overhead primary and secondary feeder lines, 3) manholes, transformer vaults and pull boxes.

The first category generally includes multiple banks of vitrified clay, concrete or steel pipe ducts which may or may not be concrete encased. As in the case of sewer lines, the joints in the duct banks are, with the exception of steel pipe, very close together (2' to 3') and impose a continuous support problem. Duct banks up to 6 or 8 ducts are not as much of a problem as duct banks which can vary from 24 to 60 ducts, which constitute a substantial weight and bulk problem in support. Also, the fragile nature of the duct bank requires provision of lateral stability.

Utility power organizations follow underground procedures which must be understood by all parties concerned, especially the contractor. While performing the work of supporting the facility, workers are engaged in very close proximity to "hot" lines, whether primary or secondary, where the danger of a mistake can be fatal.

Prior relocation of duct banks whether on-site or off-site, can be a very expensive proposition. Usually both the owner and utility company prefer to avoid the extra excavation, backfill, disruption of traffic and cost and leave the problem to the contractor with provisions in the contract documents to quantify and qualify the work. In many instances the contractor has generally elected, or followed as directed, the support in place scheme. When damages do occur, and the facilities are unacceptable to the utility agency for restoration, the result is delay due to the installation of a replacement duct bank and pulling in of new cable, an expensive and time consuming procedure. Considering the fact that in most cases the old ducts and cables cannot be put out of service (i.e. abandoned) until the new is in place and working, the delay to the completion of the project could well be considerably more than the time to engage in prior-relocation work.

Determination of how to handle each individual duct bank is a matter of judgement where age and condition is the predominant factor. In several areas on the BARTD system, galvanized steel pipe (4") duct banks were installed prior to contract award, with concrete encasement placed by the contractor during the restoration phase of his contract.

On the Washington METRO System, the procedure of replace and maintain involves no relocation prior to contract award. If the condition of the duct bank facility is poor, the contractor is required to install a new duct system, while supporting and maintaining the existing facility. If the existing duct bank is considered supportable, replacement can be postponed until final restoration work at the completion of the contract. In many cases, replacement becomes mandatory during the course of the work due to conflict between the existing duct and the proposed underground structure. Most cables do not have sufficient slack at the vaults for more than minor movement of the ducts. Lateral or vertical movement of an entire duct bank increases in difficulty with size.

Overhead lines on the other hand, can be relocated with relatively little work unless new lines are required by statute to be placed underground. Overhead facilities in general should be cleared from the construction area if at all possible to facilitate the contractor's soldier pile or diaphragm wall placement operation. The operation can become very costly if much of the time is spent erecting and dismantling equipment to avoid the overhead lines. The danger of live high tension wires in close proximity to the boom of a crane should be avoided where pile placement operations are in progress.

Electric manholes, transformer vaults and to a lesser degree electric pull boxes, impose substantial support problems to cut-and-cover operations. Many of the older manholes are of brick wall construction, and for all practical purposes, unsupportable. The sheer weight and mass of these and concrete vaults make consideration of hanging support impractical. They must be demolished, while maintaining the contents within operable and undamaged, and a temporary wooden manhole built and supported for the duration of the cut-and-cover operation. A replacement manhole of reinforced concrete can be built during the restoration stage of the project. Where a traffic deck is required, manholes and vaults are supported directly from the deck beams. In some cases, manhole and vault roofs must be removed to accommodate the temporary deck structure.

Transformer vaults, because of sheer weight, should be removed from the cut-and-cover area. This can involve extensive rerouting of duct banks and feeders.

2.3.7 Telephone lines - Telephone facilities can be divided into ducts (or duct banks), and manholes. The problems encountered with these facilities are very similar to electrical facilities except that the low voltage and current normal to telephone facilities are not hazardous.

Duct and duct banks generally vary from single ducts to 48 duct banks of rigid conduit or ceramic type ducts. Very few, if any, have been encased in concrete.

Generally the choice of alternative maintenance methods of construction is a judgement decision. Some preference is given to breaking out the duct completely; replacing a duct containing cable with split plastic duct (ABS, acrylonitrile butadiene-styrene, or PVC, polyvinyl chloride) and empty ducts with new whole conduits. The duct bank is then either supported in place as is, or encased in concrete and supported. Other contractors on both the BARTD and WMATA systems have preferred to break out the ducts and provide a temporary timber box duct for the duration of the contract; restoring the duct bank at the end of the contract, utilizing plastic conduit. Still others supported the duct in place, strengthening the duct by reinforced concrete encasement or by strapping timber planks on all sides. This approach has not always been successful in preventing damage.

Manholes, depending on size, have two alternate solutions. They can be broken out, and a temporary wooden manhole provided for the duration of construction and a new manhole built during restoration. Alternatively the original manhole can be supported in place. Size, weight, and integrity of structure are the major factors in these decisions.

As with electrical facilities, conflict with the proposed underground structure may well dictate drastic revisions to the system within the cut-and-cover area, and the one paramount consideration is how long the splicing of the new cables will take. For a large duct bank containing thousands of lines, this period can be from 6 to 12 months or longer and present a major time conflict to the proposed construction. The problem will also exist if the alternative of remove and replace is chosen. Detailed prior investigation of all alternative methods is mandatory in this type of situation.

2.3.8 Gas lines - Gas distribution mains and services can be classified into large mains and local service feeders.

Like water mains, older gas mains are predominately cast iron, and the relocate, or replace and support techniques are generally the approach taken for this utility. The cast iron pipe is virtually unsupportable due to the potential leakage at joints which present a very severe explosion risk.

These cast iron mains and laterals are replaced with welded steel pipe which is wrapped with adhesive tape for rust prevention or coated with asphalt and taped. There will occur instances where a gas main can be taken out of service for the duration of the cut-and-cover project, with the service laterals fed by other nearby lines. This alternative should always be considered.

Because gas is heavier than air it tends to flow downward into depressions such as cut-and-cover operations, with the danger of asphixiation or explosion. Whenever possible these lines should be relocated outside of the work area. Where this cannot be done (at intersections, etc.) special care should be taken to prevent accidental breakage by construction equipment or traffic vibration. On the BARTD system, temporary wrapped welded steel pipe mains were installed at sidewalk level or below sidewalks outside of the excavation and restored in the street with steel pipe during the restoration stage.

The Washington Metro System generally follows the approach of construct new at the early stages and support and maintain for the duration of construction where applicable, or removal of the line within the site for the duration of the construction with replacement at the end of the contract.

2.3.9 Steam lines and other special utilities - Steam mains are, in most instances, cast iron pipe with mechanical joints, insulated, and concrete jacketed. Guides, anchors and expansion joints are provided at intervals along the main.

On one BARTD system project, a temporary main was installed by the cut-and-cover contractors, and a new permanent system installed at the end of the contract.

The location of the temporary line could become a problem if the cut-and-cover area is generally cluttered with utilities, since the steam line, for safety considerations, should be located so its proximity to the public and to other utilities will not cause damage due to a leak or break.

Expansion joints, restraining blocks and other appurtenances cause this utility to generally be a high-maintenance cost item during the construction period and the possibility of off-site permanent relocation should not be overlooked as an alternative solution. Other special utilities entail similar problems, as previously noted and each must be treated as a unique individual problem where they occur.

2.3.10 Municipal utilities and facilities - This general category includes street lighting, traffic and pedestrian signals, fire alarm and police communication systems, transit trolley lines, parking meters, and signs.

Street lighting, together with traffic and pedestrian signals, is generally required to be maintained throughout the course of construction with temporary facilities, by the contractor, and subsequently restored. The conduit feeders of these facilities are seldom salvagable and provisions are normally included in the contract to restore a completely new system at the end of the contract. This may, or may not, include new electroliers, traffic and pedestrian signals.

Fire alarm and police communication facilities can generally be permanently relocated off-site at the early stages of construction.

Parking meters and signs are normally removed and stored for the duration of the contract and restored to their original location during the restoration stage.

Existing street car and trolley overhead power lines and tracks within a cut-and-cover construction area, such as those encountered on several segments on BARTD on Market Street, in San Francisco pose a major problem to efficient construction. In this case, the contractors had to live with the problem, since the alternative of relocating the street cars and trolley buses off-site was prohibitively expensive and traffic-wise practically impossible. Constant disruption of contractor's slurry wall and pile placement operations, as well as a negative effect to the efficiency of contractor's overall activities, made these sections more expensive and time consuming than those without this particular problem.

2.3.11 Summary -

B. Maintain, Support, Replace Utilities
   1. Types of existing utilities in site area
      a. Sewers, incl. catch basins, manholes

b. Water lines; mains, local, lateral

c. Gas lines: mains, local, laterals

d. Electrical and telephone conduits, including duct banks, junction and splicing chambers, etc.

e. Special lines, such as steam lines

f. Light standards and traffic signals

g. Electrical and telephone poles and overhead wires

h. Trolley wires and supporting poles

2. Alternate methods of handling existing utilities

   a. Support in place over excavation

   b. Replace with temporary utility (lightweight steel sewer flume, wooden splice chambers, etc.)

   c. Replace with permanent utility (steel water line instead of cast iron, replace old pipe with new, etc.)

   d. Relocate temporarily within site area due to conflicts, etc.

   e. Relocate temporarily outside site area (under or over sidewalk, or parallel street).

   f. Relocate permanently in site area

   g. Relocate permanently outside site area

3. Constraints imposed by utility owners

   a. Private utility (gas, electric) may require doing their work on force account or their chosen contractors.

   b. Utility may require upgrading their utilities even though existing facility may be in reasonably good condition.

   c. Local statute may require new electric or telephone lines, replacing overhead lines, be placed underground.

4. Constraints imposed by type of structure

   a. Depth of structure may require relocation of utilities. This may cause particular problems with large sewers that cannot be raised.

b. Portions of structure above general roof level (vent structures, stairways, etc.) may conflict with alignment of existing utilities.

5. Constraints imposed by construction methods

a. Continuous wall ground supports (steel sheeting and slurry concrete) require at least temporary relocation of all utilities crossing wall line.

b. Access to site for excavation and material handling might require relocating utilities that could otherwise remain in place.

c. Bracing and decking beam interference might require relocating utilities.

2.4 PROTECTION OF ADJACENT STRUCTURES

Structures located in the vicinity of a cut-and-cover project are likely to experience a certain amount of vertical and lateral movement as a result of subsoil deformations caused by the adjacent excavation.

These structures include private, commercial or public buildings, elevated roadways, retaining walls, etc. In urban areas churches, monuments and statues are likely to fall within the zone of influence of the excavation project. The amount of protection rendered such structures will depend on many factors and will vary from the simplest form of boarding up windows and removing excessive projections to installation of complete, expensive and permanent underpinning systems. It is imperative that all factors and conditions which may affect the type and scope of required structure protection are determined in the early stages of contract planning and design, as this work has to be substantially complete before general excavation can begin.

A complete geotechnical study must be undertaken to determine the expected behavior of the area subsoil during cut-and-cover tunneling operations. The relationship of lateral yielding of the ground support system to vertical settlement of adjacent ground must be determined. This evaluation shall include, but not be limited to, the rate and magnitude of differential settlement that may be experienced at various locations away from the bulkhead. Also, the effect of area dewatering must be studied, in order to determine if this will cause additional settlement as a result of increased overburden pressure, and possible transport of fine soil particles.

2.4.1 Factors affecting protection of adjacent structures - The type of ground support system to be employed on the project is a major factor in determining the magnitude of required protection of adjacent structures. Lateral deflection of the excavation support will result in both horizontal and vertical movements of the wedge shaped volume of soil directly adjacent to the bulkhead wall. Reduction or elimination of this so-called "zone of influence" or "zone of failure" will reduce, if not obviate, need for protection of adjacent structures.

a. A rigid bulkhead, properly installed and braced, will permit little or no yielding of the retained material, which will leave the in-situ soil pressures within the adjacent soil mass virtually unchanged. Hence, the zone of influence for a reasonably rigid diaphragm wall will be limited to an area directly behind the bulkhead, if in effect it exists at all.

b. In the case of steel sheet piling or soldier pile and wood lagging support systems, the amount of bulkhead deflection will generally exceed that of the diaphragm wall. Therefore, the zone of influence will extend a greater distance behind the excavation wall for these systems.

c. The extent of the zone of influence, and the magnitude of soil deformation within the zone, is also dependent on other inherent factors for "flexible" wall systems.

c.1 Workmanship plays a major role in the soldier pile system, as improperly installed lagging, combined with the lack of backfill will result in aggravated soil movement. Workmanship is also important in steel sheet piling installations, as sheets driven out of interlock, will permit the inflow of retained material, if remedial measures are not properly undertaken.

c.2 The frequency of bracing levels will dictate the relative stiffness of the bulkhead wall. A tightly braced excavation, using a flexible system, can in effect achieve characteristics similar to the rigid diaphragm wall. Hence, it is recommended that distances between bracing levels be reduced on all bulkheads in the vicinity of important structures, even if that structure is underpinned, in order to minimize the horizontal soil movement in surrounding ground.

c.3 Overexcavation below the required limits to facilitate easier removal of material before installing subsequent bracing levels will tend to increase bulkhead deflection. Thus, particular attention should be paid to the extent and depth of excavation in areas where wall movement is to be kept to a minimum. One popular method of overexcavation is to cut a slot in the middle of the trench to permit unrestricted travel by equipment after bracing members are in place. This mode of staged excavation would be permissible as long as side slopes are deemed adequate to provide required lateral restraint at prescribed intermediate excavation level.

d. The combination of a conservative design and good workmanship in the installation of a bulkhead will tend to reduce deformation, with resulting benefits in the form of reduced soil movements and a lesser need for protection of adjacent structures. In certain instances it may be deemed economical to design the bulkhead for at rest (in-situ) pressures rather than the more common active pressures, in order to achieve a stiffer and less yielding retaining structure and thereby reducing the protection requirements.

e. The extent of the zone of influence will vary depending on the relative stiffness of the ground support system and on the geological make-up of the retained soil. Given average conditions of both, the zone of influence generally has negligible projection behind the rigid diaphragm wall, i.e. the soil is not disturbed outside the limits of the excavation. The extent of influence at ground surface behind a flexible bulkhead will be approximately equal to the depth of the cut. Conservatively, a practical maximum limit for extent of the zone of influence adjacent to flexible bulkheads would be a horizontal ground level distance of two times the excavation height with an approximate minimum distance of one-half the excavation height. As stated above, unusual soil conditions could make these limits invalid, thus the need for a thorough geotechnical investigation is once again emphasized.

f. Dewatering, within an excavation retained via a pervious bulkhead, can affect the surrounding area and may cause widespread subsidence if the original ground water level is substantially lowered. On the other hand it may have only limited or localized effect, and may not be a factor in determining the extent of protection required for adjacent structures. In any case, the full impact of this phenomenon shall be properly investigated by the appropriate agency. In certain cases, when an impervious bulkhead wall is used, and a high water table exists, localized dewatering may have to be undertaken at adjacent structures, in order to permit installation of protection systems beneath the foundations, e.g. underpinning. This dewatering, if required, would be of such limited scope, that subsidence problems generally would not be a serious factor.

g. Many factors affect the method and scope of protection that may be selected. In urban areas, the most frequent structures are buildings, hence most of the factors discussed herein relate to building structures, but some of the factors are applicable to other forms of construction, including churches, monuments, statues, and elevated roadways.

g.1 The size of the building is of great importance, as it dictates the foundation load and the associated surcharge load on adjacent bulkhead. Also the protection scheme must be compatible with the size of the structure, e.g. a system, which during installation, requires a substantial reaction from the building foundation (jacked piles) can not be implemented for a light structure.

g.2 The use of the building and its basement must be taken into consideration as it will dictate the working area that can be made available to the contractor. Many times this factor alone will decide the method of protection to be employed. If the building lacks a basement, all work generally will have to be accessed from the outside.

g.3 The importance of the structure must be evaluated in order to assign it a priority. As a rule, any building three stories or less in height is often classified as an unimportant structure, and as such will generally only receive a minimal amount of protection. In specific areas, depending on the value of structures, it may be more economical to either purchase and raze the structure or accept certain temporary damage that can be repaired at conclusion of the contract. Such damage often is limited to a cosmetic nature for small lightly loaded structures, and would have no effect on the structural integrity of the building.

g.4 For a major building, the structural properties must be carefully evaluated. This can be accomplished by site visits and by examining existing building plans. An important factor is to determine the building's tolerance to movement and differential settlement without developing structural damage. Steel frame structures have high displacement tolerance, but unequal subsidence may produce devastating effects on rigid masonry structures.

g.5 Of additional importance from a structural standpoint, in determining scope of protection, is the depth and type of foundation supporting the building. Buildings with very deep foundations may be founded below the zone of influence, thus not requiring any support, e.g. multi-story basements and pile and caisson foundations. Buildings that have poor foundations or those where extent of foundations are unknown will require special treatment. This may include removal and replacement of footings or repair to original foundations. In certain cases, several protection scheme alternatives may have to be considered.

g.6 Certain structures may have religious or historical values which generally precludes any permanent alteration.

g.7 The location of sidewalk vaults and utilities that may enter the basement must be noted so that proper support can be evaluated. Sidewalk vaults located in transit station areas are often demolished during general excavation. Hence, these vaults can serve a dual purpose during support operations; access holes can be cut as required, as long as pedestrian traffic is maintained, and vaults can serve as temporary storage areas for excavated material from underpinning schemes.

h. Ultimately, the location of the structure with reference to the ground support system becomes the major factor in determining the need and scope of protection. Although soil movement generally is confined to the zone of influence, some nominal displacement may occur beyond this limit. Hence, certain large buildings may have one portion within the zone and another outside the area of expected settlement. This will produce a unique problem, whether to support only a portion of the building or to provide the entire structure with support. This analysis will have to take into account the expected settlement within the zone of influence, possible subsidence beyond the zone and the structural ductility of the building towards differential settlement.

i. A detailed preconstruction survey is mandatory for all buildings which may need protection. This survey shall ascertain all pertinent facts of pre-existing conditions so as to dispel all doubts of the origin and timing of damage that may occur after construction has commenced. Permits for right-to-enter and to perform the protection work shall be applied for well in advance of project start, as they may be difficult to obtain should legal disputes arise. Building owners should be made well aware of potential protection work and space requirements to perform the work. A clear and precise proposal will tend to dispel much apprehension and will undoubtedly result in an earlier approval.

2.4.2 <u>Types of protection systems</u> - Although underpinning may be the most common form of protection, other methods can be used. Protection systems are considered by six major categories as follows: Underpinning, Contingency Support, Ground Consolidation, Maintenance of Ground Water Level, Rigid Ground Support and Miscellaneous Building Support and Protection.

a. <u>Underpinning</u> - Underpinning is defined as a permanent construction designed to transmit foundation loads directly through the underpinning to such lower level, below the zone of influence, as is necessary to secure the underpinned structure, and which will relieve the adjacent ground and structure from undue lateral pressures. Several variations of underpinning exist, but they can be grouped into two major types, i.e. pile type and pier type. The pile type is used when depth of penetration becomes excessive for hand-mining operations (greater than 35 to 40 feet (10 to 12 m) and where groundwater is likely to be encountered prior to reaching required penetration. Underpinning by means of the pile method will in effect alter the foundation characteristics from a spread footing to a pile footing. All of the underpinning systems described below can be incorporated in the bulkhead system, if location permits and adequate bending strength is provided to resist the increased lateral loadings.

a.1 Jacked pipe piles are installed under existing footings. Small tightly braced, jack pits are dug under the footing in a segmental sequence, so that only a small portion (25% maximum) of the footing is undermined at any one time. In certain buildings, due to size of footing or composition of subgrade material, temporary support may have to be provided prior to excavating beneath the footing. Temporary support is a method for removing column or wall loads from their foundations and carrying these loads on a temporary framework to alternate footings, which have been especially installed for this purpose. If such support is provided the entire footing may be exposed at one time, eliminating the need for the sequential jack pit excavation, with associated cost savings, and offsetting in part the cost for providing the temporary support system. In the jack pit, short pipe sections, 4 to 6 feet (1.2 to 1.8 m) long, and generally 12 to 14 inches (300 to 350 mm) in diameter are jacked open ended using the underside of the footing as reaction. Splices in pile segments may be sleeved type or butt welded. The pipe pile is partially cleaned periodically during installation and completely upon reaching required depth and subsequently filled with concrete. Piles are load tested and prestressed against the underside of footing. Completed pile clusters are encased in concrete to form an integral part of the existing foundation. Again it must be emphasized that this method is only applicable for buildings where footings can resist a thrust of approximately 100 tons, as the design load for each jacked pipe pile will vary from 50 to 80 tons. These piles are generally designed as end bearing elements.

a.2 Small diameter pile clusters is another form of pile underpinning. The installation method for these piles is different from that of the jacked piles as the small diameter piles are generally installed from above and through holes cored in the footing, thus eliminating the costly operations of excavating the jack pits and installing temporary support where required. They can be either driven pipes, 2 to 6 inches (50 to 150 mm) in diameter, or drilled and cast-in-place with approximately the same diameter as the pipe option. After the pile is completed and load tested it is bonded to the footing via the cored hole. As each pile has a limited capacity in the range of only 2 to 10 tons, a substantial number may be required to support the total foundation load. Although this underpinning method has obvious advantages over the conventional jack pile method, it has several limitations: the piles have low individual capacity; the method requires penetration of existing footing; and the piles are basically friction piles with virtually negligible end bearing capacity. Penetration of footing can be obviated if new footing is constructed, but then a temporary support system may have to be employed. Also, the subsoil below zone of influence may be unsuitable for friction piles, requiring the use of the end bearing jacked pipe piles. These small diameter underpinning piles are at present mostly used on European projects, partly due to a lack of exposure in the U. S. and partly due to building code restrictions in this country.

a.3 Caissons or piles, in combination with needle beam supports, can be used as a form of underpinning. In this method the caissons or piles are installed adjacent to footings and tested to required capacity. Foundation load is then transferred to the caisson or pile members with a steel or concrete support beam grillage, i.e. needle beams. This method is generally used when a portion or all of the transit structure is to be located beneath the building in question and which would result in interference between the transit structure and underpinning piles placed directly under existing footings.

a.4 Bracket piles can be used as an alternate form of underpinning. In this scheme the supporting piles are installed outside the footing. Foundation loads are carried to the piles through cantilevered brackets. This method is generally only applicable to small and lightly loaded structures and due to difficulties in installing these piles within the building, the method is rarely used when more than the front row footings need to be underpinned.

a.5 Concrete pier underpinning is installed by excavating tightly braced pits under the footing to the required subgrade. The most common plan dimension of these pits is 3 x 4 feet (1 x 1.2 m) with a 1-foot (300 mm) plus or minus variation in either dimension. If the footing is large, the pits are installed in a segmental order much the same as described for the pile pits (see above). As each pit excavation is completed, the pit is filled with concrete, usually unreinforced, to within about 3 inches (75 mm) of the underside of the footing. Piers are prestressed in this space using a non-shrink grout (dry pack) or steel wedges or a combination of both. Individual piers are keyed together to form a larger pier, generally of the same size as the existing footing. Pier underpinning is used where depth of underpinning can be limited to 35 to 40 feet (10 to 12 m) and where groundwater will not interfere with the pit excavation. If the above conditions can be adhered to, this method is generally less expensive than either of the previous underpinning schemes. However, this method is perhaps the one which is the most dependent on good workmanship, and it does have the same requirements for temporary support as outlined in 2.4.2, a.1.

b. Contingency Support - Contingency support or construction underpinning can be provided in buildings where the likelihood of differential settlement can not readily be assessed or where settlement may depend on the contractor's bulkheading methods. Contingency support is also only used when the magnitude of the expected subsidence is less than 2 inches (50 mm) and it is generally only installed in buildings with good, or at least moderate, tolerance to unequal movements. This method is a corrective scheme rather than a positive system such as underpinning. It should be pointed out that although the use of the contingency support is temporary (during construction), the facilities used in this scheme may become a permanent part of the supported structure. Many variations of contingency support exist. A few basic systems are as follows:

b.1 Control Jack Piers are concrete piers, placed under the footing and used as a reaction to monitor the position of the foundation. This is often accomplished via hydraulic jacking between footing and pier, allowing the pier to subside as required. At completion, the space between pier and footing is permanently filled in the same manner as described above for pier underpinning. The control piers are usually only 5 feet $\pm$ (1.5 m$\pm$) deep and with a plan area of approximately 2 x 4 feet (0.6 x 1.2 m).

b.2 Column/Wall pick-up is performed in much the same manner as described for jack piers, with one basic exception; the column or wall is separated from the footing and the vertical adjustment (if required) is made between the column or wall and the footing, allowing the footing to settle, but maintaining the rest of the structure at original elevation. This system requires the installation of reaction brackets on either columns or walls. When it is deemed that monitoring is no longer required the column or wall is again connected to footing by filling possible opening with appropriate materials and subsequently the reaction brackets are removed.

b.3 Pressure grouting is another technique for maintaining the foundation of a structure. In this method, grout pipes are driven to appropriate depths into the subsoil beneath the footings and then activated as required. This system can only be used in combination with appropriate soils. A cohesive and impervious subsoil does not readily permit grout penetration, hence it would render this method ineffective.

b.4 It should again be emphasized that the contingency support schemes are corrective measures which are implemented after a specific amount of settlement has taken place. Thus, producing a certain amount of "racking" of structure, which it must be capable of withstanding.

c. Ground Consolidation - Ground consolidation can be provided within the zone of influence and thereby stabilizing and reducing the amount of deformation of this soil mass.

These techniques are generally limited in their effectiveness to provide protection of adjacent structures and are quite restricted as to proper application (see above). However, when used correctly they can be compatible with other cut-and-cover operations, e.g. excavation and ground control. These ground consolidation methods are discussed in Section 2.5. They are all related to geotechnical procedures and include three basic methods: grouting (both chemical and cement), freezing techniques and surcharge load (to provide additional confining overburden pressure).

d. Maintain original ground water level - The effects of this method have been discussed above and is also covered in Section 2.5.

e. Provide rigid ground support - This method for providing protection of adjacent structures is directly related to the bulkheading procedures and was discussed in Section 2.4.1. Although ground support is considered elsewhere, it is included here as the surcharge load from an adjacent structure is supported by the bulkhead wall in the form of increased lateral loading.

f. Miscellaneous building support and protection - These methods of protection are less drastic in nature than those described above and little, if any, permanent alteration is required. The procedures covered under this heading are usually employed when only nominal (less than ½ inch (13 mm)) settlement is predicted. However, they may be implemented in combination with any of the above techniques as a cautionary measure. The methods, which are all quite basic, are: strengthen and shore building, protect store fronts and board up windows, remove canopies, cornices and all other interferences, remove or support sidewalk vaults as required, etc. The above items may be undertaken singularly or in any combination that suits the particular structure in question.

g. Instrumentation - In order to achieve a successful program for protection of adjacent structures, a detailed monitoring system must be maintained. Settlement markers shall be placed on all structures within at least 300 feet (100± meters) of project excavation limits. These markers shall be in place well in advance of any construction, including, but not limited to, dewatering, pile driving, excavation and structure support work. Readings of these markers shall be made at intervals commensurate with the movement observed.

2.4.3 <u>Selection of protection system</u>

a. Structures can generally be classified into three categories:

<u>Category A</u> - Structures which will experience settlement regardless of bulkheading procedure employed (except for rigid diaphragm walls).

<u>Category B</u> - Structures which may experience settlement depending on bulkheading procedure employed and level of workmanship maintained during installation.

<u>Category C</u> - Structures which will experience no appreciable settlement regardless of bulkheading procedure employed (except in the case of a catastrophe).

Structures in <u>Category A</u> generally require some form of <u>Underpinning</u>. The individual method can be selected by careful examination of the various options described above and their particular application to the structure in question and by thorough knowledge of the geotechnical conditions at the site. The extent to which underpinning is provided depends on the limits of expected differential settlement (zone of influence).

Structures in <u>Category B</u> can usually be adequately protected by methods described under <u>Contingency Support</u> or <u>Ground Consolidation</u>. The individual scheme shall be selected in the same manner as outlined above for Category A - Buildings.

Structures in <u>Category C</u> generally require only nominal protection and can, in most cases, be properly protected by items described under <u>Miscellaneous building support and</u>

protection. It is possible that many buildings that fall into this category may not need any form of protection. Local site conditions will determine this factor.

b. Protection of adjacent structures, especially underpinning and contingency support items, could, in certain cases, be performed under a separate contract. This work needs to be substantially complete before cut-and-cover operations, which may affect adjacent property, can commence. However, in order to let a separate contract for protection work, the scope of this work must be of such magnitude to justify a separation of the contractural responsibilities, and a definite saving in construction progress for the entire transit project must be realized.

2.4.4 Summary

C. Protection of Adjacent Structures

1. Factors affecting protection of adjacent structures
    a. Type of bulkhead
    b. Proximity of structure excavation
    c. Workmanship (on bulkhead installation)
    d. Bracing levels
    e. Overexcavation
    f. Type of subsoil
    g. Dewatering
    h. Size of structure
    i. Use of structure
    j. Value of structure
    k. Structural integrity of structure
    l. Depth of type of foundation
    m. Accessibility to foundation
    n. Sidewalk vaults
    o. Utilities
2. Types of Protection
    a. Underpinning
    b. Contingency support
    c. Ground consolidation

d. Maintenance of original ground water level
e. Rigid ground support
f. Miscellaneous building support and protection

3. Constraints imposed by type of transit structure
    a. Wider and/or deeper structure increases the amount of required structure protection.
    b. Structures which encroach on sidewalk areas necessitate removal of sidewalk vaults, signs, marquees, etc.
4. Constraints imposed by construction methods
    a. Protection of adjacent structures is more critical for flexible bulkheads than for rigid wall systems.
    b. Amount of deformation is dependent upon workmanship, especially for soldier piles and lagging scheme.
    c. Unrestricted or excessive excavation could result in wall deflections exceeding those predicted by design.

2.5 GROUND WATER CONTROL

Ground water control is not always needed; but when used, it should be adequate for the purpose. A timely, efficient installation will often eliminate costly delays during construction. A high water table does not necessarily mean an elaborate dewatering system is needed, but it does indicate that some degree of ground water control is required and additional investigation is warranted.

Geotechnical investigations should be made early so that an adequate dewatering system can be installed prior to start of excavation. Some ground conditions require weeks or months of pumping for dewatering preparation. It is preferable to perform the investigations in the pre-bid stage. If dewatering is needed, it is better to know it when determining project costs; if it is found to be unnecessary, the contractor need not include it in his bid as a contingency.

2.5.1 Factors that afford ground water control - The most obvious factor and the one most easily obtainable is the elevation of ground water. When drilling for earth samples the elevation of ground water should be noted when it is first observed. Later observations may show the water to be lower in the hole, because it was merely perched water, or higher in elevation due to artesian pressure. To note these changes is of help to the soils engineer; to fail to note them, may result in misleading information. It is also valuable to know the composition and permeability of the soil (Ref. 5). Other factors which affect the flow of water are the proximity of a ground water recharge source (lake, underground stream, etc.), the size in thickness and area of the aquifer, and the uniformity of the soil. While empirical equations have been derived for estimating ground water flow, they assume uniformity of permeability and thickness of an infinitely large aquifer, conditions which rarely exist. A competent soils engineer, familiar with these limitations should interpret results of tests and theoretical calculations. If it is suspected that dewatering may be required, additional investigations should be made including drilling observation wells and performing pump tests. The wells can be located so they can later be used to monitor the ground water level during construction.

There are site and construction factors which must be considered in selecting a ground water control system. In an urban area the nature of the ground and possible settlement effect on existing buildings may make dewatering undesirable. In constructing the BARTD system Civic Center Station, the contractor was required to use a cast-in-slurry diaphragm wall for ground support, and to recharge the ground water outside to maintain the original water level while dewatering within the cut. (Ref. 7). Such a requirement not only affects ground water control but also places limitations on the choice of ground wall support and the bracing system. Efficient operation of men and equipment in a cut-and-cover project normally will require reasonably dry working conditions regardless of other considerations. In a free-draining soil such as sand it may be sufficient to keep ahead of the major excavation with excavated trenches and sumps for dewatering. If the soil contains an appreciable amount of clay or silt, even a small inflow of water may be sufficient to bog down heavy equipment and impede or stop progress of the work. If this happens, the decision to install a dewatering system is already weeks or months too late.

Another consideration that can affect the choice of a dewatering system is the cost of labor. Conditions vary from high productivity with minimum labor requirements to low productivity with high labor requirements. In a few areas, maintenance of dewatering can be done on an as-needed basis, rarely requiring more than one shift a day. In other areas, union agreements require a pumpman on duty around the clock, seven days a week. On a project requiring dewatering for twelve to twenty-four months, this becomes a major consideration. In at least one city the labor agreements requiring an operator for each sump pump and well point system, have been broadly interpreted to apply to deep well systems as well. Such a dewatering system may have twenty to fifty wells each with its own pump thereby requiring twenty to fifty pumpmen. This interpretation of work policy has forced contractors in the area to use less efficient (for the particular ground conditions) ejector systems requiring fewer pumps.

2.5.2 Types of dewatering systems - The various ground water control systems in common use have been adequately described in previous works (see Ref. 1, 5, 6). They will be briefly discussed here, from a point of applicability to transit tunnels.

a. Trenching, sumping and pumping - This is the least expensive method of dewatering when used under ideal conditions. It is most effective in free draining soils with little to moderate flow. When used with soldier piles and lagging the system must have sufficient capability to drain the surrounding area. Inside a diaphragm wall it need only drain the excavated cut plus any leakage through the wall and piping water under the diaphragm wall.

b. Cut-off walls - While not a separate system this is useful when used in conjunction with other dewatering methods to limit the amount of water to be handled. The simplest method consists of lowering the bottom of the diaphragm walls to increase the length, and therefore resistance, of water travel under the diaphragm. An unbalanced head at the invert can cause a quick, boiling condition. It is preferable, when possible, to extend the diaphragm to an impervious layer. Another method consists of placing a continuous grout curtain outside and parallel to the ground support wall.

c. Wellpoint dewatering system - This system uses a common header and pumping system to which the individual wellpoints are attached. The wellpoints consist of pipe and screen 2" to 3" (50 mm to 76 mm) in diameter, placed 3' to 6' (1 m to 2 m) on centers around the perimeter of the cut. They are limited to drawdown of 15' (5 m) at about 3 gal/min (0.0002 m^3/s). Wellpoints are relatively inexpensive to place and operate and have been used quite extensively, mostly in shallow, sloped cuts. Although wellpoints can be placed in several vertical stages in a deep sheeted excavation they lose their advantage of economy and present problems of disruption of other work during installation of each new stage.

d. Deep wells - Used to draw down the water table in one stage, each well contains a submersible pump of 1 hp to 50 hp depending on anticipated flow requirements. The pump is placed near the bottom of the pipe in a perforated screen section. The pipe, in turn, is placed in a drilled hole sufficiently large to place 8" to 12" (200 mm to 300 mm) of filter sand around it. The wells are placed 50' to 200' (15 m to 60 m) on centers depending on the expected drawdown curve. This type of dewatering system is commonly used in relatively deep cut-and-cover transportation projects where heavy water flows are expected.

e. Jet-eductors (or ejectors). Using a pipe and header set-up similar to wellpoints, the ejector uses the Venturi principle to raise ground water in a single stage. A vacuum is created by forcing water under pressure down a pipe through the Venturi and up a second pipe, carrying with it 3 to 30 gal/min (0.0002 to 0.0020 m^3/s) of groundwater. Spacing varies from 3 ft to 10 ft (1 m to 3 m). This system is most efficient for a medium to deep excavation with relatively light water flows.

f. Dewatering aids - Additional ground water control methods are available to the contractor in conjunction with the systems listed above, for special local problems.

Sand drains may be used in areas of stratified soil consisting of varying permeable and impervious layers. Filter sand is placed in vertically drilled holes to aid travel of water between pervious strata, reducing the number of wells that would otherwise be needed.

Electro-osmosis can be used with a wellpoint system for dewatering difficult fine clays and impervious silts. The system is based on electric reduction of water to its basic components of oxygen and hydrogen by use of electrodes placed in the ground. Anodes consisting of any convenient metal conductor shape are placed alternating with wellpoint cathodes containing a suction pipe. While attractive in theory, the system has serious drawbacks and is rarely (if ever) used for transportation tunnels in an urban area. Unpredictable secondary effects include soil swelling, possible reduction of soil stability, consolidation of adjacent soils and increased corrosion of nearby steel support piles. In addition, power requirements are high and time required for reducing the water content comparatively long.

g. Compressed air - While used successfully for driving soft ground tunnels, its application to excavation from the ground surface has been limited to small shafts and caissons, neither of which are applicable to long cut-and-cover sections or busy urban areas where traffic must be maintained. While it is possible to use compressed air to excavate under a previously placed roof (see Ref 1) the high labor costs of compressed air work alone would rule this out for serious consideration in the United States except for possible isolated sections with unusual considerations. Using compressed air with under-the-roof construction combines all of the problems of compressed air tunneling without the offsetting advantages of being able to use a shield or soft ground tunneling machine.

h. Soil consolidation - The use of soil consolidation can eliminate dewatering by making the soil in the vicinity of the tunnel impervious to inflow from surrounding soil. This can be done by grouting or freezing and both techniques have been used successfully for shafts and special problem areas. It is doubtful, because of high costs that these can be generally considered for large cut-and-cover projects except, as in the case of compressed air, for isolated sections with special problems or considerations. A major breakthrough in price could make these alternates more attractive, but care must still be exercised in using either method under appropriate conditions only. Freezing on a large scale can cause ground swell, frozen utilities and other frost related problems. Grouting requires unusual skill and reasonably uniform, accommodating soil to be successful over a large area.

2.5.3 Summary -

D. Control Ground Water

1. Considerations for control of ground water
    a. Unbalanced head of water
    b. Permeability of soil and anticipated flow
    c. Proximity to ground water recharge source (river, bay, underground stream)
    d. Stratified deposits resulting in artesian pressures

   e. Possible effect on adjacent structures of lowering ground water table
   f. Requirements for reducing inflow due to excavation procedures
2. Options for control of ground water
   a. Lower water table by draining into excavation, sumping and pumping
   b. Use of cut-off wall to prevent boiling in bottom of cut (not applicable to soldier piles and lagging)
   c. Lower water table with wellpoints and headers
   d. Lower water table using deep wells
   e. Lower water table with jet-eductors
   f. Lower ground water using deep wells within excavation and recharge water level outside (not practical with soldier piles and lagging)
   g. Use of compressed air to keep excavation free of water (applicable only to caissons or below an airtight roof)
   h. Consolidate ground outside excavation by chemical grout or freezing
3. Constraints imposed by type of structure - none; however, structures may require waterproofing and/or measures to prevent uplift.
4. Constraints imposed by construction methods
   a. High inflows of water can seriously impair excavation by rubber tired equipment.
   b. Existing water levels cannot be maintained with soldier piles and lagging support.
   c. Wellpoints are limited to about 15 ft of depth per stage.

2.6 INSTALLATION OF STREET DECKING

Decking over a cut-and-cover operation has many advantages. In addition to affording access directly for pedestrians and commercial and private vehicles it allows the contractor to place his equipment directly over any part of the project. The existence of deck beams simplifies the hanging support of utilities which would otherwise require special support members. By using removable slabs or pads it is possible to raise or lower material where required and to place concrete by gravity in many instances. The decking also minimizes the problems of noise and dust that bother nearby residents and the traveling public. It has but one major disadvantage; it is expensive. This often outweighs all of the advantages, and complete decking is used only where necessary.

For the purpose of this study, we can assume that the work involved in a major cut-and-cover project in an urban environment encompassing several blocks or more of a major street does necessitate the use of decking to maintain traffic flow through the site area. This section will consider the various options available to the contractor and the requirements and restrictions which affect his decision.

2.6.1 Requirements of the use of decking - the relative needs of decking are similar to those described in Section 2.2.2, the needs of traffic access to the site area before the placement of decking. Public use of decking, in order of importance, is pedestrian traffic, emergency vehicles, service and delivery vehicles, mass transportation vehicles and private cars. When pedestrian sidewalks are included on the decking, they should be separated from driving lanes by substantial curbs and barricades. Sidewalks and deck timbers should be coated with special non-skid material. Use of the deck by the contractor may be restricted to non-rush hours unless the deck is sufficently wide to contain both equipment and traffic. If possible, off street access should be provided to the work area below. Curb parking should be restricted to necessary deliveries and pick-ups during non-rush hours only.

Under circumstances where decking is required for the traveling public anyway, it should be planned for maximum possible use by the contractor's equipment, with a minimum of unnecessary interference to street traffic. The greatest need for deck space by the contractor is during excavation and for material handling. Excavation generally involves the use of a clamshell crane and dump trucks; material is handled by cranes lifting from flat bed trucks. Barricades and flagman may be needed to protect motorists during these operations. The design of the deck structure should be based on the most severe loading conditions likely to occur. Generally the loads imposed by construction equipment exceeds that of normal or even heavy traffic.

2.6.2 Option of temporary or permanent decking - The type of deck structure most commonly used is a temporary deck consisting of steel beams on 8' to 12' (2.4 m to 3.6 m) spacing with timber decking above. 12" by 12" (300 mm x 300 mm) timbers are often used for traffic portions of the deck, placed singly or in mats of 3 to 8 connected timbers. The steel beams may be 24" to 36" (0.6 m to 0.9 m) deep or more depending on the span, and are supported on the sides by the ground wall support system. For long spans the beams can be supported at or near the center by an intermediate row of soldier piles. This also aids in the placing of decking on half the streets at a time to reduce interference with traffic. Other types of temporary decking will be described below. The major advantages of this type of temporary decking are ease of building non-typical sections, accessibility to utilities and work area below, and possible re-use of materials in typical sections. Other advantages are relatively low material cost compared to a permanent deck and the convenience of needing only a "rough fit" to the existing street rather than the close tolerance required for a permanent structure.

Though it is not common in cut-and-cover work, it is possible to install a permanent deck supported by soldier piles or concrete diaphragm walls prior to starting general excavation. If support is by soldier piles, the load should be later transferred by columns or extended concrete walls to the structure. As in the case of temporary decking common to most projects, a center row of piles could be used to provide intermediate support. The permanent deck would be a continuous street bridge, and would have to be designed, constructed and maintained as such. It would probably be constructed as a reinforced concrete beam and slab deck or a composite steel beam and concrete slab deck. If steel beams are used they should be encased in concrete for fire proofing and for reducing maintenance costs. Pre-cast concrete members are preferable to cast-in-place concrete because of the long cure time required before traffic loads could be allowed on the cast-in-place deck.

The major advantage of a permanent deck would be reduced disruption to the street through the elimination of the need to dismantle the temporary deck and restore the street at the end of the project. It is likely however, that the initial disruption would be more protracted due to the need for greater precision in placing the deck. In a situation where there are no utilities to consider, or the utilities can all be permanently relocated, it is possible to combine the street and roof of the structure for additional saving of time and work. Where the roof of the structure is too low for this and utilities do exist, some fill can be placed on the roof to support the utilities. Access to utilities can be provided from the street, structure or building basements. Where the water table is high it could create a problem below a permanent deck. Extending diaphragm walls above the water table would help but might interfere with lateral connections to adjacent buildings. Other advantages of permanent decking are reduced backfill loading on the structure and ease of prefabrication of typical sections. The complexity of non-typical sections might preclude prefabrication, require special cast-in-place work, thereby cancelling out the potential advantages.

2.6.3 Other options of temporary decking - Various combinations of materials can be used in lieu of a timber deck on steel beams. Precast concrete, cast-in-place concrete or steel plate can be substituted for deck timbers. Concrete is considerably heavier than timber and cast-in-place concrete has a long cure time disadvantage. Steel plate decking is thinner than timber but more expensive. Timber is more adaptable for non-typical sections than steel or concrete and is easily removable for construction purposes. Concrete beams could be substituted for the steel beams, but are heavier and require special casting. Here again, cast-in-place concrete beams have the disadvantage of cure time to consider. Each of these options or combinations might prove an attractive alternative in special situations.

Where the deck is sufficiently wide, or when the traffic to be carried is restricted, it may be possible to leave an open slot or intermittent shaftways for ease of construction purposes and ventilation of work area. Usually this advantageous situation does not exist, and the contractor must do with temporary deck openings that can be recovered for traffic use.

In certain situations a contractor must provide movable traffic bridges to carry vehicles over an otherwise impassable opening. This is most likely to occur where temporary or permanent decking is to be placed across the full width without benefit of center support. This work is slow because of the small area where work can take place. The wider the opened portion of street, the heavier and more cumbersome the bridge must be.

2.6.4 Other factors affecting decking - Non-typical sections of structures affect ground support, bracing and decking structures. These might be caused by the joining of two line structures, curves, entrance structures, vent structures, access shafts, etc. Each of these mean special fabrication and fitting of deck beams and mats with resultant slowdown of progress. A structure with a single span or arch roof will usually preclude center pile supports. The resulting long spans for deck beams make these beams quite deep, heavy and more difficult to handle. Long spans mean more interference with traffic during installation and deep beams can create problems with existing utilities.

The need for construction equipment to occupy deck space has been mentioned previously. Any restrictions imposed by not being able to use the deck during rush hours must be considered in planning operations. It must be realized that the impact on certain operations such as excavation may raise the cost of this work considerably. If access directly from the street is unavailable during concrete operations it may be necessary to pump the concrete.

Cross streets pose a special problem when installing decking. If traffic must be maintained, it is necessary to construct the intersection one quadrant at a time. It may be more attractive to perform this work on weekends when reduced traffic can be diverted. Although it has not been tried in the United States an "Umbrella Deck" (Ref. 1) has been used for work on intersections in England. While this type of bridge might be too expensive for decking installation at one intersection it might be worth considering on a long highway or rapid transit line tunnel with several intersections the same size.

2.6.5 <u>Summary</u> -

E. Installation of Street Decking

1. Use of decking
   a. Pedestrians
   b. Emergency and service vehicles
   c. Mass transit vehicles
   d. Private cars - daytime
   e. Private cars - rush hour
   f. Contractors equipment
2. Alternate decking options
   a. Permanent decking
   b. Movable decking - to be used to place deck beams across full width, or under-the-roof construction
   c. Fixed (temporary) deck beams with movable deck mats (timber), or slabs (pre-cast concrete) or steel plates
   d. Fixed decking for part of deck with portions (slot or shaftways) left open - (where volume of traffic permits)

e. Fixed deck beams with some decking left open and movable mats or slabs elsewhere

3. Constraints imposed by type of structure
   a. "Appendage" structures (such as entrances) or wide structures in a narrow street may require decking of pedestrian walkways.
   b. Arch roof on structure may require decking to be installed for full width of street rather than staged side to side.
4. Constraints imposed by construction methods
   a. Room required by equipment for excavation, material handling, etc.
   b. Need of access at various locations for placing concrete for permanent structure

2.7 INSTALLATION OF THE GROUND WALL SUPPORT SYSTEM

In discussion of the many activities, procedures, methods and problems involved in cut-and-cover operations, the one single operation that is most critical from a technical and cost standpoint is the ground support system. In most projects the only operation that is more expensive is the construction of the permanent structure itself, where there is less opportunity for major cost improvements once the basic concept is developed, (see Ref. 1). As discussed in Section 2.10 on permanent structures, the most potential cost savings involve avoiding features that add unnecessarily to the cost without increasing efficiency. In June, 1970, at the Advisory Conference on Tunneling held in Washington, D.C. a report was presented on cut-and-cover construction, (see Ref. 10). This report presented the findings of questionnaire responses representing 380 cut-and-cover projects in 17 countries. The consensus of opinion regarding major features of construction "with problems still pending and improvements needed" rated "ground support" the highest priority.

It can be seen by the following descriptions of eight basic ground support systems that much thought has already gone into this important operation. It is one of the aims of this study to point the way a little more definitively, to potential future improvement.

Cut-and-cover construction contracts in both highway and transit applications generally permit a contractor to design the ground support system. Under certain conditions the contract may specify the type of ground support required because of special subsurface, ground water or building support considerations.

Minimum design criteria are usually specified in the contract documents. The criteria includes tabulated values of lateral earth pressures for both dewatered and non-dewatered sections, additional values for lateral pressures due to traffic and construction equipment loads, and methods for deriving building and construction surcharge loads. The criteria also includes design standards and allowable unit stresses for determining loads to be used in the design of the decking and the excavation support system. The support system is designed to support the temporary deck, be adequate to restrain earth pressures, and carry all surcharge loads, such as building, traffic, utilities and construction loads. The design should also provide for an orderly, staged removal of bracing to satisfy the sequence of concrete placement, without overstressing the ground wall support members during this procedure. This is occasionally a major factor in determining support member sizes.

The design criteria establishes standards, sufficient to permit construction of the permanent structure in a safe and expeditious manner, with minimal movement or settlement of the adjacent ground, and to avoid damage to adjacent buildings, facilities, or utilities.

Several types of support systems are used, employing various techniques and materials, but these systems can be divided into flexible and semi-rigid wall systems. The degree of elasticity and yielding is the major difference between the two systems. The semi-rigid wall systems incorporate strength and stiffness to minimize an inward lateral movement associated with general excavation. The lateral earth loads are usually higher due to hydrostatic pressures, since these systems are designed to be water tight and eventually become a part of the permanent exterior wall. Examples of the rigid system include diaphragm walls such as the reinforced concrete slurry wall system, and interlocking concrete piles.

The flexible wall systems allow for predictable deflections to occur due to inward movement of soil caused by readjustment of stress in the adjacent ground as excavation proceeds downward. The elastic deformations of the component parts due to these loads are generally of greater magnitude than those realized in the semi-rigid system.

The magnitude of tolerable lateral movement is one of the determining factors in the selection of the type of ground wall support system to be used. Other factors which influence the selection are the method of installation, the arrangement of the bracing relative to excavation and concreting sequences, the physical characteristics of the ground, ground water considerations, and the proximity of buildings to the excavation site.

The wall support installation sequence in a typical urban area generally begins with an exploratory trench, or series of pits, dug along the proposed wall support line. This is common for all ground support methods. The purpose of this trench is to remove the pavement and subgrade material, and to locate precisely all utility lines and laterals crossing the wall support line.

The following descriptions of various ground wall support systems will describe installation procedures, materials used, and major advantages or disadvantages of the systems.

2.7.1 Soldier piles and lagging - Rolled steel shapes or reinforced concrete sections are used as soldier piles, with the former most common. Lagging can consist of timber planks, precast concrete, pneumatically applied concrete (shotcrete), or steel plate. The most frequently used lagging material is timber planks because of their adaptability. Until about ten years ago, piles were usually driven with impact or vibratory hammers. Because of current noise restrictions, it is now common practice to place piles in pre-drilled holes. Piles can be driven the last few feet required below subgrade for a toe-in, or the hole made deeper and structural concrete fill placed below the invert. In either case the remainder of the hole is backfilled with lean concrete or sand from invert to subgrade. Lagging is placed concurrently with excavation with spaces provided for free drainage of ground water to prevent buildup of hydrostatic head, an important factor in the reduction of active loads. The advantages of this wall system are the ease of installation, low cost and minimum disturbance to utilities. Conflicts with utility location can often be avoided by shifting the locations of the piles while maintaining a maximum predetermined distance between piles. Lowering of the general water table, however, may result in excessive settlement in certain marginal soils, endangering nearby buildings. Installation of lagging in non-cohesive soils could present a problem, as would poor installation techniques which result in leaving voids behind the lagging. The lagging operation combines well with the excavation operation when organized properly. Proper procedure requires careful stripping of the ground between soldier piles for installation of lagging to prevent voids and eventual ground settlement. Any voids that are created should be immediately packed with inert materials. Loose and running ground conditions prove highly disadvantageous to this system, resulting in extremely slow progress. The work is dependent on skilled workmen and the productivity rate will reflect the degree of efficiency in the operation.

2.7.2 Steel sheet piles - Sheet piles are rolled steel shapes with continuous interlocks on the sides. These are driven sequentially to provide a relatively watertight, flexible, continuous steel sheet wall. The piles must be driven by either an impact or vibratory hammer, which may create a noise or vibration problem. The installation usually involves use of a template for horizontal and vertical alignment.

Any utility line crossing the sheet pile line would have to be relocated twice to maintain continuity, unless it can be permanently relocated (i.e., the first relocation moves the utility away from its original position so that the sheet pile can be installed, and the second relocation moves the utility back to its original location). No lagging installation is required, since the sheet pile system serves as a combined soldier pile and lagging system. The lateral earth pressures are higher than when using soldier piles and lagging due to the ground water surcharge and may result in deflection between support points, unless additional levels of bracing are used. The vertical or axial load carrying capacity is negligible and precludes use of a deck structure unless supplemental carrying components are added to the system. The passive resistance below the excavation invert, however, is high and a distinct advantage from a design standpoint. Proper cutoff length can prevent boiling of the bottom where a differential ground water head exists.

The most common problem for the contractor involves placement tolerance. The ideal situation would require the sheet piles in direct contact with a horizontal steel wale at all support levels, but due to placement tolerances, this is seldom the case. Consequently, the gaps between individual steel sheet piles and the wale must be packed or wedged with blocking of either hardwood or steel. This work is a high labor cost item and with wood blocking, periodical maintenance is required. The presence of boulders in the ground increases the problem of driving sheet piles and in some cases, may preclude its use.

2.7.3 <u>Closely spaced reinforced concrete piles</u> - This semi-rigid system incorporates circular or rectangular reinforced piling which can either be cast-in-place or precast. The spacing between piles is kept to a minimum, and ideally the piles are in contact along the entire vertical interface to form a continuous wall. This can be described as the concrete pile version of a steel pile wall without the interlocks. Piles that simply touch adjacent piles are called tangent piles; those that overlap by drilling part way into the adjacent pile are called secant piles. Ideally no lagging is required, but due to less than perfect placement which normally increases with depth, work may be required to seal gaps between piles. Placement of the piles is generally on a good production cycle where alternate piles are placed in sequence followed by placing of the intermediate ones. The problem with utilities is similar to that encountered with sheet piles, usually involving double relocation. The system is more adaptable to relatively shallow cuts up to 30 to 35 feet (9.1 m to 10.7 m) and would necessarily become massive in deep cuts due to the higher loads encountered. The transfer of loads at the support points reflects the differences in allowable stresses for concrete and steel, and could result in very large connections.

An efficient, economical ground wall support of cast-in-place tangent piles has been used successfully in constructing the Edmonton, Alberta transit system. The downtown area has ideal ground conditions for such a system. The overburden consists of a firm glacial till with a permanent water table far below the track invert, permitting drilled holes to remain open without casing until filled with reinforcing and concrete. Since the system is not watertight, it cannot be used to full advantage where there is a high permanent or periodic water table.

2.7.4 <u>Soldier piles with cast-in-place reinforced concrete</u> - This system includes many variations in methods of placement but basically consists of installation of a permanent reinforced concrete wall poured between pre-placed soldier piles concurrently with the progress of excavation. The walls are poured in vertical segments or lifts which do not exceed the stand-up capacity of the soil, making it unsuited to loose, granular soil conditions. Pre-fabricated sections of reinforcing steel are placed between piles and may be tied to the piles for continuity prior to concreting. A variation in the sequence would be the installation of lagging behind the outside flange of the pile in marginal soils to permit casting of the reinforced concrete between the flanges of the soldier piles. Although this system results in a relatively permanent wall system completed simultaneously with excavation, the low passive resistance below excavated level together with the build-up of hydrostatic head behind the wall could well produce lateral loads which would make this system impractical for deep cuts. Progress of this portion of a project would be comparatively slow, although the overall time required for the project may be reduced due to the completion of the permanent wall during excavation. Due to normal pile setting tolerance variations in this type of work, a finish wall is necessary if the structure is to be used by the public (i.e. rapid transit station). The possibility of loss of ground in non-lagged areas and the danger of run-in under the walls as well as potential invert heave make this scheme riskier with increasing depth. Leakage may be a problem since the construction joints between lifts are difficult to seal.

2.7.5 Pneumatically applied concrete (shotcrete) walls - Pneumatically applied concrete has, through technological improvements, become a competitive alternative to the conventional materials in use in support systems. Shotcrete lagging, combined with either steel or reinforced concrete soldier piles, has been used in shallow cuts for building foundations. The application is practical to even deep cut-and-cover projects under ideal conditions. Although similar to the soldier pile and timber lagging wall support method, a distinct advantage is the improved contact of the gunned lagging to the soil and the subsequent elimination of voids, which is always a potential problem with timber. Although comparable in production rates with installation of timber lagging, the costs are higher. The installation procedure involves an application of an initial layer of shotcrete, installation of wire fabric reinforcing, and subsequent application of a final coat. Provisions for relief from ground water build-up can be accomplished by leaving slots or weep holes at intervals. Ground stand-up time is a limiting factor with shotcrete as it is with cast-in-place concrete. The system can also be used incorporating shotcreted wales and tiebacks, although practical limitations of depth would necessarily restrict this alternate to shallow and medium depth cuts.

2.7.6 Cast-in-place reinforced concrete slurry wall - The reinforced concrete slurry wall system is a semi-rigid wall system which provides a continuous reinforced concrete wall without soldier piles. Alternate slots are excavated along the wall line, utilizing bentonite slurry, more commonly known as driller's mud. The slurry, which is a stable suspension of powdered bentonite in water, is used to keep the excavated slot stable and prevent sloughing. The slurry replaces the excavated material until the concrete is poured by tremie methods. The installation procedure requires equipment for slurry mixing, circulation and cleaning, together with special slot excavating equipment, resulting in high initial cost. The excavated slot is narrow, 1' to 3' x about 20' long (0.3 m to 1.0 m x 6 m) and can be quite deep, 80' (24 m) or more.

Preassembled reinforcing steel cages are lowered into the excavated slot, and concrete is tremied into the slot, displacing the bentonite slurry.

Once the initial alternating slots have been concreted, the remaining space between the completed sections is excavated, reinforcing steel installed, and tremie concrete placed in the same manner. Various key and waterstop configurations have been used to make the connection watertight. Embedded items such as bearing plates for wale attachment or pipe segments for tieback installation can be attached to the reinforcing cages prior to concreting to facilitate later installation of the bracing system. The cost of the system is comparatively higher than other systems, although the completed wall does have the advantage of being available for incorporation in the completed structure. Production is dependent on skilled personnel and is considerably slower than the soldier pile and lagging system. The system will, if carried below proposed invert, create good passive resistance below excavated level. The system can be considered watertight if vertical joints are adequately sealed.

The presence of utilities involves additional relocation costs due to the continuity of the wall system. Supplemental structural components would be needed at the surface if a traffic deck is required.

2.7.7 Soldier pile and tremie concrete wall - The soldier pile and tremie concrete system is a semi-rigid system that provides a continuous structural wall consisting of soldier piles spaced at predetermined intervals with cast-in slurry concrete wall panels (usually unreinforced) between the piles. The wall thickness is equal to the depth of the soldier pile section, normally a rolled steam beam. The construction procedure starts with the placing of alternate soldier piles in predrilled holes, kept open with bentonite slurry. Vertical alignment of the pile section is critical in this system to avoid too long a span for the unreinforced concrete. The hole is backfilled with lean concrete, which displaces the slurry. Slots are then dug between the soldier beams, again utilizing slurry. A special digging bucket is used and the sides of the soldier beam serve as a guide. Tremie concrete is placed in the excavated slot. An alternate method is sometimes used where every other pile is placed in drilled holes and a double length slot is excavated. The center pile is placed in the slot and tremie concrete is used to fill the double slot using two tremie pipes. Care must be exercised to raise the levels of concrete simultaneously to avoid dislocating the center pile.

Initial costs are high due to the slurry equipment requirements, and installation is slower than other wall systems. The completion of the permanent wall is an advantage when constructing the permanent structure. The wall is thick, 2' to 3', (0.6 m to 1.0 m) and rigid, resulting in low inward deflection and ground settlement which can reduce underpinning requirements. The system creates passive resistance below excavated level. It is comparatively watertight, which reduces the cost of ground water control. Qualitative control and accuracy is required at all stages of the work, and the system is dependent on skilled workmen. The continuity of the wall system, requires complete relocation of all utilities which cross the wall line. The poured-in-place feature eliminates voids between the wall system and the surrounding soil, although the inside face of the wall is rough and will require a facia wall and localized repair work due to irregularities.

2.7.8 <u>Precast concrete segments placed in slurry trench</u> - Several variations of a basic precast wall system have been used successfully in Europe, since 1970 under the patented "Panasol" system of Soletanche in France. The precast wall can vary in configuration and size and includes tongue and groove continuous paneling and precast tee beam and panel combinations. The wall excavation procedure is similar to the cast-in-place slurry wall system. A trench is excavated through a special slow setting grout slurry. Upon completion of the slot, the precast panels and component beams are lowered into the trench, and aligned, and the grout allowed to set. The grout slurry is an important component of this system since the segments do not completely fill the trench as in the case of the cast-in-slurry walls. The setting of the grout which is equal in strength to the surrounding soil, assures elimination of voids and filling of all irregularities in the trench, thus minimizing potential settlement in the adjacent soil. The predominant use of this system to date has been in relatively shallow to medium depth excavations utilizing tieback bracing. The wall system appears to be readily adaptable to the use of wales and struts which may be required for deep cuts. This system has many advantages for incorporation into the completed structure as compared to a cast-in-place diaphragm wall. Since the sections are cast in a yard, better control and uniformity of concrete can be achieved. There is no concern that reinforcing is coated with slurry. A good finish can be cast on the inside face eliminating the need, in most cases, for a finish wall, and waterproofing can be placed on the outer face. Dowels, keys, recesses and bearing plates can be incorporated into the casting as required.

One of the major disadvantages of this system is high initial cost. To determine whether or not the advantages can outweigh this additional cost is one of the objectives of this study. In deep excavations, the length and weight of the sections can become a serious consideration in transporting and handling in a crowded urban environment. Some of the possibilities for reducing this problem include: (1) placing hollow segments which are subsequently filled in place, (2) use of horizontal joints to reduce individual segment size, and (3) use of sections that are more efficient for carrying loads than simple rectangular panels. None of these solutions are simple; horizontal joints, for instance, have to be spliced for moment continuity; but they do point to possible future improvements.

2.7.9 Summary -

F. Install Ground Wall Support

1. Considerations in choosing type of support
   a. Initial cost of installation
   b. Time required for installation
   c. Cost impact on other operations (underpinning, excavation, permanent structure, etc.)
   d. Number of bracing levels required (comparative)
   e. Incorporation of walls in permanent structure
   f. Effect on utility maintenance
   g. Dependence on soil standup time
   h. Compatibility with dewatering requirements (including cut off walls)
   i. Dependence on skill of workmen

2. Methods of ground wall support
    a. Soldier piles and lagging
    b. Steel sheet piles
    c. Closely spaced concrete piles
    d. Soldier piles with cast-in-place reinforced concrete
    e. Sprayed-in-place shotcrete walls
    f. Reinforced concrete walls cast in slurry trench
    g. Soldier piles and tremie concrete cast in slurry trench
    h. Pre-cast wall segments placed in cement slurry trench
3. Constraints imposed by type of structure
    a. Appendage structures outside the general excavation line (entrances, vents, etc.) may necessitate complex ground wall supports.
    b. Depth of structure affects size and shape of supports needed.
    c. Shape of structure (i.e. arch) may negate advantages of a slurry concrete wall being incorporated into completed structure.
4. Constraints imposed by construction methods
    a. Some systems more compatible with tie-back bracing than others
    b. Continuous wall systems more affected by utility maintenance than others

2.8 BRACING OF THE GROUND WALL SUPPORT SYSTEM

Bracing systems are a portion of the overall ground support system, in conjunction with the wall support. The contractor is usually responsible for the design of the bracing system. Compatibility and integrity between the wall and bracing system require this approach. Usually, more than one type of bracing system can be used with any wall support system. For this reason the bracing system is being considered separately in this study. The minimum design criteria usually given in specifications is discussed in Section 2.7.

In some instances, several levels of the permanent structure steel framing have been specifically prescribed to be utilized as temporary bracing, a method which is meant to realize economies to the overall cost of the structure. This is not always as efficient as anticipated, as the degree of accuracy required for setting permanent steel can limit progress of excavation and other dependent operations.

Several types of bracing systems are in use today and the following descriptions will elaborate on the variations of each type, the installation and removal procedures, the material and labor requirements and the advantages or disadvantages of the system. The choice of bracing is closely related to ground wall support, excavation and construction of the permanent structure; all of which must be considered. In shallow cuts it is sometimes possible to design the ground support wall so as to eliminate additional bracing; to act as a cantilever if no decking is required or as a simple span from invert to decking. Maximum deflection must be carefully checked in this type of consideration.

2.8.1 Conventional wales and struts - There are many variations to this type of internal bracing system which incorporate multiple levels of longitudinal horizontal beams (wales) placed adjacent to the wall line, and transverse horizontal compression members (struts), arranged and designed to carry the ground loads imposed on wall supports across the cut. Steel is used exclusively in cut and cover projects of any magnitude and it is not uncommon to see high strength grades used as primary or secondary component members in medium and deep cuts where the lateral loads can be substantial. Wood is not efficient to carry heavy loads and concrete is too massive and unwieldy.

The system is compatible with any wall system mentioned in the previous section. The degree of ease in installation can vary depending on the complexity of the permanent structure, the wall support, and the details of the bracing design. Several factors enter into this. The fabrication of individual pieces in a shop is considerably more efficient and ecomonical than attempting to field fabricate the members. Prefabrication is more of an aid where the ground support system can be placed with a fair degree of accuracy. Where there is much variation, considerable trimming and blocking is required. Simple but structurally adequate joint and connection design will lessen difficulty in installation. Physical limitations of size and length should always be considered. There have been embarrassing instances where an extra long strut or wale could not even be lowered into the excavation due to interference caused by utilities or other internal bracing members. Excessive use of secondary bracing should be avoided to keep the number of pieces and joints to a minimum. The fabrication of joints is a very high labor cost item.

Rolled steel shapes, such as "H" or "I" beams are commonly used as wales. Although "H" beams are often used for struts, in wide cuts it is not uncommon to see large diameter pipe used instead. The work requires skilled personnel, and the labor cost is high. The placement of the various levels of bracing must be coordinated with the excavation to assure that the support system will perform as designed, and deflection of the wall support kept to a minimum.

Preloading of a completed bracing level is used to minimize inward deformations by introducing a stress into the bracing system before further excavation releases increased lateral pressure. This also aids in countering effects of extreme temperature variations.

Advantages of wale and strut bracing include the fact that they have been used successfully on many projects and, since members are completely exposed to view, it is relatively easy to monitor installation and stresses. If designed and scheduled properly, the installation should proceed at the same pace as the excavation with a minimum of interference.

This type of bracing becomes impractical in side slope excavation where the grade on one side of the cut is appreciably higher than the other. Cross bracing in extremely wide cuts is also impractical. The major drawback to this system in normal cut-and-cover excavation is the physical interference of excavating through the struts and between the bracing levels.

2.8.2 Wales and raker system - This system is used most effectively in wide, shallow excavations such as building foundations, though very seldom used in transit or highway applications of cut-and-cover work. The wale and raker system will be described as an alternative system but with the understanding there is minimal application to transportation cut-and-cover work. The wales are placed in a similar manner to the strut and wale system: the major difference between the two systems is the positioning of the sloped raker, which serves the same function as a strut. The raker normally has the lower end attached to an anchor block in the invert or is blocked to an invert slab.

The system can be extremely awkward in medium cuts, and in deep cuts, impractical. The location of the raker often requires blocking out in construction of the invert slab and walls, and the diagonal position of this raker is an encumbrance to efficient operation. The use of this system is limited to extremely wide cuts where cross bracing becomes impractical. It then becomes an attractive alternate to multiple internal rows of piling serving as supporting points for cross bracing.

2.8.3 Cross struts without wales - This system eliminates the use of wales and consists of cross struts which are placed directly pile to pile. It is most often used in long narrow trenches and rarely considered in larger excavations such as cut-and-cover projects for highways or rapid transit. The system should be limited in use to firm and stable soil conditions or to stiff, continuous diaphragm wall systems. The elimination of the wale eliminates a horizontal continuity in the wall system, and for this reason should be used only with wall systems that can provide this continuity. Where applicable, the system is usually the most economical of the internal systems. When used with piles, alignment of piles on opposite sides of the cut is important. A possible use of this system would be in a long and relatively simple, narrow cut-and-cover project, where proper staging between excavation, concreting and backfilling could incorporate multiple reuse of the struts.

2.8.4 Tiebacks or earth anchors - Though rock anchors and ties have been used for some time, the use of earth anchors to hold ground support is a relatively recent development. The improvement in tieback technology has progressed considerably in the past decade, and with new developments in installation procedures and better understanding of its functional properties in different soil conditions, the use of tiebacks in deep cuts is more common today. The system appears to be adaptable to any wall support system and has been successfully used with soldier piles, precast or cast-in-place panels, and other diaphragm walls. Several parameters, other than compatibility with the ground support system, are vital to the determination of the use of this system for successful installation. These parameters include ground suitability, economics, and site conditions.

The soil and groundwater conditions must be suitable for tieback installation. Non-cohesive sands, for example, might preclude conventional installation of tiebacks, and require special procedures. Soft clay or excessive water may present difficulty in achieving proper anchorage. Thorough knowledge of the soil and ground water conditions must be obtained by adequate soil investigation prior to the construction stage of the project to consider this bracing alternative.

The installation itself requires a certain amount of space within the construction site. Encroachment on to private property and the difficulties of removal of the system must be considered. The presence of deep utilities, old piling and abandoned structures are all encumbrances to orderly and efficient tieback installation which must be investigated thoroughly. If even one structure is too deep or one property owner reluctant to allow the use of ties under his property, it could affect this decision for a whole section of cut-and-cover work. The design considerations dealing with adequate performance of the ground support wall must also assess the possibility of excessive deformations through the use of tiebacks.

Tiebacks are generally driven or installed in a pre-drilled hole and grouted to form an earth anchor. There are variations both in the technique of installation and the composition of the tieback itself. The tiebacks can range from deformed steel bars to high strength steel strands. Structural steel "H" beams have been used for tiebacks under special conditions. The anchorage methods can vary from end belled anchors or plate anchors to dependence on soil to grout encasement friction. Special equipment is used in marginal soils, where augering with bentonite slurry might be required to keep the hole from caving.

The typical tieback can vary in length from 30 to 80 feet (9 m to 24 m) or more and carry design loads of 100 to 150 kips. Special installation involving bundled strands of high strength steel have been installed for loads in excess of 1,000 kips, but the costs in this case were comparatively high.

The degree of success in the installation of tiebacks is dependent on the special skills of a crew, where teamwork is vital. The system can become a very costly one if too many failures occur during proof testing, generally accomplished at 130 to 150 percent of the design load.

The installation production rate is relatively high, and coordinates well with other simultaneous activities. For deeper excavations, the total cost of this system can be quite high and substantially more than a conventional wale and strut bracing system. This is due to the fact that the resistance of the ground is the limiting factor of strength of an individual tie. The number of ties required increase rapidly with increased depth.

The principal advantage of using tiebacks is the unobstructed work space available within the cut-and-cover site. Substantial economies can often be realized in excavation, backfill, and the construction of the permanent structure. Tiebacks are ideally suited for side hill cuts where ground elevation varies considerably, since tiebacks, unlike cross bracing, do not depend on equal transfer of load across the cut.

2.8.5 Combined systems - Combined systems including two or more of the systems described previously, have been used with success on many projects. An example would be the use of tiebacks for the lower levels of bracing with the two upper levels as conventional struts and wales where ties would infringe on existing structures. Economics must be considered in the use of combined systems, since continuity of an operation generally allows for efficient production, whereas disruption or termination of one activity and initiation of another is often costly and time consuming due to demobilization, mobilization and re-learning.

2.8.6 Special bracing situations -

a. End Bulkheads. The ends of a cut-and-cover construction site confronts the contractor with similar lateral support problems as the longitudinal sides of the excavation. The difference is that with an internal bracing system, it is impractical to strut the full length of the cut. Conventional wales and diagonal struts are commonly used to distribute the loads from the end bulkhead to the adjoining sides. Occasionally, a cluster of vertical footing piles will be placed on both sides adjacent to the end bulkhead to provide the passive resistance for the loads from the bulkhead. Tiebacks have been also used successfully on end bulkheads even when tunnels have penetrated the vertical face. Where cuts are wide and deep, the loads can be quite substantial and the geometric configuration of this portion of the bracing system can be complicated and costly.

Contract interface problems often confront the contractor at the end bulkhead. Substantially more consideration is required in end bulkhead design to avoid construction scheduling and compatibility conflicts. Examples of the incompatibility of wall and bracing systems at adjacent construction sites are numerous. Structural redesign problems at end walls also arise when untimely tunnel penetrations from neighboring contractors occur. The general approach often used in the language of contracts requiring adjacent contractors "to cooperate," must be more accurately defined to avoid unnecessary delay and cost to the owner.

b. Structural projections and appurtenant structures. Structural projections, such as fan shafts and adits, subway entrances, ventilation shafts and other miscellaneous appurtenances projecting outside of the main wall support line result in higher ground support and excavation costs than in typical sections of the structure. Additional bracing fabrication, change in routine, and special work in a confined area all contribute to this increase of cost. Rectangular shaped projections are not as complicated in terms of distribution of loads from the adjoining area, as the odd angle or curved structures which are common to some subway entrances and fan chambers. The additional cost considerations should be investigated during the design stage to determine whether the added costs are justified or can be reduced through redesign. While some of these structures must be located to the side of the main structure (i.e. subway entrances must be located on the sidewalk) others can be placed within the normal excavation limits.

c. Access for construction activities. Access to the work area is required for construction purposes throughout the project. While some sites can provide access from an area off the street, this is exceptional and more commonly the contractor must provide some means of raising muck and lowering materials and equipment through the decking and bracing. With internal bracing this requires one or more areas of special framing to permit openings larger than normal bracing clearances. Probably the ultimate of construction access requirements exist where a tunnel must be driven from the end of a station or open cut area and provision must be made for lowering a large tunneling machine or shield weighing 50 to 150 tons. Providing construction openings where tiebacks are used creates little or no disturbance; for internal cross-bracing, openings must be given special consideration and design. Where the permanent structure framing is to be utilized as ground support bracing, access openings may become a major concern of both designer and contractor.

d. Rebracing during construction. While the designer and contractor will design and schedule the installation and removal of bracing for maximum efficiency, there are times when the type of ground support, configuration of permanent structure, or depth of cut will necessitate a rebracing sequence. This can be true for internal bracing or tiebacks.

In designing ground support, the usual practice is to place a bracing level a few feet above each slab level (base slab, intermediate slabs and roof) with a maximum ground support span of 16 feet (4.9 m) depending on ground conditions. The bracing level is installed as soon as excavation permits, about 2 feet (0.6 m) below the bracing level. If more room is needed to clear construction equipment, an adequate berm should be left on the sides and a slot provided near the center of excavation for starting excavation of the next level. This will not only protect the integrity of ground support but also minimize deflection of the wall and the accompanying ground settlement. As the structure is built, the base slab is poured and the bracing level above is removed in order to place walls and the slab of the next level. The wall support must be adequate for the worst construction condition: between any last bracing level and the excavated bottom during excavation, or between a slab and the next higher bracing level during concreting. Occasionally the height of a structure level is greater than the allowable span of ground support. This is often the situation on stations of the Washington subway where the roof arch may rise to more than 30 feet (9.1 m) above the base slab. A similar situation exists where extreme depth or a particular type of support system may require bracing levels closer than 16 feet (4.9 m). In the examples to be used in this study, this can be seen for the tieback bracing for 70 feet (21.3 m) depth in Figure 8. When this occurs it will require placing a temporary bracing level across from wall to wall of structure or ground support wall, until the next slab can be placed and achieve sufficient strength to avoid a lateral load on unbraced structure walls. This extra level might consist of a new strut system or a modified bracing level previously removed from below. The removal of the rebraced level is expensive due to limited space and access. The steel members must be cut into short lengths for removal with little salvage value. This should be taken into consideration and avoided, if possible.

2.8.7 Summary -

G. Brace Ground Wall Support

1. Considerations in choosing type of support bracing
    a. Suitability of bracing to ground wall support system
    b. Width and depth of cut
    c. Cost of bracing system on the basis of depth of excavation (comparative)
    d. Accessibility: excavation; material handling; building structure
    e. Suitability to all ground conditions
    f. Dependence on skill of workmen
2. Methods of bracing ground wall support
    a. Internal bracing system, struts and wales (struts only, from pile to pile, suitable only for narrow trenches)
    b. Rakers and wales

c. Tiebacks - down into rock
d. Tiebacks - grouted in soil
e. Wall support designed as simple span, invert to decking
f. Wall designed as cantilever span from invert (suitable only for shallow excavations not requiring decking)

3. Constraints imposed by type of structure
   a. Plan sections other than rectangular box shape may complicate internal bracing
   b. Bracing may have to be left embedded in structure during construction
4. Constraints imposed by construction methods
   a. Internal bracing limits excavation methods
   b. Deep excavations with limited strength wall support (sheeting) may require excessive levels of bracing
   c. Typical construction procedures for these structures make raker type support bracing impractical

2.9 EXCAVATION

The start of a cut-and-cover construction operation may seem slow and even uncoordinated to the casual visitor or sidewalk superintendent. Traffic rerouting, exploratory trenches, utility relocation, installation of a dewatering system, underpinning buildings, and installation of ground wall support and deckings all merge into a prelude to the first act: excavation. Seeing the excavation deepen, day after day, (assuming they can through openings in the deck), the sidewalk superintendents finally concede that there is work going on down there. Smooth coordinated progress of excavation, bracing installation and lagging placement, (where ground support system requires) is the evidence of a well planned job. These operations are mutually dependent, and if not properly organized and managed the results can be confusion, delay and increased costs.

2.9.1 Excavation of the first lift - Although exploratory trenches, preliminary relocation of utilities, and placing of ground wall support all involve removal of soil, mass excavation is considered to begin with the removal of paving over the length of the cut. If traffic must be maintained and decking placed in two stages, the excavation of the first lift will also be done in two stages, first one side of the street then the other. This excavation is slow and will involve much hand work around utilities. The depth to be excavated is 5' to 10' (1.5 m to 3 m), sufficient to expose most utilities except for deeper sewers. Excavation will probably proceed from one end to the other, followed closely behind by the installation of decking: preparing the pile tops, setting pile caps and setting deck beams. The deck beams not only carry traffic and construction equipment, but comprise the first level of ground support bracing, and carry temporary and permanent utilities.

Digging is done by backhoe, gradall or front-end loader, loading directly into dump trucks for off-site disposal. The rate of progress is determined, to a great extent, by placing decking steel and handling utilities. The size of excavation crew and amount of equipment is merely sufficient to keep pace with these operations as the excavation cannot get too far ahead of the decking utility supports. If decking is to be staged, one side must be completed for several hundred feet before starting the other in order to maintain traffic continuity. At intersections, maintaining traffic continuity requires working (excavating) in quadrants.

If the work is to be done in an area where all existing utilities can be relocated, the first lift will be similar in scope to that previously described, but can proceed at a faster pace without the restrictions imposed by working with live utilities. If the area to be excavated is off the street or in a park area, the operation can utilize larger excavation equipment on a regular production bases. Thus it is seen that the urban street setting of a cut-and-cover project contains its own inherent limitations and restrictions which simply do not exist in other, more accommodating, situations. To apply unit prices and excavation progress developed under different circumstances to this environment would be misleading.

2.9.2 Excavation of the second lift - The second lift is considered to extend from below the deck beams to a couple of feet below the next bracing level, or to a depth of about 20 feet (6 m). On some projects this level will be excavated by the same methods to be used for lower levels; on others it is possible to utilize methods that are less restrictive than those required below. If, for instance, it is possible to ramp down to this level, either between deck beams or from a side lot, it is then possible to excavate with front end loaders which transport the muck to the surface and load directly into waiting dump trucks for off-site disposal. The fewer times that muck is transferred or transported, the less expensive is the excavation. Another possibility for excavating the second level with decking in place, is to use a large backhoe or gradall excavating between deck beams, with the timber decking removed, and loading directly into dump trucks. Below the second lift the depth becomes too great for this type of equipment to reach from the deck.

If the project does not require decking it may be possible for dump trucks to travel down a ramp to be loaded in the cut by a front end loader or backhoe. The use of scrapers that are so effective in cut-and-fill highway work is precluded, even without decking or internal bracing, by the restrictions of working room and street travel. The relatively narrow, confined transportation tunnel work must be done by smaller equipment, with resulting higher unit costs.

2.9.3 Excavation of remaining lifts below the second lift - A lift is generally considered the distance between bracing levels. With internal bracing this may be as much as 16 feet (4.9 m) and require two successive (or simultaneous) excavation passes. In deeper cuts, tiebacks will usually require more levels of bracing due to load limitations on individual earth anchors. The excavation of each lift proceeds progressively along the trench from one end to the other with the setting of temporary bracing following closely behind and proceeding in the same direction. To minimize deflection of the ground support, excavation should not exceed that actually required for placing the bracing. If internal bracing is used, this causes a problem in excavating the next lift because 6 to 10 feet (2 m to 3 m) headroom is needed for equipment. Usually it is possible to keep side berms close to the bracing with a sloped trench in the middle of the cut for equipment. When scheduled properly, both excavation and bracing crews work continuously from one level to the next.

Except where it is possible to bring trucks down into the cut, excavation at these depths consist of three operations, digging, lifting muck to the surface, and disposal, each performed by different equipment. Digging may be done by dozers, front end loaders, or small backhoes; and very often by a combination of all three, depending on the soil, the number and location of transfer points to lifting equipment and the rate of production expected. The small combination front end loader and backhoe is a favorite for careful perimeter excavation near the ground support wall and for digging drainage ditches. Larger, crawler mounted dozers and front end loaders are more efficient for pushing or hauling muck to the transfer point.

Whether or not decking is needed, it is usually not practical to do the digging directly from the surface for the full length of the cut with a clamshell or other crane type excavator. The most common excavation method in use on recent BARTD and WMATA projects utilizes a clamshell of 2 cu. yd. to 4 cu. yd. (1.5 m^2 to 3 m^2) capacity to lift the previously excavated muck and load directly into dump trucks. The distance between lift points should be a function of the equipment below that is moving muck for lifting, but may be limited by utility location, intersections and other surface considerations.

Of the various crane buckets, the clamshell is the most efficient for this type of operation for a variety of soils. Once the material has been loosened by the digging equipment the need is to remove the muck as fast as it can be excavated. The size of crane required for handling this large clamshell bucket may impose a greater load on the deck than does normal traffic. It is usually more advantageous to design the decking for the heavier load than to limit the size of crane and hence excavation production. As mentioned in the section on decking, the larger clamshell bucket will not fit between deck beams and special framed wells are needed for clamming. These wells must be included in each bracing level when internal bracing is used.

Although this is the method most commonly used for lifting muck, alternate methods have been tried or suggested and some work quite well under special conditions. Several contractors on the BARTD system used conveyors for bringing muck to the surface. Conveyors can move muck efficiently and more rapidly than a crane under ideal conditions. They become troublesome when the soil contains some clay and may become completely inoperable when working in a stiff clay. Another drawback is the frequent handling of long conveyors in a confined area as the work progresses. And, a breakdown of even one conveyor means a shutdown of the whole system. Other conveyance systems that have been suggested are vertical bucket conveyors, vertical auger conveyors, hydraulic pipelines and pneumatic pipelines. Each of the systems have potential advantages for special conditions. One disadvantage of each is that they require expensive special equipment that does not have the versitility of a crane. The vertical conveyors and pneumatic pipelines are limited to a sandy material; the hydraulic pipeline requires an elaborate mixing system and continuous operation.

The third operation in the excavation cycle, disposal, has become fairly standardized for cut-and-cover work. Rear end dump trucks of 10 cu. yd. or 20 cu. yd. (7.6 m^3 or 15.3

m^3) capacity are used to haul away excavated muck. The trucks may be loaded directly by the lifting clamshell or a storage hopper may be used on the street to load the trucks. A hopper is useful for regulating the flow of muck and preventing a late truck or down time on the crane from holding up excavation progress. The hopper can be loaded by either clamshell or conveyor. If the original soil is of satisfactory quality to use as backfill, and a storage area is available, a portion of the muck will probably be stored for this purpose. The remainder will be sold or dumped as economics and the current local market for fill dictate. Disposal of excavated material is often a problem for a prospective contractor in an urban area; providing an alternate or directed disposal area within a short haul distance of the site will help reduce the overall cost of the project.

2.9.4 Construction factors that affect excavation methods and progress - Aside from width and depth the major construction constraint to excavation progress is the obstruction caused by the bracing system. In addition to the mutually dependent scheduling mentioned previously, the physical interference of internal bracing slows excavating equipment and requires modifying procedures reducing the efficiency that could be achieved in an open area. In a shallow cut it may sometimes be economical to design the ground support system to span from the invert to the deck level to avoid the need for additional bracing. In deeper cuts, tieback bracing leaves the excavation relatively unencumbered. While in most cases this will not permit a radical change in method of excavation, it does allow for greater progress with the same type of equipment. The use of soldier piles and lagging may restrict excavation which cannot proceed more rapidly than the placing of lagging and the hand mining accompanying it. By the same token, a certain amount of manual work is required for concrete diaphragm walls as well. These walls, whether cast-in-place or precast in slurry must be cleaned and repaired as the excavation proceeds. To postpone this work till excavation is completed would necessitate erection of scaffolding for this purpose. Any serious defects in the wall should be corrected as soon as they are uncovered.

The slurry consists of a colloidal solution of expanding clays with a portion of native soil in suspension. It is a soft jelly-like substance when dry and clings tenaciously to the rough concrete surface. It must be forcibly cleaned from the concrete to permit inspection of the structural integrity of the wall. Leaving this coating would be detrimental to remaining construction activities as well as being aesthetically unacceptable. The most efficient method for cleaning the remaining slurry is a high pressure water jet, but this can create problems for excavation equipment in soils containing clay or silt.

2.9.5 Summary -

H. Excavation

1. First lift-expose utilities, prepare for decking
    a. Backhoe excavating and loading trucks
    b. Front end loader excavating and loading trucks
    c. Hand excavation around utilities
2. Second lift-down to first bracing level
    a. Backhoe on deck excavating and loading trucks
    b. Backhoe below deck excavating and loading trucks
    c. Front end loader excavating, hauling, and loading trucks on deck
    d. Front end loader excavating and loading trucks below deck
    e. Front end loader(s) and/or dozer(s) pushing earth to central area, lift by clamshell and load trucks on deck
3. Additional lifts below second
    a. Pushing or hauling earth below by front end loader(s) or dozer(s), to central loading area
    b. Loading trucks below by front end loader with ramps to surface (not practical where there is internal bracing)

c. Lift earth by clamshell and load trucks above deck, directly or from hopper
d. Load hopper below and lift to surface by flight conveyors or bucket conveyor to hopper above deck (not practical for clay or earth with clay binder)
e. Lift material from below by hydraulics, remove water above and deposit earth in hopper
f. Lift earth from below by pneumatic pipeline to special hopper (not practical for clay or earth with clay binder)

4. Constraints imposed by type of structure
   a. No major restrictions - shallow structure without need for internal bracing not restricted by 3b above
5. Constraints imposed by construction methods
   a. Internal bracing limits activity below deck. Use of trucks below not practical. May limit lifting by clamshell or flight conveyors
   b. Soldier pile and lagging requires hand excavation for placing lagging
   c. When using soldier piles and lagging, general excavation cannot proceed faster than hand excavation and placing of lagging
   d. Walls cast or placed in slurry trench require special cleaning. Most efficient method, water jet, may cause problems for some soil conditions (non-draining)

2.10 CONSTRUCTION OF PERMANENT STRUCTURE

The construction of the permanent structure begins on the designer's drafting table. More than in any other phase of the project, the cost of this activity is affected by the experience and ability of the designer. Within the general configuration required by site conditions, the details of the structure may have a greater effect on its cost than the construction methods employed by one contractor or another. The basic design is completed before a contractor is chosen, and except in unusual circumstances the design is considered fixed and unchangeable. Even where a contract contains a value engineering clause it is unusual to make any major revisions to the structure itself. In understanding this, the designer must consider it an obligation to include efficiency of construction among other considerations required in the design. To leave inherently difficult construction problems to the contractor to solve does not dispose of them, it merely raises the cost of the project.

2.10.1 General construction options - Three basic construction options will be considered in the course of this study. These are: 1) Traditional construction. After completing excavation to invert, the structure is built in sequence, base slab, walls and roof. Then backfill is placed and the utilities and pavement restored. 2) A second option, of partly inverted construction, is possible when using concrete diaphragm walls. The excavation is completed to invert and the invert slab placed. The roof is then placed, and during the backfill and restoration activities the interior finish walls and slabs are constructed. 3) A third option is called under-the-roof construction. While this method has been used in Europe and Canada, it has not yet been tried in the United States. With this method the roof is placed on the concrete diaphragm walls and backfilled while excavation to the invert is carried out below. Then the invert slab is placed and interior finish work completed.

The purpose of options 2 and 3 is twofold; to reduce the period of surface disruption, and by performing operations simultaneously instead of sequentially, to reduce the total overall construction time. While some individual activities are more expensive with this method because of limited access after placing the roof, savings are realized in reducing construction time. Whether these savings can offset the cost increases, and if so, under what conditions is this most likely, are some of the answers sought by this study. In any event, the choice of options should be made early in the design period before decisions on details not commensurate with the basic construction methods have been made.

2.10.2 Structural design options - The primary structural materials used in transportation tunnels are steel and concrete. Other materials may be used for architectural purposes or special features. The way these materials are to be used determines to a great extent the construction methods to be followed in building the structure.

1) Structural steel with concrete arches - Much of the old New York subway system was constructed by this method, and while it is not used in more recent systems, it is of interest historically. In this system square-framed structural steel columns and beams at 5 foot (1.52 m) on center intervals constitute the primary support. Unreinforced concrete arches (called jack arches) are poured between steel members leaving the inside flanges exposed. These flanges are later painted. Because the contact area of each steel beam with the concrete poses a potential route for water seepage, an unusually effective waterproofing is required. This is provided by hand laid brick and mastic for the walls and multi-ply membrane waterproofing on the roof, an expensive procedure with today's labor costs.

2) Reinforced concrete box structure - This type of structure has replaced the old steel and jack arch for most rapid transit line structures and highway tunnels. It is basically stable, relatively easy to form and place, and easier to maintain. The thick concrete walls and slabs require less waterproofing than jack arches. This type of structure is used for the typical highway tunnel and rapid transit line structure of this study as shown in Figures 2 and 3 of Section 1.

3) Reinforced concrete arch structure - This is a variation of the box structure used for wide span structures without center wall or column support. It has been used for some BARTD and WMATA stations and some highway tunnels. If this type of structure is placed beneath a street where traffic is

to be maintained, street decking spans become a problem because center supports interfere with effective forming of the arch.

4) Reinforced concrete walls and composite slabs - This is an alternate to the concrete box structure where wide spans are involved such as multi-lane highway tunnels or, in one case considered in this study, a rapid transit station (see Fig. 4). The section used is similar to several on the BARTD system. The walls are formed and poured as in a box structure. The slabs consist of reinforced concrete and steel beams encased in concrete. The steel beams are prestressed in compression prior to placing concrete. A pattern of steel shear studs welded to the top flanges enable the steel and concrete to act together more effectively as a composite member. As the slab is loaded the concrete takes most of the compression and the steel beams previously stressed in compression take the tension.

5) Cast-in-slurry ground support walls - When this type of ground support is used, whether it is an SPTC wall or reinforced concrete, it is capable of being incorporated into the final structure. Since the excavated trench constitutes the form for the tremie concrete, the wall has a rough finish that may vary by several inches. Although the cast-in-slurry wall is structurally capable of being used as the permanent wall of the structure it is difficult to key slabs into this wall. An inside wall or pilasters must be provided for this purpose. If any sort of architectural finish is required, a finish wall should be placed inside. Since it is possible for water to find its way between these walls they should either be anchored together or relief drainage provided to prevent a pressure build-up that could damage the thin finish wall. The slurry coating outside the wall aids in minimizing ground water inflow.

6) Precast concrete ground support wall - This type of ground support can also be incorporated in the completed structure. Since a better finish can be cast into the panels than is

possible in cast-in-slurry, it is not necessary to add a finish wall in most cases. Care must be taken to align the panels to achieve a uniform wall. Waterproofing can be incorporated on the outside face, leaving only the joints to be sealed. With both cast-in-slurry and precast walls that are to be used for structure walls it is best to bring the walls up to the underside of the roof which must bear on it. While it is possible to key in interior slabs that do not carry much weight, the roof requires adequate bearing because it carries the weight of the fill. The roof can be either cast-in-place or precast reinforced concrete, or composite steel and concrete.

2.10.3 Construction methods - In building the structure some procedures are similar to those used on structures above ground, but many are modified by being confined below street decking. The far ranging tower crane that sits above most buildings under construction today is impractical for this situation and is replaced by a traveling hydraulic unit crane on most cut-and-cover projects. Heavier lifts such as the setting of steel beams may require a larger crane. With the limitations of decking and internal bracing there are many places that a crane traveling on the deck cannot reach. Sometimes it is possible to use a small hydraulic crane below for setting steel. If not, other equipment may be pressed into service. Electric tugger hoists with suitably located pulleys may be used. On occasion, a fork lift or a modified front end loader may be utilized in setting steel.

Forms for concrete also differ in many respects from their counterparts in building construction. The typical column and slabs of most buildings are replaced by heavy walls and slabs below ground. Many of the walls poured against the ground support require one-sided forms, a situation that rarely exists in buildings, and which requires more substantial bracing than two-sided forms. Wherever possible, inside concrete dimensions should be kept uniform. When a contractor has many pours to make with similar dimensions he can reuse forms and the cost will be lower. If the number of similar pours exceeds about twenty, it may pay him to use traveling steel forms for even greater savings. On the other hand, every pour that must be made using specially constructed wood forms in a shape or location that prevents reuse, increases cost. A theoretical design that reduces concrete quantity at the expense of additional forming should be critically reviewed by the designer, as it may actually increase costs instead of decreasing them. The same is true of some architectural concrete pours which increase forming costs more than the results justify. Specifications interpreted as requiring high quality concrete finishes on concrete that will later be covered by an architectural covering, or never even seen by the public, has increased the price on more cut-and-cover projects than most designers realize.

Placing concrete below the surface is no longer the wheel barrow and shovel operation of many years ago. Modern efficient concrete pumps developed in the last 15 years have made it possible to deliver concrete to any part of a project provided access holes or pipes are not too far apart. While distance is still a limiting factor, plugged concrete lines for short distance pours have become rare and more likely caused by delay, carelessness or a poor concrete mix. Another well developed concrete placing method used below ground is the belt conveyor system. Though not quite as versatile as the concrete pump, it is less expensive when large volumes are to be placed. When used properly in an applicable location it is a very efficient placing method. The least expensive method for placing concrete in accessible locations has been, and still is, gravity placing. Working through the decking naturally limits this possibility, but by using hoppers and drop pipes, conctractors still use this method where they can.

2.10.4 Limitations imposed by type of structure and other construction operations - When discussing the responsibility of the designer it must be recognized that the designer is often limited by the type of structure required. While it must be recognized that the least expensive, most easily constructed structure is uniform in cross section and confined between parallel vertical planes, there are times when this is not possible. Station entrances, for example, must be placed to the sides of the station as they cannot emerge in the middle of traffic. The type of roof to be used may be dictated by the span required by the function of the structure, or in the case of a station, by asthetic considerations of the public. One of the major advantages of cut-and-cover tunnels over deep tunnels is that the roof, not having to carry as much overburden can be designed as a flat span for a greater width, and placed with relative ease. In deep tunneling an arch is the most efficient roof, but if you need to span a three or four lane highway as in the case of the recent Eisenhower Tunnel in Colorado, the arch becomes quite large, and support presents a major problem.

The most important interference consideration in building the structure is ground support bracing, decking, and decking support. Actual physical interference of these members with the structure slows production, requires modification of methods and increases time of completion. The spacing of bracing levels should, within the limitations set for maximum span width of ground support, be planned for orderly removal during construction of structure. Removal of bracing should not place undue stress on cantilevered walls. By placing bracing levels above slab levels this can be avoided. While interior cross bracing causes more interference than tiebacks, the tieback wales create similar problems when the structure is to occupy the full width of excavation.

Where the structure slab incorporates steel framing it may be possible to utilize this as a bracing level. However, setting of permanent steel during excavation may, because of the greater degree of precision required, slow excavation progress. Where physical interference of a bracing member or deck support member with the concrete portions of the structure cannot be avoided, special procedures must be followed. Reinforcing steel and forms may have to be modified and a minimum depth of concrete blocked out to later allow cutting the member and patching the concrete.

Restrictions imposed by placing the roof out of sequence with respect to excavation and concreting has been mentioned previously.

In placing concrete for the permanent structure, care must be taken by the contractor in scheduling activities to avoid a situation where loads will be placed on the structure before the concrete has achieved its required strength. The consequences of failing to foresee this difficulty can result in needless delay or expensive reposting.

2.10.5 Summary -

I. Construction of Permanent Structure

1. General construction options
   a. Complete excavation; construct structure from bottom to top; backfill
   b. Complete excavation; construct base slab and roof; backfill and complete structure simultaneously
   c. Excavate for roof; construct roof; backfill above roof and excavate below roof; complete structure
2. Structural design options
   a. Steel columns and beams with concrete jack arch walls and slabs
   b. Reinforced concrete walls and slabs (box structure)
   c. Concrete arch structure
   d. Reinforced concrete walls and composite slabs
   e. Cast-in-slurry wall ground support incorporated in structure with finish wall
   f. Precast wall ground support incorporated in structure
3. Construction methods
   a. Place structural steel
      Crane, above
      Crane, below

Hoist

Fork lift or front end loader

b. Concrete Forms

Wood, single use

Wood, multi-use

Steel

c. Place Concrete

Pump

Conveyors

Gravity

4. Limitations imposed by type of structure

a. Roof span required by structure use (number of lanes, etc.)

b. Depth of structure may negate advantages of constructing roof first

c. Possible advantages of utilizing slab beams as ground support bracing

d. Type of structural walls and slabs affect amount of waterproofing required

5. Limitations imposed by construction methods

a. Ground support bracing may have to be left in place (projecting through structure while structure is constructed)

b. Decking support may have to be left in place (projecting through structure) while structure is constructed

c. Placing roof first, restricts excavation below

d. Placing roof first precludes placing concrete below by gravity

2.11 BACKFILL

The last stage of backfilling could be considered part of final restoration together with restoration of utilities and repaving. These items are so mutually dependent that it is difficult to separate them. In a deep cut, backfill, like excavation, is best considered in more than one vertical stage where different criteria determine production rates, and even methods of performance vary. Below the level of utilities, backfill is a distinct operation limited mainly by interference with bracing removal and problems of working from, and through, decking.

2.11.1 Backfill from roof to underside of utilities - As in the case of excavated material, backfill is transported by dump truck. The contractor then has several options for placing and compacting the fill. Although it is sometimes possible to reuse the original excavated material as backfill, two conditions must be met for this option. First, the material must meet the backfill specifications of the contract which usually limits the percentage of clay, rubble and organic material. Although clay or silt may be present in the original soil, they exist in a pre-consolidated condition which cannot be duplicated in the backfill operation. Secondly, there must be a storage area at the site, or within reasonable distance, available to the contractor for the duration of the project. If either of these conditions cannot be met the contractor must obtain backfill from some acceptable source in the area.

The most common method for placing the material is to remove sections of decking and dump the material from above to form a pile on the roof. The material is then spread by small dozers or front end loaders in appropriate horizontal layers for compaction. Utilities and roof waterproofing must be protected from falling dirt. In some areas this method cannot be used because of the number of utilities present or because they cannot be adequately protected. Another possibility is to construct down ramps, if the depth is not too great, and either run dump trucks down, or dump above and use front end loaders to transport and spread the fill. This is usually not possible if internal bracing has been used. If conveyors have been used for removing excavated material, they can be utilized in transporting backfill, otherwise they would be too expensive to buy and install for this operation alone. It would be impractical to place backfill by hydraulic pipeline and use of a pneumatic pipeline presents problems of dissipating the nozzle energy to make it safe in a confined area.

Compaction is usually done by vibrating drum or sheepsfoot roller, either self propelled or pulled by a small tractor. Headroom and turning room are limiting factors favoring small compaction equipment instead of the larger more efficient machines used on road work. This is particularly true with internal bracing, but even with the tiebacks, fill must be placed and compacted within a couple of feet of the wales before they can be removed. The compaction of the last few feet under each wale must often be done by small hand held vibrator plates. The work of placing fill is therefore interdependent with removal of bracing and, for maximum efficiency, must proceed at the same pace. Ponding is usually insufficient to achieve the compaction required beneath a city street. Insufficient drainage between ground support walls can create problems for equipment movement and the permanent structure work still in progress (shafts, entrances, etc.). Ponding is therefore not ordinarily permitted or desirable for backfill compaction for transportation tunnels.

2.11.2 Backfill from underside of utilities to street sub-base - As mentioned earlier this last 10' (3 m) or less portion of the backfill operation is closely tied to other restoration work proceeding simultaneously. In addition to restoring permanent utilities and adequately tamping bedding material beneath existing preserved utilities, orderly removal of the decking must be geared to these operations. This will be discussed under restoration, but it will suffice here to mention that if the decking was installed in sections in order to maintain traffic, it must be removed the same way. In this case, utility restoration and backfill must follow suit.

It is unlikely that in most areas it will be possible to use self propelled, mechanized equipment for backfill at this level because of utilities. As the decking is removed dump trucks will have to unload sufficient material for the area of the next section to be removed. It may be possible to dump from the side if the work is done on half the street width at a time. Most spreading and compaction must be done by hand because of space limitations and to avoid damage to utilities. Since the other work of restoration is also slow, this need not be a limiting factor, but many men are required and the unit cost is high. After the utilities are covered it is possible to use light compacting equipment, but by then the work is almost complete.

2.11.3 Factors affecting backfill operations - In addition to the items mentioned above, there are others which may affect backfilling methods or production on individual projects. Work on appendage structures projecting above the roof should proceed concurrently with backfilling, but these structures also limit movement of equipment in spreading and compacting the backfill.

If the decking is supported on piles projecting through the roof of the structure, the piles must be cut and the decking reposted from the roof slab in order to complete waterproofing the structure and allow for easy removal of deck supports later. An alternative, if the roof is less than about 10 feet (3 m) below the deck is to place corrugated pipe around the piles while backfilling around them. This will permit them to be cut and removed later. In either event this operation will restrict backfill work.

2.11.4 Summary -

J. Backfill
   1. Placing backfill options
      a. Dump through decking
      b. Dump on top and place by front end loader
      c. Ramp down from surface for dump trucks
      d. Conveyors from side or deck
   2. Constraints imposed by type of structure
      a. Depth of structure may make ramps impractical for initial placing

b. "Appendage" structures (vents etc.) may preclude ramps for backfilling

3. Constraints imposed by construction methods
   a. Need to transfer load of decking to roof of structure may delay backfilling and reduce options of placing
   b. Utility location may complicate placing backfill
   c. Backfilling around existing utilities to remain in place must be done by hand
   d. Replacing existing or temporary utilities delays backfilling operations
   e. Removal of decking must be coordinated with backfilling
   f. Distance and availability of backfill if previously excavated material is not acceptable

2.12 RESTORATION

After backfilling has reached the level of utilities, the work of restoration begins. Utilities that were temporarily bypassed outside the excavation (gas, water, etc.) are replaced by new lines. Temporary utilities, sewers, manholes and junction chambers are replaced with permanent facilities. Other utilities that have been carefully maintained, and withstood the rigors of construction, are cut loose from their decking support after preparation of bedding. New connections are made and tested for water, gas and sewer lines, electrical and telephone cables are pulled; and, where necessary, new service laterals are installed to adjacent buildings.

As backfill proceeds over the utilities, the temporary decking is removed. The street sub-base is placed and compacted and topped by new pavement, first one side of the street, then the other. Last, the sidewalk is replaced if it has been disturbed, while temporary walkways are provided on the newly completed roadway. Temporary bridging is used to span from the roadway to building entrances until the sidewalk concrete has set. There are few options left to the contractor in construction methods and little chance at this stage of improvement through technical innovation. Conversely though, there exist in this period, many factors that can and often do, cause unnecessary delay and increased costs.

2.12.1 Utility problems - Theoretically, the restoration of utilities should proceed relatively smoothly compared to the problems of finding, exposing and relocating utilities in the early stage of the project. Everything is now exposed to view and final locations known. In practice the restoration period too often becomes one of hectic activity when utility owners and local public works departments suddenly realize that the street that has been open so long is soon to close. Utilities that have been maintained for two years or more are critically inspected. The contractor may suddenly find that a utility that was judged in good condition when first uncovered is no longer acceptable and must be replaced, or that current use projections require increased capability for the utility. Normal waiting time for material may exceed what is available by several months if the utility is unusual. If last minute design changes have been made to the appendage structures above the roof, several utilities may be affected. By their very presence alone, these structures limit the street area available for placing utilities.

The biggest problem to be faced by the contractor is usually coordination of the work when the restoration of utilities is done by the utility owners themselves. It is difficult enough to coordinate and monitor all operations of the contractor and various subcontractors involved in backfill, deck removal, utility restoration, traffic relocation, delivery of utility material and hauling away used decking material. The utility owners that install their own utilities are considered in the same category as a subcontractor by the owner. In actual fact, however, the contractor has no real control over the utility field forces and must rely on their continued cooperation to meet schedules. Although many utility companies will try to cooperate, they have their own scheduling and manpower problems that are more concern to them than are those of the contractor. Even one operation out of phase will affect all others and may cause lost time and rescheduling repercussions which can have a ripple effect. Added to the other difficulties is the fact that temporary utilities must remain in use until the permanent utilities are completed, tested and put in service.

2.12.2 Problems connected with decking removal and repairing - In Section 2.11 the effect of decking removal as it concerns backfilling was discussed. The orderly removal of decking is another scheduling problem that affects other restoration activities. It cannot proceed faster than the backfill around utilities and the subsequent removal of maintained utility supports. At intersections, this work, as in the case of initial installation, must be done in quadrants if traffic is to be maintained in both directions. Any decking of sidewalk areas will usually increase the number of stages required for removal, as this work must wait till part of the street has been paved to reroute pedestrians.

Even where the original sidewalk has not been disturbed it has become common on large projects to require the contractor to put in a new sidewalk. This may be part of the original contract, or added later as an extra. The idea is, that as long as the street is disrupted anyway, it is a good time to restore what may be an old patch-quilt type of sidewalk with a modern uniform or special textured version. While this may be a valid decision, it can complicate an already difficult schedule if the decision is made at the last minute.

In summary, the restoration period instead of bringing a sigh of relief, too often ends in a cry of anguish because of insufficient planning and cooperation of all concerned.

2.12.3 Summary -

K. Restoration

1. Problems connected with utilities

a. Coordination of utility replacement with backfill

b. Coordinating utility replacement when work is done by owner agency.

c. Rebuilding manholes, splicing chambers, etc.

d. "Appendage" structures (vents, stairs, etc.) may restrict amount of street width available.

2. Problems connected with decking

   a. Decking must be removed in stages.

   b. Cross-streets increase the number of stages required for deck removal.

   c. Pedestrain decking increases the number of stages required for deck removal.

3. Problems connected with replacing pavement

   a. Pavement must be replaced in stages, coordinated with deck removal.

   b. Cross streets increase the number of stages required for placing pavement.

   c. Replacing sidewalk requires temporary bridges for pedestrian access to buildings.

SECTION 3

3.0 APPLICABILITY OF CONSTRUCTION METHODS

For almost every activity, sub-activity and operation in the performance of a cut-and-cover tunnel contract, a multiplicity of methods and equipment options are available for consideration. No one single set of factors applicable to all activities can be used in choosing between alternate methods. As each activity is unique, each must be considered first on those factors governing the activity or operation and secondly by its effect on the overall construction process. In most cases the choice will be based on economic considerations. Which method is least expensive? Which type of equipment can perform the operation most satisfactorily? At times it may be economical to use equipment that is not ideally suited to a particular task simply because it is at hand, rather than wait for delivery of more suitable equipment. In other cases factors such as safety or public relations will dictate a solution that is not the least expensive. Gas lines will be temporarily moved out of the cut wherever possible even though it might be cheaper to support them in place. Construction barricades will be painted attractive colors instead of being left bare, and will usually contain windows for honorary superintendents. Each day presents new decisions and choices to be made, most of them simple and straightforward for experienced builders; some quite complex, involving technical, economic, or even legal complications.

The discussions in Section 2 describing the basic activities and methods pointed toward some of the more obvious choices that are made given the general situations of this study. This section will try to define more specifically the factors and parameters pertaining to those activities involved in important construction decisions. In many instances the choice of a particular method may have little impact on the overall project provided a proper decision is made. For instance, the expected flow of ground water in a particular soil may be such that either deep wells or ejectors could be used. The choice between them would not affect the overall project nearly as much as delaying the decision to dewater till after excavation had begun. Likewise the method used to underpin a particular building would not have as much impact on the project as the choice of using a rigid diaphragm wall to avoid underpinning.

3.1 TRAFFIC CONTROL

The decision to install decking to maintain traffic is almost axiomatic in the urban environment of the job site chosen for this study. It is included here to show how the factors affecting this decision differ from those of other activities. The table shown on Figure 10 indicates the major considerations are the need for access by pedestrians and various types of vehicles, and a limited number of options of traffic flow. The contractors' operations have very little effect, if any, on this decision, but in planning these operations consideration must be given of the necessary traffic patterns imposed by these other factors.

This table, and the others in this section, use an approximate relative evaluation of available options. These options are rated as: (✓) Best solution, (+) Possible alternate solution, (-) Less desirable solution, or (0) Undesirable. Needless to say the best solution is not always the same for all situations, but has been chosen as best for the general conditions of this study. In some cases no one solution is best and several acceptable alternates are indicated. The table also indicates that solutions acceptable for some considerations may be completely undesirable for others. Were this table made for a site with little or no traffic, the results would be quite different. For the site of this study the table indicates the advantages of temporary deckding to maintain traffic.

A. TRAFFIC CONTROL								
	NARROW STRUCTURE				WIDE STRUCTURE			
NEED FOR ACCESS TO SITE	TRAFFIC ROUTED TO SIDE STREETS	TWO WAY TRAFFIC REDUCED TO ONE WAY	TRAFFIC ON TEMPORARY SIDE ROADS	TRAFFIC MAINTAINED ON INSTALLED DECKING	TRAFFIC ROUTED TO SIDE STREETS	TWO WAY TRAFFIC REDUCED TO ONE WAY	TRAFFIC ON TEMPORARY SIDE ROADS	TRAFFIC MAINTAINED ON INSTALLED DECKING.
PEDESTRIANS	o	–	+	✓	o	–	+	✓
EMERGENCY & SERVICE VEHICLES	o	–	+	✓	o	–	N.A.	✓
TRANSPORTATION VEHICLES	–	+	+	✓	+	+	N.A.	✓
PRIVATE CARS - DAYTIME	+	+	+	–	+	–	N.A	+
PRIVATE CARS - RUSH HOUR	–	+	+	+	+	–	N.A	✓
NORMAL TRAFFIC - ONE WAY	–	N.A	+	✓	+	–	N.A	✓
NORMAL TRAFFIC - TWO WAY	–	+	+	✓	+	–	N.A	✓
CROSS TRAFFIC AT INTERSECTIONS	+	+	–	✓	+	–	N.A.	✓

✓ BEST SOLUTION
+ POSSIBLE ALTERNATE SOLUTION
– LESS DESIRABLE
o UNDESIRABLE

Figure 10

3.2 MAINTAIN, REPLACE OR RELOCATE UTILITIES

There are almost as many ways of handling utilities as there are different types of utilites. This subject has been covered thoroughly in Section 2.3 and, as with traffic control, is repeated here in Figure 11 as being illustrative of a type of activity with its own unique problems and solutions.

In general, it is less expensive to carry small local utilities below the decking than to relocate them. Most local lines must be kept in the street area in any event to provide continuous service to adjacent buildings. Large mains may sometimes be rerouted. Large water lines and all gas lines should be temporarily rerouted outside the cut for safety. In the case of cross lines at intersections, this is sometimes not possible and should be protected in place. Additional valves may have to be installed to insure fast shutdown in emergencies. Sewers, manholes and splicing chambers are usually too heavy to carry and may be replaced with light weight temporary facilities and rebuilt during restoration. Generally pipe without couplings such as bell and spigot cast iron or concrete cannot be hung efficiently without secondary support and should probably be replaced.

B. MAINTAIN, REPLACE OR RELOCATE UTILITIES								
UTILITIES	RELOCATE PERMANENTLY OUTSIDE WORK AREA	RELOCATE PERMANENTLY INSIDE WORK AREA	REPLACE WITH NEW MATERIAL & SUPPORT IN PLACE	RELOCATE TEMPORARILY OUTSIDE WORK AREA	RELOCATE TEMPORARILY INSIDE WORK AREA	MAINTAIN (SUPPORT) IN PLACE	REPLACE WITH TEMPORARY MATERIALS	REPLACE UNDERGROUND (OVERHEAD WIRES ONLY)
SEWERS: LARGE MAIN	+	0	0	+	−	0	✓	N.A.
LOCAL	+	−	0	+	+	0	✓	N.A.
MANHOLES & JUNCTION CHAMBERS	+	0	0	+	+	0	✓	N.A.
WATER: MAIN	+	0	0	✓	0	0	+	N.A.
LOCAL	+	−	−	+	+	✓	−	N.A.
GAS: MAIN	+	0	+	✓	0	+	0	N.A.
LOCAL	+	0	+	✓	0	+	0	N.A.
ELECT. & TELE. CONDUITS: DUCT BANKS	+	+	+	0	+	✓	−	N.A.
SINGLE CONDUITS	+	−	+	−	+	✓	−	N.A.
SPLICING CHAMBERS	+	0	0	+	+	0	✓	N.A.
SPECIAL LINES (STEAM, ETC...)	+	+	−	0	+	✓	−	N.A.
LIGHT & TRAFFIC POLES	0	+	+	+	✓	+	+	N.A.
OVERHEAD WIRES		0	−	+	+	+	+	✓
TROLLEY WIRES	0	+	+	+	✓	+	−	0

✓ BEST SOLUTION
+ POSSIBLE ALTERNATE SOLUTION
− LESS DESIRABLE
0 UNDESIRABLE

Figure 11

3.3 INSTALLATION OF GROUND WALL SUPPORT

This is probably the most important variable in considering construction methods, and with the possible exception of traffic control and restoration, this decision affects every other major activity. This can be seen in the list of factors in the table of Figure 12.

These factors are not of equal importance and no attempt is made to weight them. Their relative influence in the decision process will vary from project to project. In general, however, it can be seen that the three methods to be used for this study have the highest ratings. In the case of the cast-in-slurry wall both the European and American versions are shown and appear to have equivalent ratings. Since the cost data most readily available comes from projects where SPTC walls have been used, this method will be used for this study. The highest rating for the remaining ground support methods is that of steel sheet piles, failing only in the important factor of disruptive impact because of driving noise and vibration.

F. INSTALLATION OF GROUND WALL SUPPORT

FACTORS IN CHOOSING GROUND WALL SUPPORT	SOLDIER PILES AND LAGGING	STEEL SHEET PILES	CLOSELY SPACED CONCRETE PILES	SOLDIER PILES AND C.I.P. CONCRETE	SPRAYED IN-PLACE SHOTCRETE	REINFORCED C.I.P. CONCRETE IN SLURRY TRENCH	S.P.T.C. IN SLURRY TRENCH	PRECAST CONCRETE PANELS IN SLURRY TRENCH
INSTALLATION: COMPARATIVE COST	✓	+	−	−	+	−	−	−
TIME REQUIRED	+	+	−	−	✓	+	+	+
DISRUPTION IMPACT ON AREA	+	O	−	−	+	+	+	+
EFFECT ON OTHER OPERATIONS: UTILITY MAINTENANCE	+	−	−	+	+	−	−	−
UNDERPINNING	−	−	+	+	+	✓	✓	+
GROUND	−	+	O	O	O	+	+	+
EXCAVATION	+	+	−	−	−	+	+	+
BRACING	+	+	−	−	−	+	+	✓
DEPENDENCE ON SOIL STAND-UP TIME	−	+	+	O	−	+	+	+
DEPENDENCE ON WORKMEN SKILL	−	+	−	−	−	+	+	+
EFFECT OF INCREASED DEPTH.	+	−	−	−	−	+	+	+
INCORPORATION INTO STRUCTURE	O	−	−	+	−	+	+	✓

✓ BEST SOLUTION
+ POSSIBLE ALTERNATE SOLUTION
− LESS DESIRABLE
O UNDESIRABLE

Figure 12

3.4 BRACING GROUND WALL SUPPORT

The discussion in Section 2.8 effectively narrowed the choice of bracing to internal bracing consisting of struts and wales, and tieback earth anchors. While additional options are shown in the table of Figure 13, their usefulness is limited to special situations, and the table reinforces the conclusions reached in the discussion.

Actually the tieback anchors might not be used in the wider cut sections for the typical street section shown on Figure 1, because of possible interference with the deep basement. While internal bracing has been the accepted method for most cut-and-cover work in the past, tiebacks have been used increasingly in the last ten years. Materials and techniques have improved and it is to be expected that this method will be used more frequently in the future. For this reason it will be considered in this study as an alternate to internal bracing for all conditions as if the interferences did not exist.

G. BRACING GROUND WALL SUPPORT

FACTORS IN CHOOSING GROUND SUPPORT BRACING SYSTEM	TIEBACK EARTH ANCHORS	INTERNAL BRACING			ALTER DESIGN OF WALL SUPPORT	
		STRUTS ONLY	STRUTS & WALES	RAKERS & WALES	SIMPLE SPAN	CANTILEVER
COMPATIBLE WITH GROUND SUPPORT: SOLDIER PILES & LAGGING	+	+	+	+	–	–
CAST-IN-PLACE CONCRETE DIAPHRAGM	+	+	+	+	+	+
PRECAST CONCRETE DIAPHRAGM	+	+	+	+	+	+
EFFECT ON: EXCAVATION	+	–	–	O	+	✓
USE OF DECKING	+	–	–	–	+	O
BUILDING STRUCTURE	+	–	+	–	+	+
ADJACENT PROPERTY	–	–	+	–	–	O
INCREASED DEPTH	–	+	+	O	O	O
STRUCTURAL STABILITY	+	–	+	–	+	–
SUITABLE TO: VARIOUS SOILS	–	+	✓	+	+	–
CONTROL OF GROUND WATER	–	+	+	+	+	–
ADAPTABLE TO VARIOUS STRUCTURE SHAPES	✓	O	+	–	+	O

✓ BEST SOLUTION
+ POSSIBLE ALTERNATE SOLUTION
– LESS DESIRABLE
O UNDESIRABLE

Figure 13

3.5 EXCAVATION

In section 2.9 it was suggested that general excavations be considered in lifts. The first lift consists of a depth down from the street sufficient to place decking beams and uncover most utilities; the second would continue to a depth of about 20 feet (6.1m), sufficient to install the first set of bracing. The remaining lifts will be considered together as they will be excavated in the same manner. Different criteria govern the first two lifts and the methods most efficient for these do not govern down below. This can be seen graphically in the table of Figure 14.

The primary reason for this is that the first two lifts are governed by interferences: utility exploration; utility physical interference; placing deck beams, hanging utilities and then working directly below the deck beams. There is no advantage to using high production geared equipment which cannot operate efficiently. Once these hurdles have been passed, improved progress can be made by using production type methods. Below the second lift, excavation is limited by the efficiency of the methods and equipment chosen, and the coordination with placing bracing.

H. EXCAVATION									
EXCAVATION OPTIONS / SITE CONDITIONS	BACKHOE EXCAVATING AND LOADING TRUCKS		F.E.L. EXCAVATING AND LOADING TRUCKS		CLAMSHELL LIFT & LOAD TRUCKS W/W.O. HOPPER		DOZER AND/OR F.E.L. EXCAVATING BELOW		
	ON DECK (STREET)	IN CUT	ON DECK (STREET)	IN CUT	CLAMSHELL EXCAVATING	DOZER/F.E.L. EXCAVATING	LIFT BY CONVEYOR TO HOPPER	LIFT BY HYDRAULIC TO HOPPER	LIFT BY PNEUMATIC TO HOPPER
FIRST LIFT: SUPPORT: PILES & LAGGING	+	✓	+	+	–	–	0	0	0
DIAPHRAGM WALL	✓	+	+	+	–	–	0	0	0
BRACING (N.A.) SOIL: SAND/GRAVEL	+	✓	+	+	–	–	0	0	0
CLAY	+	✓	+	+	–	–	0	0	0
WET	+	✓	+	+	–	–	0	0	0
SECOND LIFT: SUPPORT: PILES & LAGGING	+	+	✓	+	–	+	–	–	–
DIAPHRAGM WALL	+	+	✓	+	–	+	–	–	–
BRACING: DECKING	+	–	+	–	–	+	–	–	–
NONE	–	+	+	✓	–	–	–	–	–
SOIL: SAND/GRAVEL	+	+	+	+	–	+	–	–	–
CLAY	+	+	+	–	+	+	0	–	0
WET	+	+	+	–	+	+	0	–	0
REMAINING LIFTS: SUPPORT: PILES & LAGGING	N.A	–	–	+	+	✓	+	+	+
DIAPHRAGM WALL	N.A.	–	–	+	+	✓	+	+	+
BRACING: INTERNAL	N.A	0	–	0	+	✓	+	+	+
TIEBACKS	N.A	–	–	+	+	✓	–	–	–
SOIL: SAND/GRAVEL	N.A	–	–	+	+	✓	+	+	+
CLAY	N.A.	–	–	–	+	✓	0	–	0
WET	N.A.	–	–	–	+	✓	–	+	–

✓ BEST SOLUTION
+ POSSIBLE ALTERNATE SOLUTION
– LESS DESIRABLE
0 UNDESIRABLE

Figure 14

3.6 CONSTRUCTION OF PERMANENT STRUCTURE

The first portion of this study for optimizing cut-and-cover tunneling will be based on a conventional construction sequence: after completing excavation the structure is built from bottom to top sequentially, followed by backfill and restoration work. This is shown as the first major construction option in Figure 15. The other two options consist of varying degrees of inverted construction where overall job time and surface disruption is reduced by backfilling and restoring the surface simultaneously with building the structure below. These options will be compared to the conventional construction after the optimizing process is completed.

Various structural options are also included in this table and rated in accordance with the factors which affect this choice. It should be noted that only those options utilizing a structural diaphragm wall can be considered for the inverted construction methods. While this rating system cannot be considered definitive or quantified, it is interesting to note the advantages of these construction options for many of the factors considered.

I. CONSTRUCTION OF PERMANENT STRUCTURE												
CONSTRUCTION OPTIONS	CONVENTIONAL SEQUENCE - EXCAVATING TO INVERT-CONSTRUCT INVERT SLAB, WALLS, ROOF - BACKFILL						EXCAVATING TO INVERT-CONSTRUCT INVERT SLAB, ROOF BACKFILL & FINISH			CONSTRUCT ROOF-BACKFILL & EXCAVATE UNDER ROOF-INV. SLAB & FINISH		
STRUCTURAL OPTIONS / CHOICE FACTORS	R.C. WALLS & SLABS	R.C. WALLS & COMPOSITE SLABS	STEEL & JACK ARCH WALLS & SLABS	CAST-IN SLURRY WALLS & C.I.P. CONC. SLABS	PRECAST PANEL WALLS & C.I.P. CONC. SLABS	PRECAST PANEL WALLS & PRECAST PANEL SLABS	CAST-IN SLURRY WALLS & C.I.P. CONC. SLABS	PRECAST PANEL WALLS & C.I.P. CONC. SLABS	PRECAST PANEL WALLS & PRECAST PANEL SLABS	CAST-IN SLURRY WALLS & C.I.P. CONC. SLABS	PRECAST PANEL WALLS & C.I.P. CONC. SLABS	PRECAST PANEL WALLS & PRECAST PANEL SLABS
SURFACE DISRUPTION	−	−	−	−	−	−	+	+	+	+	+	✓
PROJECT TIME	+	+	−	+	+	+	+	+	+	+	+	✓
EXCAVATION	+	+	+	+	+	+	+	+	+	−	−	−
STRUCTURE WORK	+	+	−	+	+	✓	+	+	+	+	+	+
WATERPROOFING	+	+	−	+	+	+	+	+	✓	+	+	✓
USE OF DECKING	+	+	+	+	+	−	+	+	+	−	−	−
LONG SPAN REQ'D	−	✓	+	+	−	+	+	−	+	+	−	+
SHALLOW STRUCTURES	+	+	+	+	+	+	+	+	+	+	+	✓
DEEP STRUCTURES	+	+	−	+	+	+	+	+	+	0	0	0

✓ BEST SOLUTION
+ POSSIBLE ALTERNATE SOLUTION
− LESS DESIRABLE
0 UNDESIRABLE

Figure 15

SECTION 4

4.0 SELECTION AND APPLICATION OF DESIGN GUIDELINES

The assumed project site used in this study is representative of urban or city conditions. This section of the report addresses itself to the practical design of permanent and temporary subsurface structures in this type of setting.

Figure 1 in Section 1 shows the typical urban site cross-section selected for this study. The vertical and horizontal relationship of adjacent building foundations to assumed cut-and-cover structures is shown on Figures 2, 3, and 4. Ordinarily the design of permanent and temporary cut-and-cover structures are not markedly affected by adjacent buildings. However, the total cost of a given cut-and-cover project can be significantly affected by the extent of needed underpinning or other permanent protection of adjacent structures.

The criteria for analysis and design used here for both permanent and temporary structures are representative of current practice. Changes in soil type are not likely to change the direct construction cost of permanent structures as much as they will change the cost of temporary structures (i.e. ground wall support systems, internal bracing, etc.). Functional and architectural considerations remain large variables with respect to cost of permanent structures. Permanent structures known to be representative of cut-and-cover structures actually built in soils equivalent to the study soil type have been adopted as described in Section 1. The dimensions of these structures and related items which are a function of construction cost (i.e. class of concrete, form ratio, reinforcement ratio, structural steel quantities, etc.) will conform to modern, well defined structural engineering criteria and practice. Criteria for analysis and design for the types of temporary structures associated with this study are less clearly defined in currently available texts and manuals. The specific criteria for analysis and design of the particular temporary structures adopted for this research effort are therefore discussed in this section.

4.1 SOIL TYPE

In order to develop meaningful costs for this study it is necessary that the characteristics of an assumed soil type be clearly defined. The cost of the construction of cut-and-cover structures can vary significantly with the type of soil in which the structure is built. The selection of a uniformly typical medium compact granular soil (hereinafter "study soil type") should give results representative of a majority of the cut-and-cover construction cases which might be encountered. In addition, it should be practicable to extract useful results for cases of proposed cut-and-cover construction in soils significantly different from the study soil type, by making appropriate adjustments for these differences. Nevertheless, the limitations of this, or any similar research, as they apply to soil type, need to be understood. These limitations are discussed qualitatively throughout this section. The study soil type has the following physical properties:

Angle of internal friction (ϕ) ϕ = 33^{o}

Cohesion (c) c = 0

Moist unit weight (γ) γ = 120 pcf

Submerged unit weight (γ') γ' = 62 pcf

Coefficient of active lateral earth pressure (Ka):

$Ka = \tan^2 (45 - \frac{\phi}{2})$ = 0.295

Coefficient of at rest lateral earth pressure (Ko): often taken as (1 - Sin ϕ), but for this soil type the more common assumption would be Ko = 0.50

Unit weight of water (γ_w) γ_w = 63 pcf

Coefficient of passive lateral pressure (Kp):

Kp = 6.25

This would be a common assumption taken from Reference 16, Page 462, for $\delta < -\frac{\phi}{2}$, to $-2/3\phi$, where (δ) = Angle of wall friction. This value of Kp should be considered an ultimate value. In a system where passive resistance is a resisting element an appropriate factor of safety needs to be applied.

Most soils that are encountered in cut-and-cover construction will have some cohesive properties. Nevertheless, it is common practice for a broad range of silty sands, clayey sands, sandy silts, sandy clays and even clayey silts to assume an "equivalent" non-cohesive soil for design purposes. The ordinary rules of soil mechanics for a non-cohesive soil (using an equivalent ϕ) are then applied to develop applicable lateral earth pressures for design of permanent and particularly temporary structures. In addition, many medium stiff to stiff clays, when analyzed as cohesive soils ($\phi = 0$), can impart active lateral soil pressures to temporary structures of roughly the same magnitude as would be the case for this study soil type, depending upon the depth of the excavation. It is therefore evident that the selected soil type used here should be representative of a majority of cut-and-cover cases as noted above. For cut-and-cover structures to be constructed in soft clays, stiff to hard fissured clays, or very loose sands, portions of the results of this study may have to be modified.

4.2 EXTENT OF PERMANENT PROTECTION OF ADJACENT STRUCTURES

"Permanent Protection of Adjacent Structures" (hereinafter in this section 4.3 "permanent protection") is defined for this report as underpinning or equivalent protection of adjacent structure foundations. For this study this definition of permanent protection is necessary because certain ground wall support systems (e.g. soldier pile and tremie concrete walls) can be designed to offer permanent protection of adjacent foundations in lieu of underpinning.

Figures 16-2 and 16-3 show the general criteria selected for this study to determine the need for permanent protection of adjacent foundations. This criterion is representative of that developed for BARTD and WMATA cut-and-cover projects for soil types similar or equivalent to the study soil type assumed here. Permanent support will not be assumed for buildings three stories high or less, given the assumed soil type and typical urban cross-section, but tempory protection will be provided. Occasionally, three story buildings would be underpinned or otherwise permanently protected, given these study conditions. Heavy old brick or masonry buildings might, for example, be underpinned. Based on this criteria the need for permanent protection for the cut-and-cover cases selected for this study would be as follows:

Cut-and-Cover Structure		Permanent Protection 10 Story Building	Permanent Protection 5 Story Building
Figure 2			
Highway Tunnel	- 30 ft. depth	No	No
Highway Tunnel	- 50 ft. depth	No	Yes
Highway Tunnel	- 70 ft. depth	Yes	Yes
Figure 3			
Double Box R.T.	- 30 ft. depth	No	No
Double Box R.T.	- 50 ft. depth	No	No
Double Box R.T.	- 70 ft. depth	No	No
Figure 4			
Rapid Transit Station	- 50 ft. depth	No	Yes
Rapid Transit Station	- 70 ft. depth	Yes	Yes

Buildings 3 stories high or less: No permanent protection.

The criteria for analysis and design of permanent protection (on which estimated costs of permanent protection will be based) will conform to the rules of well established conventional structural engineering practice.

It is evident that the costs of permanent protection this study will be sensitive to variable depth and widths of permanent cut-and-cover structures. The urban site conditions with respect to adjacent structures are necessarily fixed. For an actual cut-and-cover case, the cost of permanent protection will also depend upon the vertical and horizontal relationship of adjacent structure foundations to the cut-and-cover structure to be constructed, upon the size of adjacent buildings and upon the number of adjacent buildings. In actual cut-and-cover cases appropriate adjustments to the results of this research effort would need to be made, given significant differences in these latter variables. In most cases, however, such adjustments would, as a practical matter, be limited to the cost of permanent protection.

4.3 DESIGN OF PERMANENT STRUCTURES

Apart from functional and architectural variables the major system variables affecting the design of permanent cut-and-cover structures are the following:

soil type
depth of structure
surcharge loads
location of permanent water table

Figure 16.1 shows the long term loadings on which the design of the permanent structures is based. It is seen that the soil parameters assigned to the assumed soil type are basic values in determining horizontal and vertical loadings due to retained or supported soil. Nevertheless, the design of these structures is comparatively insensitive to changes in soil type or to other variables for the following reasons:

a) At most cut-and-cover sites the permanent water table is relatively near the surface; and if the permanent water table is high any change in soil type or its competency will not result in a commensurate change in design vertical and lateral loads on the permanent structures. Vertical loads, exclusive of surcharge, almost always consist of non-cohesive backfill -- so that this load need not be considered a variable if the water table is high. Even if the permanent water table is low, the difference in vertical loading between the two conditions would only be the difference between moist unit backfill weight and saturated unit backfill weight -- which for this study would be insignificant. Lateral loads (exclusive of surcharge) will vary somewhat with changes in

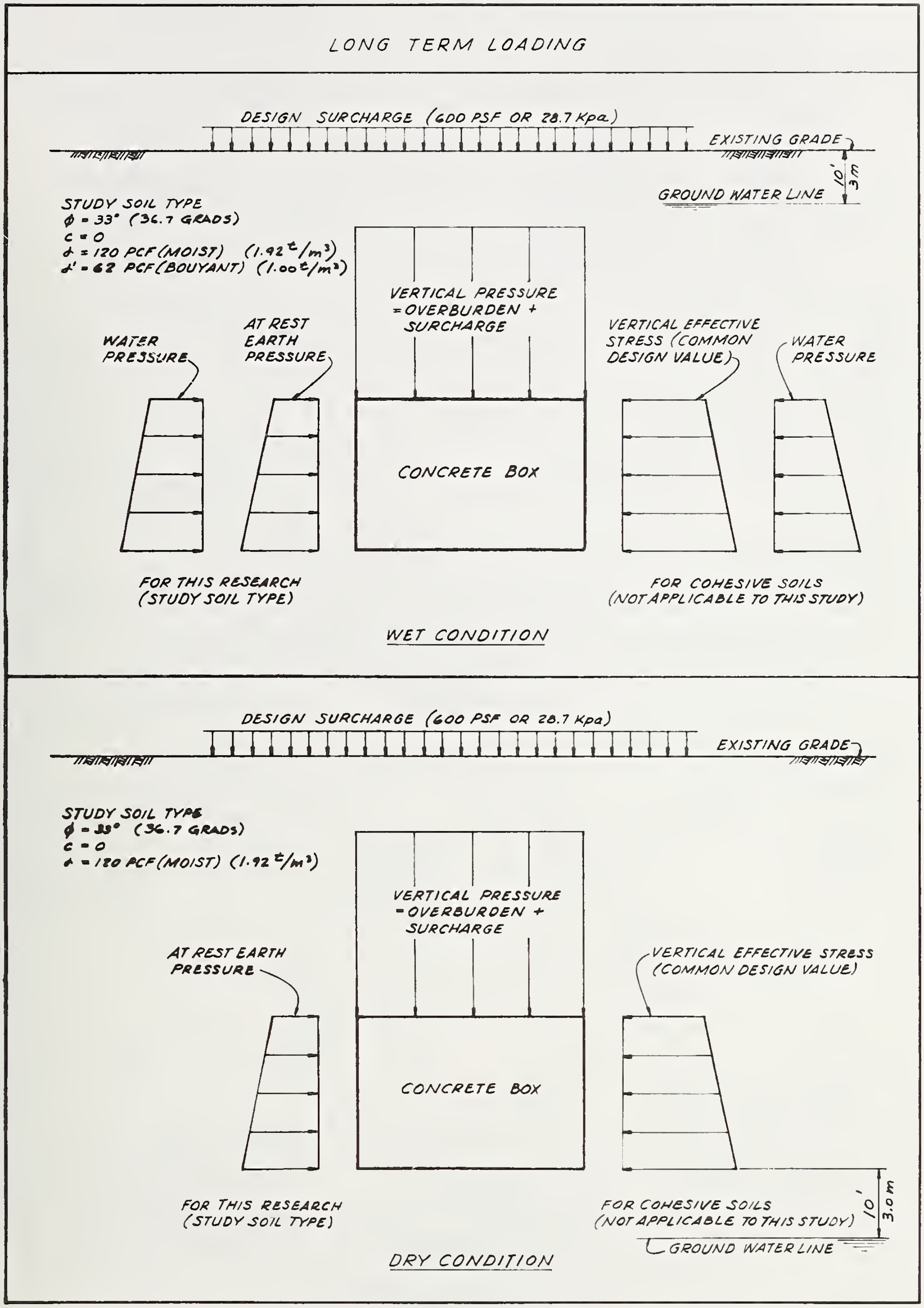


Figure 16 - Sheet 1

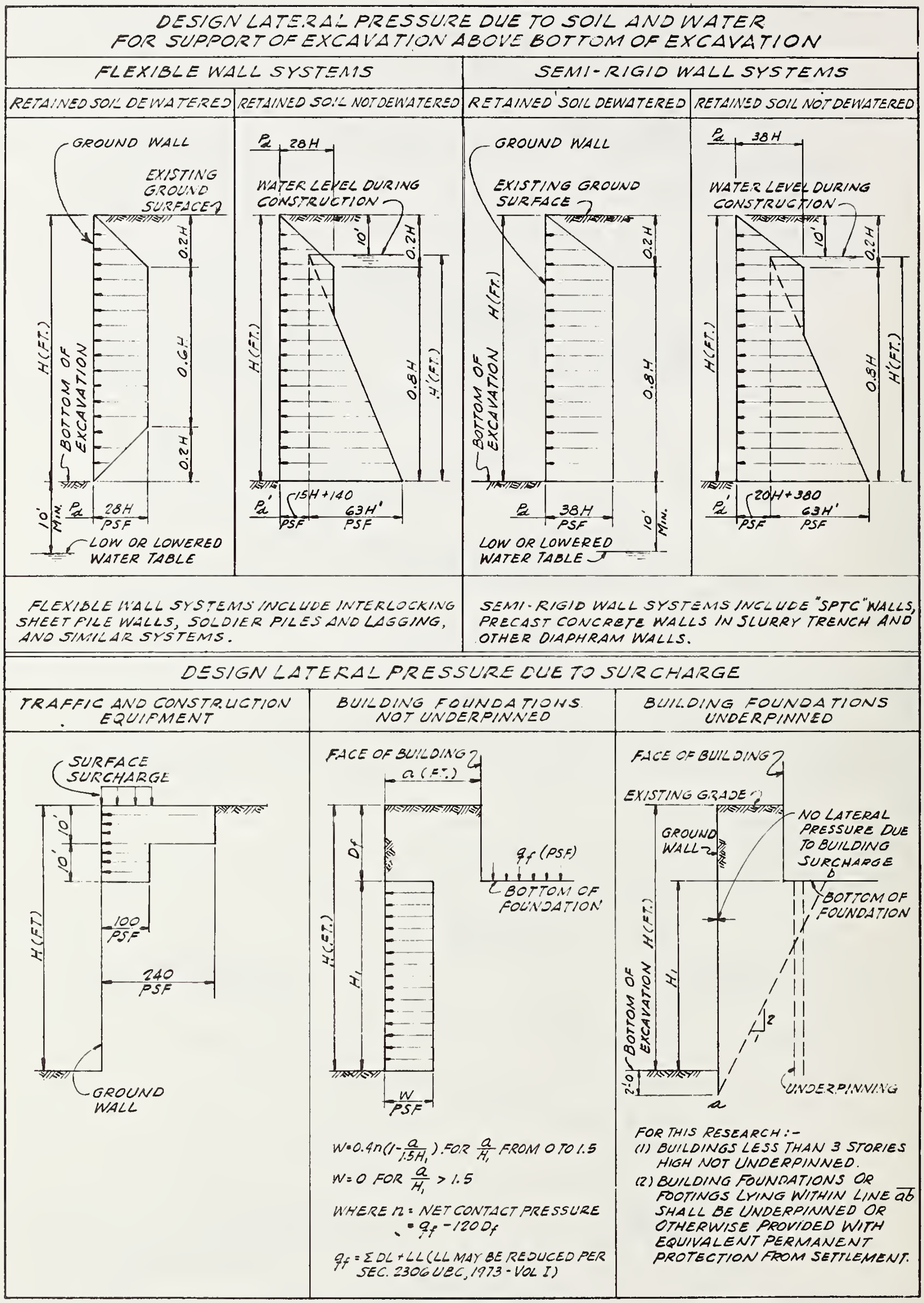


Figure 16 - Sheet 2

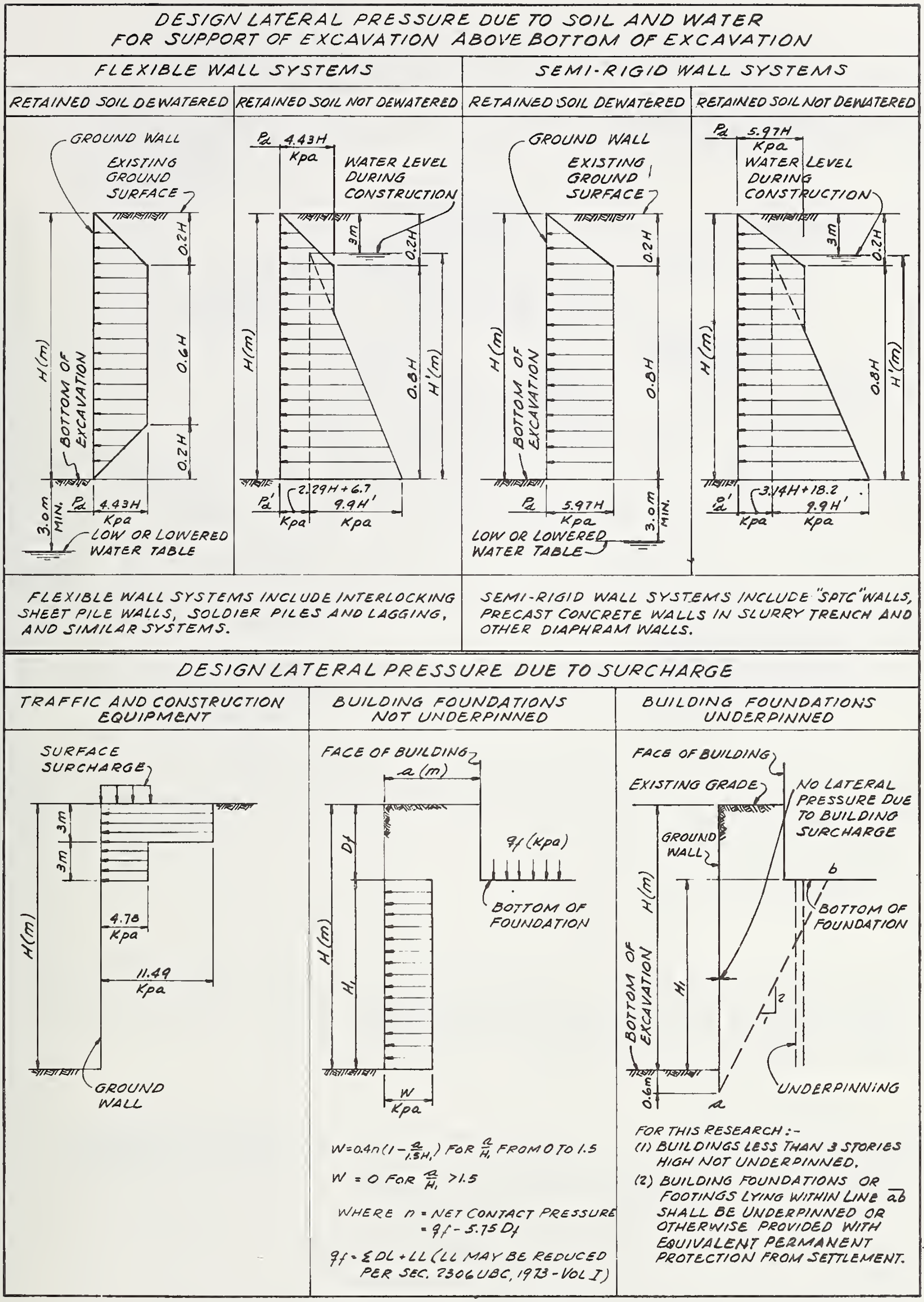
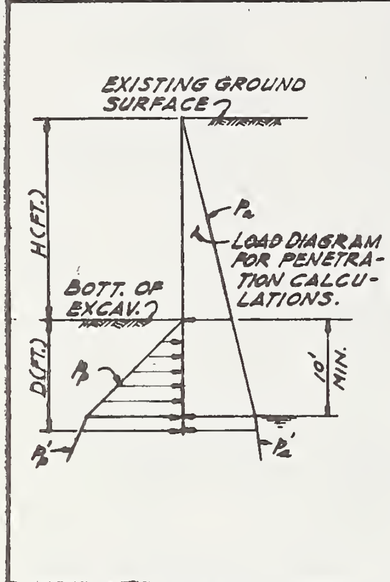


Figure 16 - Sheet 3

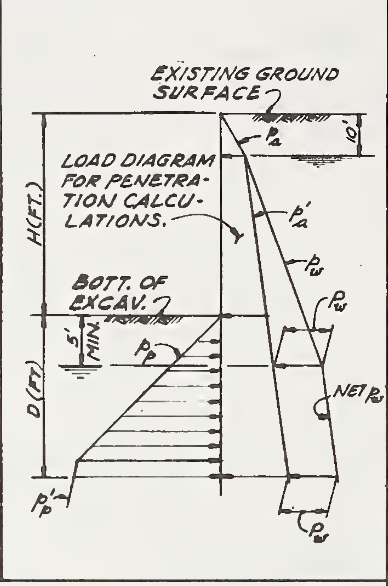
DESIGN PASSIVE RESISTANCE AND DESIGN VERTICAL BEARING CAPACITY OF SOIL BELOW BOTTOM OF EXCAVATION

DESIGN PASSIVE RESISTANCE

RETAINED SOIL DEWATERED

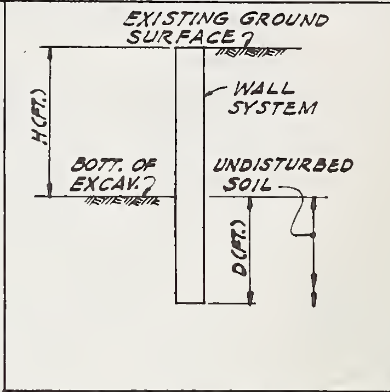


RETAINED SOIL NOT DEWATERED



DESIGN VERTICAL BEARING CAPACITY

RETAINED SOIL DEWATERED OR NOT DEWATERED



WALL SYSTEM	DESIGN VERTICAL SOIL SUPPORT		
	BEARING CAPACITY	UNITS	REMARKS
SOLDIER PILES AND LAGGING	K(b+d)D	KIPS PER PILE	d (FT.) b (FT.)
CONTINUOUS CONC. WALL	K(B+D)	KIPS PER FT. OF WALL	B = AVG. WIDTH OF WALL

P_a = 35 PSF PER FT. OF DEPTH
P_a' = 18 PSF PER FT. OF DEPTH
P_p = 500 PSF PER FT. OF DEPTH
P_p' = 250 PSF PER FT. OF DEPTH
P_w = 63 PSF PER FT. OF DEPTH
D = PENETRATION BELOW BOTTOM OF EXCAVATION

NOTES:

1. IF SOLDIER PILE IS SET IN CONCRETE, SUBSTITUTE 3.14 R FOR (b+d).
   R = RADIUS OF CONCRETE ENCASEMENT.
2. $K = H/25$

SOIL PROPERTIES

SANDY SOIL - MEDIUM DENSE
$\phi = 33°$ $C = 0$
γ (MOIST) = 120 PCF
γ' (SUBMERGED) = 62 PCF
γ_w = 63 PCF

DECK STRUCTURE LOADS

LOADS FOR ROADWAY DECK STRUCTURES SHALL BE TAKEN FROM THE STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS, 1973.

BASIC UNIT STRESSES

1. STRUCTURAL STEEL SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS, AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) 1970.
2. REINFORCED CONCRETE - BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI 318-63) WORKING STRESS DESIGN.
3. TIMBER - UNIFORM BUILDING CODE, 1973 EDITION, VOL. I.

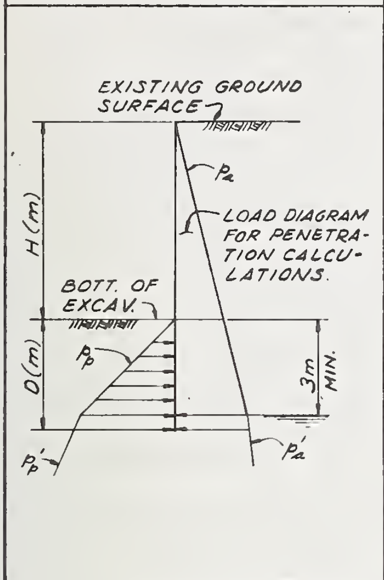
REFER TO SECTION 4.4.1(m) FOR ALLOWABLE STRESS INCREASES FOR TEMPORARY STRUCTURES

Figure 16 - Sheet 4

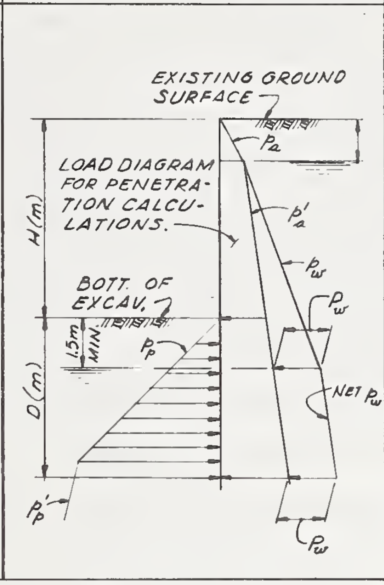
DESIGN PASSIVE RESISTANCE AND DESIGN VERTICAL BEARING CAPACITY OF SOIL BELOW BOTTOM OF EXCAVATION

DESIGN PASSIVE RESISTANCE

RETAINED SOIL DEWATERED



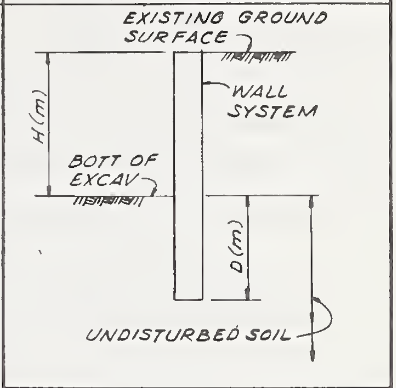
RETAINED SOIL NOT DEWATERED



P_a = 5.5 Kpa PER m OF DEPTH
P_a' = 2.8 Kpa PER m OF DEPTH
P_p = 78.6 Kpa PER m OF DEPTH
P_p' = 39.3 Kpa PER m OF DEPTH
P_w = 9.9 Kpa PER m OF DEPTH
D = PENETRATION BELOW BOTTOM OF EXCAVATION

DESIGN VERTICAL BEARING CAPACITY

RETAINED SOIL DEWATERED OR NOT DEWATERED



WALL SYSTEM	DESIGN VERTICAL SOIL SUPPORT		
	BEARING CAPACITY	UNITS	REMARKS
SOLDIER PILES AND LAGGING	K(b+d)D	KN PER PILE	d(m) b(m)
CONTINUOUS CONC. WALL	K(D+B)	KN PER m OF WALL	B = AVG WIDTH OF WALL

NOTES:

1. IF SOLDIER PILE IS SET IN CONCRETE, SUBSTITUTE 3.14 R FOR (b+d)
   R = RADIUS OF CONCRETE ENCASEMENT
2. K = 6.3H

SOIL PROPERTIES

SANDY SOIL - MEDIUM DENSE
ϕ = 36.7 GRADS, C = 0
γ (MOIST) = 1.92 t/m³
γ' (SUBMERGED) = 1.00 t/m³
γ_w = 1.00 t/m³

DECK STRUCTURE LOADS

LOADS FOR ROADWAY DECK STRUCTURES SHALL BE TAKEN FROM THE STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS. 1973.

BASIC UNIT STRESSES

1. STRUCTURAL STEEL SPECIFICATION FOR THE DESIGN, FABRICATION AND ERECTION OF STRUCTURAL STEEL FOR BUILDINGS, AMERICAN INSTITUTE OF STEEL CONSTRUCTION (AISC) 1970.
2. REINFORCED CONCRETE - BUILDING CODE REQUIREMENTS FOR REINFORCED CONCRETE (ACI318-63) WORKING STRESS DESIGN.
3. TIMBER - UNIFORM BUILDING CODE, 1973 EDITION, VOL. I.

REFER TO SECTION 4.4.1(m) FOR ALLOWABLE STRESS INCREASES FOR TEMPORARY STRUCTURES

Figure 16 - Sheet 5

soil type. An index of design lateral pressure, given the study soil type and a high water table would be $K_o \gamma' + \gamma_w$ = 0.50 (62) + 63 = 94. An index of design lateral pressure given a cohesive soil type might be $\gamma' + \gamma_w$ = 62 + 63 = 125 (but more likely less). A change from non-cohesive soil (the study soil type) to cohesive soil might therefore result in lateral pressure on the permanent structure walls 33% higher. However, the actual effect on the permanent structure walls would be much less due to other structural considerations, and due to buoyancy considerations which often control the selection of the physical dimensions of the structure. The above values are illustrative and are based on assumed soil parameters. Less competent sands could be compared with more competent cohesive soils (or vice versa) -- and the results (i.e. the design of permanent structures) would not change significantly -- given a high water table. Even if the permanent water table is low, if the soil type is cohesive, the design lateral pressure for long term loading would ordinarily be equal to or approach vertical soil stress (γ = 120 pcf, index = 120) so that the design of the permanent structures in this case also would not change markedly from the high water table design. For the purposes of this study only the unlikely combination of low permanent water table and non-cohesive soil type results in significant changes in the design of assumed permanent cut-and-cover structures. Figures 2, 3, and 4 show structural dimensions applicable to both high and low permanent water tables for each of the study cases -- based on the study soil type selected for this research. These dimensional differences would be less apparent if the soil type were cohesive (which is implicit from the above text). For a cohesive soil type the high permanent water table dimensions would be more typical -- regardless of the location of the water table.

b) Surcharge loads to be assumed above the permanent cut-and-cover structures need not be considered a variable for this study. A common design value for street and sidewalk live loads over cut-and-cover permanent structures is 600 psf. This value has been assumed appropriate for this study.

c) Building surcharge loads do not ordinarily impart significant additional lateral soil pressure to the walls of permanent cut-and-cover structures, unless the buildings in question are not underpinned, are relatively tall (heavy), and are unusually close to the plane of the cut-and-cover structure walls. To compute building surcharge effects it is usual practice to develop empirical relationships which can be used conveniently for most types of building surcharge problems and which will give results on the side of safety. The empirical formula shown on Figures 16-2 and 16-3 may be used with safety for computations of lateral soil pressure due to building surcharge on either permanent or temporary structures -- given the study soil type selected. For the cases selected the effect of building surcharge on lateral pressures imparted to permanent structure walls is almost negligible -- ranging from an increase in lateral pressure of 0% (most cases) to 10% (a few cases).

To develop the basic design of the cut-and-cover permanent structures shown on Figures 2, 3, and 4, many similar structures (i.e. structures constructed under similar site conditions) were studied. Tentative designs were then chosen which conformed generally to the "as built" structures which were studied. These tentative designs were then checked structurally to be certain that they conformed to the design criteria. Modifications were made as required and the final designs adopted.

4.4 DESIGN OF TEMPORARY STRUCTURES

Figures 17-1 through 17-5 show the major material types and quantities which will be used to estimate the cost of temporary structures. In order to develop this data, a comprehensive design effort was required -- utilizing what might be described as "current practice in the design of braced land cofferdams and construction decking." The general subject of braced land cofferdams, particularly, has been discussed extensively in several standard texts on soil

GROUND SUPPORT AND BRACING SYSTEMS SOLDIER PILES AND LAGGING WITH INTERNAL BRACING OR TIEBACKS WET OR DRY SOIL CONDITIONS								
			4 LANE HIGHWAY TUNNEL RAPID TRANSIT STATION (3)			DOUBLE BOX LINE STRUCTURE		
			H=30'	H=50'	H=70'	H=30'	H=50'	H=70'
GROUND SUPPORT SYSTEM		PILES	W24×84	W24×130 (5)	W24×160 (V-50) (6)	W24×84	W24×130 (5)	W24×100 (8)
		PILE SPACING	8'-0" C.C.	8'-0" C.C.	8'-0" C.C.	8'-0" C.C.	8'-0" C.C.	8'-0" C.C.
		LAGGING D.F.	3× MATERIAL TO -20' ELEV., 4× MATERIAL -20' ELEV. TO SUBGRADE					
DECK FRAMING		DECK BEAMS (1)	W36×230	W36×230	W36×230	W36×230	W36×230	W36×230
		CAP BEAMS, INTERIOR	W30×116	W30×116	W30×116	—	—	—
		CAP BEAMS, EXTERIOR	W14×68	W14×68	W14×68	W14×119	W14×119	W14×119
		INTERIOR PILES (2)	HP14×73	HP14×73	HP14×73	—	—	—
		CAP WALES	W30×99	W30×99	W30×99	W30×99	W30×99	W30×99
		LATERAL BRACING	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5
		DECKING D.F.	12×12	12×12	12×12	12×12	12×12	12×12
INTERNAL BRACING ALTERNATE	LEVEL NUMBER 2	WALES	W14×74	W30×116	W36×135	W16×88	W30×116	W33×130
		STRUTS (1)	HP14×73	W14×111	W14×142	HP14×73	W14×103	W14×111
	LEVEL NUMBER 3	WALES	—	W30×99	W36×160	—	W30×99	W30×108
		STRUTS (1)	—	W14×87	W14×150	—	W14×87	W14×111
	LEVEL NUMBER 4	WALES	—	—	W30×99	—	—	W30×116
		STRUTS (1)	—	—	W14×103	—	—	W14×111
	LEVEL NUMBER 5	WALES	—	—	W24×76	—	—	W24×76
		STRUTS (1)	—	—	W14×87	—	—	HP14×73
TIEBACK ALTERNATE	LEVEL NUMBER 2	WALES	JC12×25	JC18×58	JC18×42.7	JC12×25	JC18×58	JC18×42.7
		TIEBACKS (4)	1¼"φ 120/150 (7)	1¼"φ 155/190	1¼"φ 120/150	1¼"φ 120/150 (7)	1¼"φ 155/190	1¼"φ 120/150
	LEVEL NUMBER 3	WALES	—	JC18×42.7	JC18×42.7	—	JC18×42.7	JC18×42.7
		TIEBACKS (4)	—	1¼"φ 120/150	1¼"φ 120/150	—	1¼"φ 120/150	1¼"φ 120/150
	LEVEL NUMBER 4	WALES	—	—	JC18×51.9	—	—	JC18×51.9
		TIEBACKS (4)	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
	LEVEL NUMBER 5	WALES	—	—	JC15×40	—	—	JC15×40
		TIEBACKS (4)	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
	LEVEL NUMBER 6	WALES	—	—	JC12×30	—	—	JC12×30
		TIEBACKS (4)	—	—	1¼"φ 120/150 (7)	—	—	1¼"φ 120/150 (7)
PENETRATION BELOW SUBGRADE		INTERNAL BRACING ALTERNATE	7'	11'	12'	7'	11'	12'
		TIEBACK ALTERNATE	10'	17'	27'	10'	17'	27'

(1) DECK BEAMS AND STRUTS ARE AT 12'-0" C.C. SPACING AND IN SAME VERTICAL PLANE

(2) INTERIOR PILES ARE AT 12'-0" C.C. SPACING AND OFFSET 2'-0" FROM ℄ OF DECK BEAM

(3) RAPID TRANSIT STATION NOT CONSIDERED AT H=30' BECAUSE OF STRUCTURAL HEIGHT.

(4) ALL TIEBACKS ARE DYWIDAG OR EQUAL. 120/150 REFERS TO GRADE OF STEEL.

(5) USE W24×145 PILE FOR TIEBACK ALTERNATE.

(6) USE W24×110 (A-36) PILE FOR TIEBACK ALTERNATE — RAPID TRANSIT STATION.
USE W24×160 (A-36) PILE FOR TIEBACK ALTERNATE — HIGHWAY TUNNEL.

(7) TIEBACKS NOTED AT 8'-0" C.C., ALL OTHERS AT 4'-0" C.C.

(8) USE W24×110 (A-36) FOR TIEBACK ALTERNATE.

ALL STEEL MEMEBERS ARE ASTM A-36 UNLESS NOTED.

Figure 17 - Sheet 1

GROUND SUPPORT AND BRACING SYSTEMS
S.P.T.C. WALL WITH INTERNAL BRACING OR TIEBACKS
WET SOIL CONDITION

		4 LANE HIGHWAY TUNNEL RAPID TRANSIT STATION ③			DOUBLE BOX LINE STRUCTURE		
		H=30'	H=50'	H=70'	H=30'	H=50'	H=70'
GROUND SUPPORT SYSTEM	PILES	W24×100	W36×135 ⑤	W36×230 ⑥	W24×100	W36×135 ⑤	W36×135 ⑥
	PILE SPACING	8'-0" C.C.	6'-0" C.C.	6'-0" C.C.	8'-0" C.C.	6'-0" C.C.	6'-0" C.C.
	LAGGING (D.F.)	3×MATERIAL TO -20' ELEV.					
DECK FRAMING	DECK BEAMS ①	W36×230	W36×230	W36×230	W36×230	W36×230	W36×230
	CAP BEAMS, INTERIOR	W30×116	W30×116	W30×116	—	—	—
	CAP BEAMS, EXTERIOR	W14×68	W14×68	W14×68	W14×119	W14×78	W14×78
	INTERIOR PILES ②	HP14×73	HP14×73	HP14×73	—	—	—
	CAP WALES	W30×99	W24×84	W24×84	W30×99	W24×76	W24×76
	LATERAL BRACING	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5
	DECKING (D.F.)	12×12	12×12	12×12	12×12	12×12	12×12
INTERNAL BRACING ALTERNATE: LEVEL NUMBER 2	WALES	W21×68	W33×130	W36×135	W21×68	W33×130	W33×130
	STRUTS ①	HP14×73	W14×142	W14×167	HP14×73	W14×127	W14×142
LEVEL NUMBER 3	WALES	—	W30×116	W36×194	—	W30×116	W33×118
	STRUTS ①	—	W14×136	W14×228	—	W14×111	W14×127
LEVEL NUMBER 4	WALES	—	—	W33×130	—	—	W36×160
	STRUTS ①	—	—	W14×158	—	—	W14×167
LEVEL NUMBER 5	WALES	—	—	W36×150	—	—	W36×150
	STRUTS ①	—	—	W14×176	—	—	W14×150
TIEBACK ALTERNATE: LEVEL NUMBER 2	WALES	][12×36	][18×58	][18×51.9	][12×35	][18×58	][18×51.9
	TIEBACKS ④	1¼"φ 155/190 ⑦	4-⅝"φ 205/230	1¼"φ 120/150	1¼"φ 155/190 ⑦	4-⅝"φ 205/230	1¼"φ 120/150
LEVEL NUMBER 3	WALES	—	][18×45.8	][18×51.9	—	][18×45.8	][18×51.9
	TIEBACKS ④	—	1¼"φ 155/190	1¼"φ 120/150	—	1¼"φ 155/190	1¼"φ 120/150
LEVEL NUMBER 4	WALES	—	—	][18×58	—	—	][18×58
	TIEBACKS ④	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
LEVEL NUMBER 5	WALES	—	—	II(S18×54.7)	—	—	II(S20×75)
	TIEBACKS ④	—	—	1¼"φ 155/190	—	—	6-⅝"φ 205/230
LEVEL NUMBER 6	WALES	—	—	II(S18×54.7)	—	—	II(S18×54.7)
	TIEBACKS ④	—	—	1¼"φ 155/190	—	—	1¼"φ 155/190
LEVEL NUMBER 7	WALES	—	—	][18×58	—	—	][18×58
	TIEBACKS ④	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
PENETRATION BELOW SUBGRADE	INTERNAL BRACING ALTERNATE	13'	30'	54'	15'	40'	66'
	TIEBACK ALTERNATE	13'	30'	54'	15'	40'	66'

① DECK BEAM AND STRUTS ARE AT 12'-0" C.C. SPACING AND IN SAME VERTICAL PLANE.
② INTERIOR PILES ARE AT 12'-0" C.C. SPACING AND OFFSET 2'-0" FROM ℄ OF DECK BEAM.
③ RAPID TRANSIT STATION NOT CONSIDERED AT H=30' BECAUSE OF STRUCTURAL HEIGHT.
④ ALL TIEBACKS ARE DYWIDAGS OR EQUAL. 120/150 REFERS TO GRADE OF STEEL.
⑤ USE W36×150 PILE FOR TIEBACK ALTERNATE.
⑥ USE W36×160 PILE FOR TIEBACK ALTERNATE - RAPID TRANSIT STATION & LINE SECTION
⑦ TIEBACKS NOTED AT 8'-0" C.C., ALL OTHERS AT 4'-0" C.C.
ALL STEEL MEMBERS ARE ASTM A-36 UNLESS NOTED

Figure 17 - Sheet 2

GROUND SUPPORT AND BRACING SYSTEMS
S.P.T.C. WALL WITH INTERNAL BRACING OR TIEBACKS
DRY SOIL CONDITION

		4 LANE HIGHWAY TUNNEL RAPID TRANSIT STATION (3)			DOUBLE BOX LINE STRUCTURE		
		H = 30'	H = 50'	H = 70'	H = 30'	H = 50'	H = 70'
GROUND SUPPORT SYSTEM	PILES	W24×94	W30×124	W36×170 (5)	W24×100	W30×124	W30×108 (5)
	PILE SPACING	8'-0" C.C.	6'-0" C.C.	6'-0" C.C.	8'-0" C.C.	6'-0" C.C.	6'-0" C.C.
	LAGGING D.F.	3×MATERIAL TO -20' ELEV., 4×MATERIAL -20' ELEV. TO S.P.T.C. WALL					
DECK FRAMING	DECK BEAMS (1)	W36×230	W36×230	W36×230	W36×230	W36×230	W36×230
	CAP BEAMS, INTERIOR	W30×116	W30×116	W30×116	—	—	—
	CAP BEAMS, EXTERIOR	W14×68	W14×68	W14×68	W14×119	W14×78	W14×78
	INTERIOR PILES (2)	HP14×73	HP14×73	HP14×73	—	—	—
	CAP WALES	W30×99	W24×84	W24×84	W30×99	W24×76	W24×76
	LATERAL BRACING	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5
	DECKING D.F.	12×12	12×12	12×12	12×12	12×12	12×12
INTERNAL BRACING ALTERNATE — LEVEL NUMBER 2	WALES	W21×62	W30×124	W36×135	W21×62	W30×124	W33×130
	STRUTS (1)	HP14×73	W14×142	W14×167	HP14×73	W14×119	W14×142
LEVEL NUMBER 3	WALES	—	W30×99	W36×150	—	W30×99	W30×116
	STRUTS (1)	—	W14×103	W14×184	—	W14×87	W14×119
LEVEL NUMBER 4	WALES	—	—	W30×99	—	—	W33×118
	STRUTS (1)	—	—	W14×127	—	—	W14×127
LEVEL NUMBER 5	WALES	—	—	W30×99	—	—	W30×108
	STRUTS (1)	—	—	W14×127	—	—	W14×111
TIEBACK ALTERNATE — LEVEL NUMBER 2	WALES	][12×35	][18×51.9	][18×51.9	][12×35	][18×51.9	][18×51.9
	TIEBACKS (4)	1¼"φ 155/190 (6)	4-5/8"φ 205/230	1¼"φ 120/150	1¼"φ 155/190 (6)	4-5/8"φ 205/230	1¼"φ 120/150
LEVEL NUMBER 3	WALES	—	][15×50	][18×51.9	—	][15×50	][18×51.9
	TIEBACKS (4)	—	1¼"φ 120/150	1¼"φ 120/150	—	1¼"φ 120/150	1¼"φ 120/150
LEVEL NUMBER 4	WALES	—	—	II 5 18×70	—	—	][18×51.9
	TIEBACKS (4)	—	—	5-5/8"φ 205/230	—	—	1¼"φ 120/150
LEVEL NUMBER 5	WALES	—	—	][18×51.9	—	—	II 5 18×54.7
	TIEBACKS (4)	—	—	1¼"φ 120/150	—	—	4-5/8"φ 205/230
LEVEL NUMBER 6	WALES	—	—	][18×45.8	—	—	][18×45.8
	TIEBACKS (4)	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
LEVEL NUMBER 7	WALES	—	—	][13×31.8	—	—	][13×31.8
	TIEBACKS (4)	—	—	1¼"φ 155/190 (7)	—	—	1¼"φ 155/190 (7)
PENETRATION BELOW SUBGRADE	INTERNAL BRACING ALTERNATE	10'	10'	12'	10'	10'	12'
	TIEBACK ALTERNATE	10'	10'	12'	10'	10'	12'

(1) DECK BEAMS AND STRUTS ARE AT 12'-0" C.C. SPACING AND IN SAME VERTICAL PLANE.

(2) INTERIOR PILES ARE AT 12'-0" C.C. SPACING AND OFFSET 2'-0" FROM ℄ OF DECK BEAM.

(3) RAPID TRANSIT STATION NOT CONSIDERED

(4) ALL TIEBACKS ARE DYWIDAG OR EQUAL. 120/150 REFERS TO GRADE OF STEEL.

(5) USE W36×135 PILE FOR TIEBACK ALTERNATE - RAPID TRANSIT STATION & LINE SECTION

(6) TIEBACK AT 8'-0" C.C., ALL OTHERS AT 4'-0" C.C. UNLESS NOTED

(7) TIEBACK AT 6'-0" C.C.

ALL STEEL MEMBERS ARE ASTM A-36 UNLESS NOTED

Figure 17 - Sheet 3

GROUND SUPPORT AND BRACING SYSTEMS
PRECAST CONCRETE WALL WITH INTERNAL BRACING OR TIEBACKS — WET SOIL CONDITION

			4 LANE HIGHWAY TUNNEL RAPID TRANSIT STATION ③			DOUBLE BOX LINE STRUCTURE		
			H=30'	H=50'	H=70'	H=30'	H=50'	H=70'
GROUND SUPPORT SYSTEM		PRECAST PANELS	24×144	36×108	36×72	24×144	36×108	36×72
		PILES (ABOVE)	W14×78	W24×100	W27×160	W18×96	W30×172	W24×120
		PILE SPACING	6'-0" C.C.	6'-0" C.C.	6'-0" C.C.	6'-0" C.C.	6'-0" C.C.	6'-0" C.C.
		LAGGING (D.F.)	3× MATERIAL TO -20' ELEV.,					
DECK FRAMING		DECK BEAMS	W36×230	W36×230	W36×230	W36×230	W36×230	W36×230
		CAP BEAMS, INTERIOR	W30×116	W30×116	W30×116	—	—	—
		CAP BEAMS, EXTERIOR	W14×68	W14×68	W14×68	W14×78	W14×78	W14×78
		INTERIOR PILES ②	HP14×73	HP14×73	HP14×73	—	—	—
		CAP WALES	W24×84	W24×84	W24×84	W24×76	W24×76	W24×76
		LATERAL BRACING	WT8×10.5 WT6×13.5	WT8×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT8×10.5 WT6×13.5	WT8×10.5 WT6×13.5
		DECKING (D.F.)	12×12	12×12	12×12	12×12	12×12	12×12
INTERNAL BRACING ALTERNATE	LEVEL NUMBER 2	WALES	W21×68	W33×130	W36×135	W21×68	W33×130	W21×68
		STRUTS ①	HP14×73	W14×142	W14×167	HP14×73	W14×127	W14×142
	LEVEL NUMBER 3	WALES	—	W30×116	W36×194	—	W30×116	W33×118
		STRUTS ①	—	W14×136	W14×228	—	W14×111	W14×127
	LEVEL NUMBER 4	WALES	—	—	W33×130	—	—	W36×160
		STRUTS ①	—	—	W14×158	—	—	W14×167
	LEVEL NUMBER 5	WALES	—	—	W36×150	—	—	W36×150
		STRUTS ①	—	—	W14×176	—	—	W14×150
TIEBACK ALTERNATE	LEVEL NUMBER 2	WALES	][C12×35	][C18×58	][C18×51.9	][C12×35	][C18×58	][C18×51.9
		TIEBACKS ④	1¼"φ 155/190	4-¾"φ 205/230	1¼"φ 120/150	1¼"φ 155/190	4-¾"φ 205/230	1¼"φ 120/150
	LEVEL NUMBER 3	WALES	—	][C18×45.8	][C18×51.9	—	][C18×45.8	][C18×51.9
		TIEBACKS ④	—	1¼"φ 155/190	1¼"φ 120/150	—	1¼"φ 155/190	1¼"φ 120/150
	LEVEL NUMBER 4	WALES	—	—	][C18×58	—	—	][C18×58
		TIEBACKS ④	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
	LEVEL NUMBER 5	WALES	—	—	II(S18×54.7)	—	—	II(S18×54.7)
		TIEBACKS ④	—	—	1¼"φ 155/190	—	—	1¼"φ 155/190
	LEVEL NUMBER 6	WALES	—	—	II(S18×54.7)	—	—	II(S18×54.7)
		TIEBACKS ④	—	—	1¼"φ 155/190	—	—	1¼"φ 150/190
	LEVEL NUMBER 7	WALES	—	—	][C18×58	—	—	][C18×58
		TIEBACKS ④	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
PENETRATION BELOW SUBGRADE ⑤		INTERNAL BRACING ALTERNATE	10'	12'	15'	10'	12'	15'
		TIEBACK ALTERNATE	10'	12'	16'	10'	12'	16'

① DECK BEAMS AND STRUTS ARE AT 12'-0" C.C. SPACING AND IN SAME VERTICAL PLANE.

② INTERIOR PILES ARE AT 12'-0" C.C. SPACING AND OFFSET 2'-0" FROM ℄ OF DECK BEAM.

③ RAPID TRANSIT STATION NOT CONSIDERED AT H=30' BECAUSE OF STRUCTURAL HEIGHT.

④ ALL TIEBACKS ARE DYWIDAG OR EQUAL. 120/150 REFERS TO GRADE OF STEEL. ALL TIEBACKS ARE 4'-0" C.C.

⑤ PENETRATIONS SHOWN ARE THOSE REQUIRED FOR PASSIVE RESISTANCE. TO PREVENT PIPING, THE SLOT EXCAVATED TO ALLOW PLACEMENT OF THE PRECAST PANELS MUST EXTEND BELOW SUBGRADE THE SAME DISTANCE AS SHOWN IN FIGURE 17, SHEET # 2. A SLOW SETTING CEMENT AND BENTONITE GROUT SLURRY FORMS THE CUTOFF WALL BELOW THE BOTTOM OF THE PRECAST PANEL.
ALL STEEL MEMBERS ARE ASTM-A36 UNLESS NOTED.

Figure 17 - Sheet 4

GROUND SUPPORT AND BRACING SYSTEMS
PRECAST CONCRETE WALL WITH INTERNAL BRACING OR TIEBACKS
DRY SOIL CONDITION

			4 LANE HIGHWAY TUNNEL RAPID TRANSIT STATION ③			DOUBLE BOX LINE STRUCTURE		
			H=30'	H=50'	H=70'	H=30'	H=50'	H=70'
GROUND SUPPORT SYSTEM		PRECAST PANELS	24×144	36×108	36×72	24×144	36×108	36×72
		PILES (ABOVE)	W14×78	W24×100	W27×160	W18×96	W30×172	W24×120
		PILE SPACING	6'-0"C.C.	6'-0"C.C.	6'-0"C.C.	6'-0"C.C.	6'-0"C.C.	6'-0"C.C.
		LAGGING (D.F.)	3×MATERIAL TO -20 ELEV., 4×MATERIAL -20 ELEV. TO PRECAST WALL					
DECK FRAMING		DECK BEAMS	W36×230	W36×230	W36×230	W36×230	W36×230	W36×230
		CAP BEAMS, INTERIOR	W30×116	W30×116	W30×116	—	—	—
		CAP BEAMS, EXTERIOR	W14×68	W14×68	W14×68	W14×78	W14×78	W14×78
		INTERIOR PILES ②	HP14×73	HP14×73	HP14×73	—	—	—
		CAP WALES	W24×84	W24×84	W24×84	W24×76	W24×76	W24×76
		LATERAL BRACING	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5	WT5×10.5 WT6×13.5
		DECKING (D.F.)	12×12	12×12	12×12	12×12	12×12	12×12
INTERNAL BRACING ALTERNATE	LEVEL NUMBER 2	WALES	W21×68	W30×124	W36×135	W21×68	W30×124	W33×130
		STRUTS ①	HP14×73	W14×142	W14×167	HP14×73	W14×119	W14×142
	LEVEL NUMBER 3	WALES	—	W30×99	W36×150	—	W30×99	W30×116
		STRUTS ①	—	W14×103	W14×184	—	W14×87	W14×119
	LEVEL NUMBER 4	WALES	—	—	W30×99		—	W33×118
		STRUTS ①	—	—	W14×127	—	—	W14×127
	LEVEL NUMBER 5	WALES	—	—	W30×99	—	—	W30×108
		STRUTS ①	—	—	W14×127	—	—	W14×111
TIEBACK ALTERNATE	LEVEL NUMBER 2	WALES	][12×35	][18×51.9	][18×51.9	][12×35	][18×51.9	][18×51.9
		TIEBACKS ④	1¼"φ 155/190	4-⅝"φ 205/230	1¼"φ 120/160	1¼"φ 155/190	4-⅝"φ 205/230	1¼"φ 120/180
	LEVEL NUMBER 3	WALES	—	][15×50	][18×51.9	—	][15×50	][18×51.9
		TIEBACKS ④	—	1¼"φ 120/150	1¼"φ 120/150	—	1¼"φ 120/150	1¼"φ 120/160
	LEVEL NUMBER 4	WALES	—	—	][18×51.9	—	—	][18×51.9
		TIEBACKS ④	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
	LEVEL NUMBER 5	WALES	—	—	][18×51.9	—	—	][18×51.9
		TIEBACKS ④	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
	LEVEL NUMBER 6	WALES	—	—	][18×45.8	—	—	][18×45.8
		TIEBACKS ④	—	—	1¼"φ 120/150	—	—	1¼"φ 120/150
	LEVEL NUMBER 7	WALES	—	—	][18×58	—	—	][18×58
		TIEBACKS ④	—	—	1¼"φ 155/190	—	—	1¼"φ 155/190
PENETRATION BELOW SUBGRADE		INTERNAL BRACING ALTERNATE	10'	10'	12'	10'	10'	12'
		TIEBACK	10'	10'	12'	10'	10'	12'

① DECK BEAMS AND STRUTS ARE AT 12'-0"C.C. SPACING AND IN SAME VERTICAL PLANE.

② INTERIOR PILES ARE AT 12'-0"C.C. SPACING AND OFFSET 2'-0" FROM ℄ OF DECK BEAM.

③ RAPID TRANSIT STATION NOT CONSIDERED AT H=30' BECAUSE OF STRUCTURAL HEIGHT.

④ ALL TIEBACKS ARE DYWIDAG OR EQUAL. 120/150 REFERS TO GRADE OF STEEL. TIEBACKS ARE AT 4'-0" C.C.

ALL STEEL MEMBERS ARE ASTM A-36 UNLESS NOTED

Figure 17 - Sheet 5

mechanics and foundation design -- and in certain design manuals published by industry and others. Nevertheless, there remain theoretical and practical considerations about which literature is at best obscure -- and about which there are differing opinions and practice in the engineering community. It is not within the scope of this research effort to attempt to advocate a uniform approach to the design of braced cofferdams. But specific criteria for design and analysis is presented and discussed herein -- in order that the basis for the results of this study can be both uniform and fully understood.

4.4.1 Criteria for Analysis and Design - Figures 16-2 through 16-5 show summary criteria for design and analysis of temporary structures applicable to this research. Figures 16-3 and 16-5 give the metric equivalent values to dimensions and empirical relationships presented in the English system in Figures 16-2 and 16-4 respectively. The design values, equations and specifications shown on these figures are discussed below:

a) Design Lateral Pressure, Flexible Wall Systems, Retained Soil Dewatered (Figure 16-2)

The magnitude of and shape of this lateral pressure diagram is based on the recommendations of Terzaghi and Peck, 1948 (Reference 12) for non-cohesive soil. Terzaghi and Peck, 1968, (Reference 13) recommend a modified lateral pressure diagram. But the 1948 diagram remains more common in current civil engineering practice and was therefore used for the study. The magnitude of lateral pressure may be taken as Pd where:

$$Pd = 0.8\ Ka\ \gamma\ H$$

$$Pd = (0.8)\ (0.295)\ (120)\ (H) = 28.3\ H\ (\text{say, } 28H)$$

where H is in ft. and the values of Ka and are taken from Section 4.1

b) Design Lateral Pressure, Flexible Wall Systems, Soil Not Dewatered (Figure 16-2)

This design condition is not common in cut-and-cover construction and is not applicable to any of the study cases incorporated into this research. This criteria would, however, be applicable to a case in which sheet piles were used for the wall system and the water table could not or would not be lowered. Values of Pd and P'd are also based on the recommendations of Terzaghi and Peck:

$$Pd = 0.8\ Ka\,\gamma\,H = 28\ H \text{ (in like manner)}$$

$$P'd = (0.8)\ Ka\ (10\gamma) + (0.8)(Ka)\ (H-10)\ (\gamma')$$

$$P'd = (0.8)\ (.295)\ (10)\ (120) + (0.8)\ (.295)\ (H-10)\ (62)$$

$$P'd = 14.6\ H + 137 \text{ (say, } 15H + 140\text{) psf}$$

It should be noted that the relationship for P'd above is applicable only to the specific case where the water table is 10 feet below grade. It is common practice, however, to develop formulae for specific cases like these.

c) Design Lateral Pressure, Semi-Rigid Wall Systems, Retained Soil Dewatered (Figure 16-2)

This type of wall system has only been used for about twenty years and appropriate design criteria is not included in standard texts.

The magnitude and shape of an appropriate lateral pressure diagram for this type of design condition is a matter of good engineering judgement for each specific case. For this study soil type the following relationship was used:

$$Pd = (0.4)\ (K_o + K_a)\ (\gamma)\ H$$

$$Pd = (0.4)\ (0.50 + 0.295)\ (120)\ H = 38.16 \text{ (say, 38 H)}$$

(values of K_o, K_a and γ from Section 4.1).

d) Design Lateral Pressure, Semi-Rigid Wall Systems, Soil Not Dewatered (Figure 16-2)

Values of Pd and P'd have been computed for this study to be the following:

$$Pd = (0.4)\ (K_o + K_a)\ (\gamma)\ (H) = 38\ H \text{ (in like manner)}$$

$$P'd = (0.4)\ (K_o + K_a)\ (10\gamma) + (0.4)\ (K_o + K_a)\ (H-10)\ (\gamma')$$

$$P'd = (0.4)\ (0.795)\ (10)\ (120) + (0.4)\ (0.795)\ (H-10)\ (62)$$
$$P'd = 19.7\ H + 382\ \text{(say, } 20\ H + 380\text{) psf.}$$

Again the value of P'd above would be applicable to this specific case (water table 10 feet below grade).

e) Design Lateral Pressure, Traffic and Construction Equipment Surcharge (Figure 16-2)

The lateral pressure diagram for traffic and equipment surcharge is an approximattion of the results which would be obtained by theory of elasticity for a 600 psf surcharge 12 feet wide (strip load) immediately behind the ground wall. This particular diagram has been specified for traffic and construction equipment surcharge on many cut-and-cover projects and has therefore been adopted here.

f) Design Lateral Pressure Due to Surcharge, Building Foundations Not Underpinned (Figure 16-2)

On cut-and-cover projects where the soil is reasonably competent, it is usual practice to develop empirical relationships for computations of lateral pressure due to building surcharge -- as was noted above in Section 4.3. The empirical formula on Figure 16.2 incorporates the following values:

q_f = Building foundation load, which is taken as the sum of all dead and live loads on the roof, floors and basement(s), using appropriate live load reduction. The load is considered as uniformly distributed over the building plan area and acting on the elevation of the building spread footing or mat foundation.

D_f = Depth of building foundation

n = Net building founation load, which is equal to to q_f minus the weight of soil replaced by the building. It is seen therefore that $n = q_f - \gamma D_f$

W = 40% of n when the building line coincides with the ground wall system. (a = 0)

W = 0 if the horizontal distance between the building line and the ground wall equals 1.5 times the depth of the cut-and-cover structure invert below the building foundations ($H_1 = 1.5$ a)

a = Minimum distance between ground wall and building foundation. Therefore $W = 0.4n\ (1 - \frac{a}{1.5H_1})$

Care must be taken to recognize building surcharge conditions where empirical relationships are not applicable. For the assumed soil type, and for this particular urban site, the above empirical equation is representative and should yield satisfactory results.

g) <u>Building Foundations Underpinned (Figure 16-2)</u>

This criteria, shown on Figure 16-2, was discussed in detail above (Section 4.2).

h) <u>Figure 16-3</u>

Metric values, corresponding to the English values shown on Figure 16-2, are shown on Figure 16-3.

i) <u>Design Passive Resistance, Retained Soil Dewatered, (Figure 16-4)</u>

The embedment below subgrade of a ground wall system is dependent upon both needed passive resistance and needed vertical bearing capacity. Where construction decks are required as part of the braced cofferdam, vertical bearing capacity will often control the design (i.e. the depth "D" below subgrade). However, in deep excavations, internally braced, required passive resistance below subgrade may control the penetration below subgrade. The passive resistance diagram shown on Figure 16-4 is somewhat conservative. Nevertheless, it is representative of the criteria often used in cut-and-cover construction and is therefore incorporated into this study. Values of active and passive pressure gradients (p_a and p_p) are based on Rankine values:

$$p_a = Ka\gamma = (0.295)\ (120) = 35.4 \text{ psf/ft. (say 35 psf/ft.)}$$

$$p_p = \frac{Kp}{F.S.}\gamma = \frac{(6.25)\ (120)}{1.5} = 500 \text{ psf/ft.}$$

where K_a K_p and γ are values discussed in Section 4.1; and the factor of safety, "F.S." is taken as 1.5 (which would be appropriate for this urban site).

"Efficiency factors" larger than 1.0 are applicable to soldier piles. For a driven soldier pile the passive resistance p_p may be considered as acting on width three times the width of the soldier pile (efficiency factor = 3.0). For a soldier pile encased in concrete the passive resistance p_p may be considered as acting on a width 2.25 times the diameter of the concrete surrounding the piles (efficiency factor = 2.25). These are relatively common values for this soil type and passive resistance type diagram.

j) <u>Design Passive Resistance, Retained Soil Not Dewatered (Figure 16-4)</u>

Values of pa, p'a, p_p and p'_p shown on Figure 16-4 are Rankine values:

$pa = Ka\gamma = 35$ psf/ft. (as above)

$p'a = Ka\gamma' = (0.295)\ (62) = 18.3$ psf/ft. (say, 18 psf/ft.)

$p_p = Kp\gamma = 500$ psf/ft. (as above)

$p'_p = \frac{Kp\,\gamma'}{F.S.} = \frac{(6.25)}{1.5}\ (62) = 258$ psf/ft. (say 250 psf/ft.)

Also: $pw = \gamma_w = 63$ psf/ft.

This diagram is applicable to diaphragm walls in this study. (The diagram would also be applicable to sheet pile walls.) The depth of wall embedment below subgrade will, however, be controlled by the need to prevent or control piping in this soil type. (See Section 4.4.3 for further discussion.)

k) <u>Design Vertical Bearing Capacity (Figure 16-4)</u>

Substantial vertical bearing capacity must be incorporated into the design of ground wall systems to account for the following types of vertical loads (as applicable).

Live and dead construction deck loads

Weight of ground wall

Weight of internal bracing

Vertical or downward component of tieback loads

Usual practice in cut-and-cover construction is to develop empirical vertical bearing capacity formulae which will give palatable results on the side of safety. The formulae suggested on Figure 16-4 are representative of this approach for this study soil type. In special cases of high concentrated vertical loads on a portion of the ground wall system this type of empirical formulae may yield unreasonable results. In these cases a more comprehensive design approach is warranted..

l) Deck Structure Loads

The minimum practical load criteria for construction decking is ordinarily considered to be A.A.S.H.O. HS-20-44 loading. Construction equipment (such as crawler cranes, truck cranes and modern transit mix trucks) often impart heavier loads. In these cases the deck structure should be designed for both A.A.S.H.O. HS-20-44 loading and applicable construction equipment loading. For this study, the following loadings were considered representative for the typical urban site and were used in the design of deck structures:

For a four-lane highway tunnel or rapid transit station construction, Figures 2 and 4 (deck width 65 ft. to 72 ft.):

(1) Any reasonable combination of three 11 ft. A.A.S.H.O. HS-20-44 traffic lanes (public traffic) plus a working 50-ton crawler crane or truck crane (contractor's working area on deck) --- or,

(2) Four 11 ft. A.A.S.H.O. HS-20-44 traffic lanes and two parking lanes (public traffic and/or equivalent contractor's work deck traffic).

m) Allowable Stresses for Temporary Structures

Allowable stresses for most of the components of cut-and-cover temporary structures are ordinarily taken at higher values than basic allowable stresses for permanent structures. (Basic allowable stresses are defined on Figure 16-4). There is general agreement in the engineering community about what these allowable stresses should be. The following allowable stresses are representative of current practice and were therefore used here:

m.1) Soldier Piles and Wales

Allowable stress shall not exceed 120% of basic allowable stress.

m.2) Sheet Piles

Allowable bending stress F_b = 0.80 Fy where Fy = minimum yield stress.

m.3) Diaphragm Walls (S.P.T.C. walls, Precast Panel Walls, etc.

Allowable stresses shall not exceed 120% of basic allowable stresses (applies to temporary loads only).

m.4) Struts

The slenderness ratio of struts shall not exceed 120 and the maximum axial stress to which the struts may be subjected shall not exceed 14,000 psi.

m.5) Timber Lagging

Allowable stresses shall not exceed 120% of basic allowable stresses.

m.6) Stress Bars or Strands for Tiebacks

Allowable tensile stress shall not exceed 0.60 f's, where f's is the minimum ultimate tensile strength (allow 0.80 f's for test load).

m.7) Deck Structure Framing Carrying Public Traffic

Allowable stresses shall be as specified in the latest edition of "Standard Specifications for Highway Bridges" as adopted by the American Association of State Highway Officials (A.A.S.H.O.).

m.8) Deck Structure Framing Carrying Construction Loads Only

Allowable stresses shall not exceed 100% of basic allowable stresses.

4.4.2 Design of Construction Decking - The loading criteria used in this study for the design of deck structures should be representative of most cases where decking is required in an urban area. The concept of temporary support piles for deck beams near midspan for the rapid transit station and highway tunnel studies (Figures 5 and 7) is compatible with assumed traffic maintenance requirements for these studies. At many urban sites, however, it is possible to divert traffic temporarily so that deck beams spanning the entire excavation can be used. The cost of construction decking ordinarily does not change markedly when this latter construction method is used; but the method does offer significant savings in excavation costs and in the cost of the construction of permanent structures (in this case the highway tunnel or rapid transit station) -- resulting in a total cost saving. The particular deck design concept selected for this research is often required as a practical matter and was therefore considered appropriate here.

4.4.3 Design of Ground Walls -

a) Soldier Piles and Lagging

Figure 17-1 shows the material quantities developed here for soldier pile and lagging type ground walls. This data is based on the design criteria detailed in Section 4.4.1. Soldier pile sizes conform to the internal bracing and/or tieback spacings also developed for this study. It should be noted that the soldier pile and lagging system is the same for the "wet" and "dry" conditions because for the "wet" condition the water table would be lowered (i.e. the site would be dewatered outside the soldier pile and lagging wall). See also Figure 5.

b) Cast-in-place S.P.T.C. Walls

Cast-in-place walls have been designed here for two distinct conditions:

"Wet" condition: permanent water table assumed 10 ft. below the ground or grade. Water table will not, or cannot, be lowered.

"Dry" condition: permanent water table assumed 15 ft. or more below subgrade (i.e. 15 ft. below the base of the cut-and-cover structure).

The cross-section on Figure 6 shows typical construction for both the "wet" and "dry" conditions. Materials quantities for these support systems are tabulated on Figures 17-2, wet condition, and 17-3, dry condition. Soldier pile sizes conform to the design criteria detailed in Section 4.4.1 and to the internal bracing and/or tieback spacings developed for this study. Tremie concrete quantities for S.P.T.C. walls will be based on the dimensions shown on Figure 6. For the wet condition the required depth of embedment of S.P.T.C. walls below subgrade is controlled by the need to reduce the hydraulic gradient in the soil below the bottom of the excavation and thus prevent piping. For the deep structures this is a far more serious consideration than passive resistance or vertical bearing capacity requirements. These depth requirements are tabulated on Figure 17.2. They are based on experimental results summarized on charts in NAVFAC, DM-7 (reference 8). The results for 50 ft. depths appear severe and for 70 ft. depths probably unpalatable. Nevertheless these are the depths of embedment below subgrade which would be required at this urban site given this study soil type. (The depths "D" tabulated on Figure 17-2 offer an approximate factor of safety of 1.5 against piping.) In analyzing the results of this study these depths of embedment should be considered maximum depths -- which for this study is appropriate. At most cut-and-cover sites the depths "D" would be less for the following reasons:

b.1) A uniformly non-cohesive, pervious soil type (the study soil type here) extending to a depth of 80 ft. or more (for a 50 ft. excavated depth) or 130 ft. or more (for a 70 ft. excavated depth) would be highly unlikely. The soil would more likely have some cohesive properties or would contain zones of soil having cohesive properties. In either case smaller values of "D" could be shown to be safe. Another likely possibility would be that the S.P.T.C. would encounter an impervious clay layer below subgrade, which would act as a cut-off and would thus reduce the requirement for embedment below subgrade.

b.2) If the soil were in fact similar to the study soil type, dewatering outside the S.P.T.C. wall would most probably be permitted, since the soil would not settle or compress appreciably due to the dewatering operation. In this case the depth "D" would be as shown on Figure 17-3 for complete dewatering and somewhere in between for a partially lowered water table condition.

For this study, however, the most useful results should be obtained by making available the data based on the "dry" and "wet" conditions as defined herein, representing maximum and minimum conditions.

c) Precast Concrete Ground Walls

The cross-section on Figure 7 shows precast concrete ground walls for both the "wet" and "dry" conditions. The design of these walls is based on the same general criteria and considerations as were used to determine the materials quantities for the cast-in-place S.P.T.C. wall (i.e. design criteria, internal bracing and/or tieback spacings, water table and resulting piping problems for the wet condition, etc.) Materials quantities for these walls are tabulated on Figures 17-4 and 17-5.

In order to estimate the cost of this alternative ground wall system consideration will be given to develop special shapes and sections, or other possible procedures (refer also to Section 2.7.9) to reduce the weight of the individual panels to be handled in constructing the wall.

For the "wet" condition deep excavations, this system has a particular advantage. Precast panels need to extend only as deep as is essential for required passive resistance below subgrade. Below that elevation the wall will consist of an impervious cement and bentonite slow-setting grout slurry which needs only to cut off water and provide required vertical bearing capacity.

4.4.4 Design of Bracing for Ground Walls - "Bracing" for the purposes of this study refers to either internal bracing or tiebacks.

a) Internal Bracing

Internal bracing may be defined here as structural steel framing placed inside the excavation between ground walls and designed to resist lateral pressures imparted to the wall system. Internal bracing for this study consists of multiple levels of wales and struts of a basic design which has long been in use. Criteria for analysis and design of internal bracing are included in Section 4.4.1.

The general arrangement of internal bracing is shown on Figures 5, 6, and 7 for the following three cases:

Figure 5: Four lane highway tunnel: depth of excavation = 70 ft.

Figure 6: Rapid Transit Line Structure: depth of excavation = 70 ft.

Figure 7: Rapid Transit Station: depth of excavation = 70 ft.

It is seen that for the 70 ft. excavation depth each of the cut-and-cover excavations requires four levels of bracing in addition to the decking at street level.

These excavations will be similarly braced for the 50 ft. and 30 ft. excavation depths. Materials quantities for internal bracing at each required level are shown on Figures 17-1 through 17-5. These quantities were developed from the applicable criteria included in Section 4.4.1.

b) Tiebacks (or Earth Anchors)

The cross sections of Figure 8 shows typical tieback bracing alternates for both the "wet" and "dry" conditions at the three study depths of 30 ft., 50 ft., and 70 ft. The general arrangement for the rapid transit line structure has been depicted, but the rapid transit station and the highway tunnel would yield a similar configuration, as this method of bracing is independent of structure width. It is possible that the number and locations of bracing levels may vary slightly due to considerations of horizontal joint locations in the permanent structure.

Tiebacks achieve their anchorage beyond the design critical failure plane -- Line A. The anchorage length must at least extend to Line B in order to obtain an approximate factor of safety of 1.5 for stability of the entire bulkhead system. The criteria for determining extent of Lines A and B (in terms of depth of excavation, "H") are indicated for the 70 ft. depth. The same criteria apply for the 30 ft. and 50 ft. depths. Tiebacks are generally installed at an angle of from 5° to 25° to the horizontal. For this study an angle of 15° was used. Deformed stress bars (i.e. Dywidag bars or equal), are used as tendons in this study. The tieback anchorage for the "dry" condition has been evaluated by using the "straight shaft" method. For the "wet" condition the tieback anchorage generally is obtained by means of a pressure grouted "bulb". The criteria for determining the anchor length, when using pressure grouting methods, are too complex for the scope of this report. However, the appropriate lengths have been evaluated and will be utilized in future cost studies. It should be pointed out that the use of tiebacks for the 70 ft. "wet" condition is perhaps beyond the limits of present practice, especially in an urban street environment. The adhesion (skin friction) between the soil and the anchor grout for the "straight shaft" anchorage was evaluated from the following relationship:

$$\text{Adhesion (A)} = \frac{Nq}{F.S.}$$

$$\text{Where: } N = \frac{f\ (\tan \phi)}{Ka}$$

and $q = \gamma D_a$ (use γ' below ground water table)

D_a = Depth from ground surface to mid point of tieback anchorage

F.S. = Factor of safety; 2.0 for tieback anchorage

f = Factor; dependent on tieback angle and factor safety; use 0.267 for 15° angle and a F.S. = 2.0.

Criteria used for analyzing lateral load on tieback bulkhead systems are included in Section 4.4.1.

The 30 ft. cut requires one level of tieback bracing for both the "dry" and "wet" conditions in combination with the deck level bracing. In the same manner, the 50 ft. deep excavation requires two levels of tieback bracing. For the 70 ft. cut, in addition to the deck level bracing, 5 levels of tiebacks are required for the "dry" condition, and 6 levels for the "wet" condition. The materials quantities shown on Figures 17-1 through 17-5 were developed from criteria contained in Section 4.4.1 and in this Section 4.4.4.

4.4.5 <u>Design Guides (Summary)</u> - The following list of texts and/or design manuals are used extensively in the engineering community for the design of temporary structures associated with cut-and-cover construction. Each of these publications has been consulted in preparing this Section 4.

<u>References, Design of Temporary Structures for Cut-and-Cover Construction</u> (Reference numbers below refer to the complete bibliographic listings starting on page 198.)

Terzaghi, 1943 (reference 11)
Terzaghi and Peck, 1948 (reference 12)
Terzaghi and Peck, 1967 (reference 13)
Teng, 1962 (reference 14)
Tschebotarioff, 1973 (reference 15)
Leonards, 1962 (reference 16)
Andersen, 1956 (reference 17)
NAVFAC, DM-7, 1971 (reference 18)
SMFD/ASCE, 1970 (reference 19)
Steel Sheet Piling Design Manual, 1974 (reference 20)
Prestressed Concrete Institute, 1974 (reference 21)
ASCE/SEONIC, 1970 (references 3, 4, 32, 33, 34, 38, 39)
AISC, 1973 (reference 22)
AASHO, 1973 (reference 23)
Uniform Building Code, (reference 24)
Timber Construction Manual, 1966 (reference 25)

SECTION 5

5.0 ANALYZING THE EFFICIENCY OF CUT-AND-COVER CONSTRUCTION

Having established the typical cut-and-cover situations, basic activities, and design of structures, ground support and decking to be used for this study, the next step in procedure consists of providing a logical format for combining all of these factors. Through this process it is possible to pause for reflection and review the results before proceeding with an economic analysis of cut-and-cover construction. It is possible at this stage to develop analytical expressions for some of these relationships, thereby simplifying the remaining analysis procedures.

The interrelationship and interdependence of various activities have been discussed in Section 2. The way to fully evaluate the impact of each important activity and site condition is through cost evaluation of the total construction process. By varying each factor in turn, while maintaining the others constant, and observing the effect on total construction costs, it will be possible to plot results and quantify trends.

For instance, assume a highway tunnel is being planned, and it is desired to use a ground support system that will prove most economical for various depths and ground water conditions. Estimates can be made for each ground support being considered for several representative depths, first for a high ground water level, and then repeated for a low water table. By plotting resulting costs on a common chart, a family of curves can be produced that will not only give the answer required, but will show trends that can be projected to other situations, other depths, intermediate ground water levels, etc. The results of this type of analysis, for all important variables considered for the chosen conditions of this study, will indicate the optimum procedures, and also will show which operations and factors have the greatest and least effect on total cost and time of completion. The methods to be used for making such cost evaluations will be given in Section 5.4 and the relation of cost and project duration will be discussed in Section 5.5.

5.1 VALUE ANALYSIS

In arriving at a format for expressing analytically the various interrelationships of factors to be considered in this type of study, two pitfalls must be avoided. The first is to indiscriminately include so many variables that the process becomes unwieldy; the second is to oversimplify and so lose flexibility and applicability to a wide range of situations.

Through the discussions of activities and methods in Section 2 and the evaluation process described in Section 3, it is possible to reduce the multitude of optional sub-activities and construction methods to a more manageable number. It would be pointless to include a number of ways of performing the same task where the overall effect on time and cost is negligible. Each additional set of independent factors included is a multiplier for all others when considering computing total cost estimates for all combinations. This means that the total number of cost analyses to be considered is a cumulative result of multiplying all independent factors.

It would be equally fruitless to try to reduce a complex construction project to a simple equation. If the relationships to be expressed are to be valid for a majority of situations, there must be valid expressions for all significant variables.

5.1.1 Activity interrelationship format - Figure 18 presents a comprehensive summation of all site factors, construction factors, and construction activities necessary for a value analysis of cut-and-cover construction. Due to space limitations it is divided into three parts, but the three combined show the total number of possible combinations to be considered. The site conditions and construction options

W = Wet Soil Condition
D = Dry Soil Condition

FOUR LANE HIGHWAY TUNNEL 2,000 FT. LONG

		SOLDIER PILES AND LAGGING											
		INTERNAL BRACING						TIE BACK BRACING					
		30'		50'		70'		30'		50'		70'	
		W	D	W	D	W	D	W	D	W	D	W	D
I A	CONTROL TRAFFIC	00	00	00	00	00	00	00	00	00	00	00	00
I B	UTILITY WORK	AO	AO	AO	AO	AO	AO	AO	AO	AO	AO	AO	AO
I C	PROTECT ADJ. STRUCT.	AN	AN	AP	AP	AQ	AQ	AN	AN	AP	AP	AQ	AQ
I D	CONTROL GRND. WATER	AT	–	AV	–	AY	–	AT	–	AV	–	AY	–
I E	DECKING	00	00	00	00	00	00	00	00	00	00	00	00
I F	GRND. WALL SUPPORT	AN	AN	AP	AP	AQ	AQ	AN	AN	AP	AP	AQ	AQ
I G	BRACING	EN	EN	EP	EP	EQ	EQ	HN	HN	FP	FP	FQ	FQ
I H	EXCAVATION	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
I I	CONST. PERM. STRUCT.	AR	AS	AR	AS	AR	AS	AR	AS	AR	AS	AR	AS
I J	BACKFILL	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
I K	RESTORATION	00	00	00	00	00	00	00	00	00	00	00	00
I N	OVERHEAD (FIXED COST)	00	00	00	00	00	00	00	00	00	00	00	00
I O	OVERHEAD (TIME RELAT.)	GN	GN	GP	GP	GQ	GQ	HN	HN	HP	HP	HQ	HQ
I P	PLANT (FIXED COSTS)	00	00	00	00	00	00	00	00	00	00	00	00
I Q	PLANT (TIME RELATED)	GN	GN	GP	GP	GQ	GQ	HN	HN	HP	HP	HQ	HQ
	ESTIMATE DESIGNATION	I GT	I GU	I GV	I GW	I GY	I GZ	I HT	I HU	I HV	I HW	I HY	I HZ

		CAST-IN-PLACE CONCRETE WALL											
		INTERNAL BRACING						TIE BACK BRACING					
		30'		50'		70'		30'		50'		70'	
		W	D	W	D	W	D	W	D	W	D	W	D
I A	CONTROL TRAFFIC	00	00	00	00	00	00	00	00	00	00	00	00
I B	UTILITY WORK	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO
I C	PROTECT ADJ. STRUCT.	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO
I D	CONTROL GRND. WATER	DT	–	DV	–	DY	–	DT	–	DV	–	DY	–
I E	DECKING	00	00	00	00	00	00	00	00	00	00	00	00
I F	GRND. WALL SUPPORT	BT	BU	BV	BW	BY	BZ	BT	BU	BV	BW	BY	BZ
I G	BRACING	EN	EN	EP	EP	EQ	EQ	KT	KU	FV	FW	FY	FZ
I H	EXCAVATION	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
I I	CONST. PERM. STRUCT.	BR	BS	BR	BS	BR	BS	BR	BS	BR	BS	BR	BS
I J	BACKFILL	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
I K	RESTORATION	00	00	00	00	00	00	00	00	00	00	00	00
I N	OVERHEAD (FIXED COST)	00	00	00	00	00	00	00	00	00	00	00	00
I O	OVERHEAD (TIME RELAT.)	JN	JN	JP	JP	JQ	JQ	KN	KN	KP	KP	KQ	KQ
I P	PLANT (FIXED COSTS)	00	00	00	00	00	00	00	00	00	00	00	00
I Q	PLANT (TIME RELATED)	JN	JN	JP	JP	JQ	JQ	KN	KN	KP	KP	KQ	KQ
	ESTIMATE DESIGNATION	I JT	I JU	I JV	I JW	I JY	I JZ	I KT	I KU	I KV	I KW	I KY	I KZ

		PRECAST CONCRETE WALL											
		INTERNAL BRACING						TIE BACK BRACING					
		30'		50'		70'		30'		50'		70'	
		W	D	W	D	W	D	W	D	W	D	W	D
I A	CONTROL TRAFFIC	00	00	00	00	00	00	00	00	00	00	00	00
I B	UTILITY WORK	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO
I C	PROTECT ADJ. STRUCT.	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO
I D	CONTROL GRND. WATER	DT	–	DV	–	DY	–	DT	–	DV	–	DY	–
I E	DECKING	00	00	00	00	00	00	00	00	00	00	00	00
I F	GRND. WALL SUPPORT	CT	CU	CV	CW	CY	CZ	CT	CU	CV	CW	CY	CZ
I G	BRACING	EN	EN	EP	EP	EQ	EQ	MT	MU	FV	FW	FY	FZ
I H	EXCAVATION	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
I I	CONST. PERM. STRUCT.	CR	CS	CR	CS	CR	CS	CR	CS	CR	CS	CR	CS
I J	BACKFILL	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
I K	RESTORATION	00	00	00	00	00	00	00	00	00	00	00	00
I N	OVERHEAD (FIXED COST)	00	00	00	00	00	00	00	00	00	00	00	00
I O	OVERHEAD (TIME RELAT.)	LN	LN	LP	LP	LQ	LQ	MN	MN	MP	MP	MQ	MQ
I P	PLANT (FIXED COSTS)	00	00	00	00	00	00	00	00	00	00	00	00
I Q	PLANT (TIME RELATED)	LN	LN	LP	LP	LQ	LQ	MN	MN	MP	MP	MQ	MQ
	ESTIMATE DESIGNATION	I LT	I LU	I LV	I LW	I LY	I LZ	I MT	I MU	I MV	I MW	I MY	I MZ

Activity Variation Table

Figure 18 - Sheet 1

RAPID TRANSIT STATION 700 FT. LONG

W = Wet Soil Condition
D = Dry Soil Condition

SOLDIER PILES AND LAGGING

		INTERNAL BRACING						TIE BACK BRACING					
		30'		50'		70'		30'		50'		70'	
		W	D	W	D	W	D	W	D	W	D	W	D
2 A	CONTROL TRAFFIC	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 B	UTILITY WORK	—	—	AO	AO	AO	AO	—	—	AO	AO	AO	AO
2 C	PROTECT ADJ. STRUCT.	—	—	AP	AP	AQ	AQ	—	—	AP	AP	AQ	AQ
2 D	CONTROL GRND. WATER	—	—	AV	—	AY	—	—	—	AV	—	AY	—
2 E	DECKING	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 F	GRND. WALL SUPPORT	—	—	AP	AP	AQ	AQ	—	—	AP	AP	AQ	AQ
2 G	BRACING	—	—	EP	EP	EQ	EQ	—	—	FP	FP	FQ	FQ
2 H	EXCAVATION	—	—	EP	EP	EQ	EQ	—	—	FP	FP	FQ	FQ
2 I	CONST. PERM. STRUCT.	—	—	AR	AS	AR	AS	—	—	AR	AS	AR	AS
2 J	BACKFILL	—	—	EP	EP	EQ	EQ	—	—	FP	FP	FQ	FQ
2 K	RESTORATION	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 N	OVERHEAD (FIXED COST)	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 O	OVERHEAD (TIME RELAT.)	—	—	GP	GP	GQ	GQ	—	—	HP	HP	HQ	HQ
2 P	PLANT (FIXED COSTS)	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 Q	PLANT (TIME RELATED)	—	—	GP	GP	GQ	GQ	—	—	HP	HP	HQ	HQ
	ESTIMATE DESIGNATION	—	—	2GV	2GW	2GY	2GZ	—	—	2HV	2HW	2HY	2HZ

CAST-IN-PLACE CONCRETE WALL

		INTERNAL BRACING						TIE BACK BRACING					
		30'		50'		70'		30'		50'		70'	
		W	D	W	D	W	D	W	D	W	D	W	D
2 A	CONTROL TRAFFIC	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 B	UTILITY WORK	—	—	DO	DO	DO	DO	—	—	DO	DO	DO	DO
2 C	PROTECT ADJ. STRUCT.	—	—	DO	DO	DO	DO	—	—	DO	DO	DO	DO
2 D	CONTROL GRND. WATER	—	—	DV	—	DY	—	—	—	DV	—	DY	—
2 E	DECKING	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 F	GRND. WALL SUPPORT	—	—	BV	BW	BY	BZ	—	—	BV	BW	BY	BZ
2 G	BRACING	—	—	EP	EP	EQ	EQ	—	—	FV	FW	FY	FZ
2 H	EXCAVATION	—	—	EP	EP	EQ	EQ	—	—	FP	FP	FQ	FQ
2 I	CONST. PERM. STRUCT.	—	—	BR	BS	BR	BS	—	—	BR	BS	BR	BS
2 J	BACKFILL	—	—	EP	EP	EQ	EQ	—	—	FP	FP	FQ	FQ
2 K	RESTORATION	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 N	OVERHEAD (FIXED COST)	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 O	OVERHEAD (TIME RELAT.)	—	—	JP	JP	JQ	JQ	—	—	KP	KP	KQ	KQ
2 P	PLANT (FIXED COSTS)	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 Q	PLANT (TIME RELATED)	—	—	JP	JP	JQ	JQ	—	—	KP	KP	KQ	KQ
	ESTIMATE DESIGNATION	—	—	2JV	2JW	2JY	2JZ	—	—	2KV	2KW	2KY	2KZ

PRECAST CONCRETE WALL

		INTERNAL BRACING						TIE BACK BRACING					
		30'		50'		70'		30'		50'		70'	
		W	D	W	D	W	D	W	D	W	D	W	D
2 A	CONTROL TRAFFIC	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 B	UTILITY WORK	—	—	DO	DO	DO	DO	—	—	DO	DO	DO	DO
2 C	PROTECT ADJ. STRUCT.	—	—	DO	DO	DO	DO	—	—	DO	DO	DO	DO
2 D	CONTROL GRND. WATER	—	—	DV	—	DY	—	—	—	DV	—	DY	—
2 E	DECKING	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 F	GRND. WALL SUPPORT	—	—	CV	CW	CY	CZ	—	—	CV	CW	CY	CZ
2 G	BRACING	—	—	EP	EP	EQ	EQ	—	—	FV	FW	FY	FZ
2 H	EXCAVATION	—	—	EP	EP	EQ	EQ	—	—	FP	FP	FQ	FQ
2 I	CONST. PERM. STRUCT.	—	—	CR	CS	CR	CS	—	—	CR	CS	CR	CS
2 J	BACKFILL	—	—	EP	EP	EQ	EQ	—	—	FP	FP	FQ	FQ
2 K	RESTORATION	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 N	OVERHEAD (FIXED COST)	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 O	OVERHEAD (TIME RELAT.)	—	—	LP	LP	LQ	LQ	—	—	MP	MP	MQ	MQ
2 P	PLANT (FIXED COSTS)	—	—	OO	OO	OO	OO	—	—	OO	OO	OO	OO
2 Q	PLANT (TIME RELATED)	—	—	LP	LP	LQ	LQ	—	—	MP	MP	MQ	MQ
	ESTIMATE DESIGNATION	—	—	2LV	2LW	2LY	2LZ	—	—	2MV	2MW	2MY	2MZ

Activity Variation Table

Figure 18 - Sheet 2

TWIN BOX
RAPID TRANSIT TUNNEL
2,000 FT. LONG

W = Wet Soil Condition
D = Dry Soil Condition

	SOLDIER PILES AND LAGGING											
	INTERNAL BRACING						TIE BACK BRACING					
	30'		50'		70'		30'		50'		70'	
	W	D	W	D	W	D	W	D	W	D	W	D
3 A CONTROL TRAFFIC	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 B UTILITY WORK	AO	AO	AO	AO	AO	AO	AO	AO	AO	AO	AO	AO
3 C PROTECT ADJ. STRUCT.	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 D CONTROL GRND. WATER	AT	–	AV	–	AY	–	AT	–	AV	–	AY	–
3 E DECKING	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 F GRND. WALL SUPPORT	AN	AN	AP	AP	AQ	AQ	AN	AN	AP	AP	AQ	AQ
3 G BRACING	EN	EN	EP	EP	EQ	EQ	HN	HN	FP	FP	FQ	FQ
3 H EXCAVATION	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
3 I CONST. PERM. STRUCT.	AR	AS	AR	AS	AR	AS	AR	AS	AR	AS	AR	AS
3 J BACKFILL	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
3 K RESTORATION	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 N OVERHEAD (FIXED COST)	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 O OVERHEAD (TIME RELAT.)	GN	GN	GP	GP	GQ	GQ	HN	HN	HP	HP	HQ	HQ
3 P PLANT (FIXED COSTS)	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 Q PLANT (TIME RELATED)	GN	GN	GP	GP	GQ	GQ	HN	HN	HP	HP	HQ	HQ
ESTIMATE DESIGNATION	3GT	3GU	3GV	3GW	3GY	3GZ	3HT	3HU	3HV	3HW	3HY	3HZ

	CAST-IN-PLACE CONCRETE WALL											
	INTERNAL BRACING						TIE BACK BRACING					
	30'		50'		70'		30'		50'		70'	
	W	D	W	D	W	D	W	D	W	D	W	D
3 A CONTROL TRAFFIC	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 B UTILITY WORK	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO
3 C PROTECT ADJ. STRUCT.	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 D CONTROL GRND. WATER	DT	–	DV	–	DY	–	DT	–	DV	–	DY	–
3 E DECKING	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 F GRND. WALL SUPPORT	BT	BU	BV	BW	BY	BZ	BT	BU	BV	BW	BY	BZ
3 G BRACING	EN	EN	EP	EP	EQ	EQ	KT	KU	FV	FW	FY	FZ
3 H EXCAVATION	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
3 I CONST. PERM. STRUCT.	BR	BS	BR	BS	BR	BS	BR	BS	BR	BS	BR	BS
3 J BACKFILL	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
3 K RESTORATION	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 N OVERHEAD (FIXED COST)	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 O OVERHEAD (TIME RELAT.)	JN	JN	JP	JP	JQ	JQ	KN	KN	KP	KP	KQ	KQ
3 P PLANT (FIXED COSTS)	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 Q PLANT (TIME RELATED)	JN	JN	JP	JP	JQ	JQ	KN	KN	KP	KP	KQ	KQ
ESTIMATE DESIGNATION	3JT	3JU	3JV	3JW	3JY	3JZ	3KT	3KU	3KV	3KW	3KY	3KZ

	PRECAST CONCRETE WALL											
	INTERNAL BRACING						TIE BACK BRACING					
	30'		50'		70'		30'		50'		70'	
	W	D	W	D	W	D	W	D	W	D	W	D
3 A CONTROL TRAFFIC	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 B UTILITY WORK	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO	DO
3 C PROTECT ADJ. STRUCT.	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 D CONTROL GRND. WATER	DT	–	DV	–	DY	–	DT	–	DV	–	DY	–
3 E DECKING	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 F GRND. WALL SUPPORT	CT	CU	CV	CW	CY	CZ	CT	CU	CV	CW	CY	CZ
3 G BRACING	EN	EN	EP	EP	EQ	EQ	MT	MU	FV	FW	FY	FZ
3 H EXCAVATION	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
3 I CONST. PERM. STRUCT.	CR	CS	CR	CS	CR	CS	CR	CS	CR	CS	CR	CS
3 J BACKFILL	EN	EN	EP	EP	EQ	EQ	FN	FN	FP	FP	FQ	FQ
3 K RESTORATION	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 N OVERHEAD (FIXED COST)	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 O OVERHEAD (TIME RELAT.)	LN	LN	LP	LP	LQ	LQ	MN	MN	MP	MP	MQ	MQ
3 P PLANT (FIXED COSTS)	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO	OO
3 Q PLANT (TIME RELATED)	LN	LN	LP	LP	LQ	LQ	MN	MN	MP	MP	MQ	MQ
ESTIMATE DESIGNATION	3LT	3LU	3LV	3LW	3LY	3LZ	3MT	3MU	3MV	3MW	3MY	3MZ

Activity Variation Table

Figure 18 - Sheet 3

ESTIMATE ACTIVITY VARIATION CODES

FIRST CHARACTER - TYPE OF STRUCTURE

1. FOUR LANE HIGHWAY TUNNEL
2. RAPID TRANSIT STATION
3. TWIN BOX RAPID TRANSIT TUNNEL

SECOND CHARACTER - MAJOR ESTIMATE ACTIVITY

A. CONTROL TRAFFIC
B. UTILITY WORK
C. PROTECT ADJACENT STRUCTURES
D. CONTROL GROUND WATER
E. DECKING
F. GROUND WALL SUPPORT
G. BRACING
H. EXCAVATION
I. CONSTRUCT PERMANENT STRUCTURE
J. BACKFILL
K. RESTORATION

N. OVERHEAD (FIXED COSTS)
O. OVERHEAD (TIME RELATED COSTS)
P. PLANT (FIXED COSTS)
Q. PLANT (TIME RELATED COSTS)

THIRD CHARACTER - TYPE OF WALL AND/OR BRACING

A. SOLDIER PILE AND LAGGING WALL ONLY
B. S.P.T.C. WALL ONLY
C. PRECAST WALL ONLY

D. DIAPHRAGM WALL - IE, S.P.T.C. OR PRECAST WALL

E. INTERNAL BRACING ONLY
F. TIEBACK BRACING ONLY

G. SOLDIER PILE AND LAGGING WALL WITH INTERNAL BRACING
H. SOLDIER PILE AND LAGGING WALL WITH TIEBACK BRACING

J. S.P.T.C. WALL WITH INTERNAL BRACING
K. S.P.T.C. WALL WITH TIEBACK BRACING

L. PRECAST WALL WITH INTERNAL BRACING
M. PRECAST WALL WITH TIEBACK BRACING

FOURTH CHARACTER - DEPTH OF EXCAVATION AND/OR WET OR DRY SOIL CONDITION

N. 30 FT DEPTH ONLY
P. 50 FT DEPTH ONLY
Q. 70 FT DEPTH ONLY

R. WET CONDITION ONLY
S. DRY CONDITION ONLY

T. 30 FT DEPTH WITH WET SOIL CONDITION
U. 30 FT DEPTH WITH DRY SOIL CONDITION

V. 50 FT DEPTH WITH WET SOIL CONDITION
W. 50 FT DEPTH WITH DRY SOIL CONDITION

Y. 70 FT DEPTH WITH WET SOIL CONDITION
Z. 70 FT DEPTH WITH DRY SOIL CONDITION

NOTE: IN THE THIRD AND FOURTH CHARACTER WHERE NO VARIABLE CONDITION APPLIES A "O" (ZERO) WILL BE USED.

Figure 19

shown across the top represent 108 possible combinations. Since the rapid transit station cannot be built at the 30 foot depth, because it would project above ground level, the number of possible combinations is reduced to 96. They have been divided to show all combinations for each type of structure on a separate page. The horizontal lines represent the major activity alternates for each condition. Using the same letter designation assigned to these activities in Section 2, A through K represent the major activities of direct cost items. Overhead and contractor's plant costs utilize letters N through Q. A more complete discussion of these additional costs and how they are arrived at will be given in Section 5.1.2.

The site factors and construction alternates shown are described in Section 1. They were all suggested in the original proposal for this study with the exception of the two bracing options, internal bracing and tiebacks. The bracing of a ground support system is not necessarily fixed by, nor dependent on, the ground wall support system chosen. Moreover, the choice of bracing can affect excavation, backfill, construction of the permanent structure, and protection of adjacent property. It was decided therefore, that this should be included as a significant variable.

5.1.2 <u>Analytical expressions of variables</u> - the coding system used in the table of Figure 18 serves several purposes. Each activity, for each of the 96 combinations, is given a four character code which designates the activity and those site and construction options which cause significant variations of that activity. Figure 19 gives the definition of these characters. The first character is a number that represents the type of structure. In addition to the activity variable, which is the second character, there are four sets of site and construction variables. In order to express these five variables in three characters, letters are used for the third and fourth characters to increase the number of options possible. The third character is a letter that represents the ground support and/or bracing options; the fourth character is a letter that represents the depth of excavation and/or wet or dry soil condition (high or low water table). The character used indicates those conditions which require a variation in the cost estimate of the activity. If the activity is not dependent on the condition (i.e. traffic control is not dependent on whether the soil is wet or dry) a zero is substituted for the character. Thus the code is an expression of a particular estimate variation for the activity in a particular estimate combination. The total construction procedure can be expressed as the sum of the activity option codes for that estimate. In addition the total project can be uniquely using the first character number and the third and fourth character letters representing the appropriate combination of variables.

For example: for the situation where a highway tunnel is to be constructed using soldier piles and lagging, with internal bracing, and excavation is to be carried to 50 feet depth in dry soil, the excavation activity can be described as: 1HEP; the project, 1GW. The total project can be described as: $\sum$ A through K + $\sum$ N through Q, or project 1GW = 1A00 + 1BA0 + 1CAP + 1E00 + 1FAP + 1GEP + 1HEP + 1IAS + 1JEP + 1K00 + IN00 + 1OGP + 1P00 + 1QGP. It should be noted that operation D, ground water control, was omitted as it is not required for this situation. The varying duration of time related items O and Q cannot be determined till direct costs estimates have established project duration.

The four character activity code designation conforms to the identification of a work item in the computer program to be used for estimating costs for this study. This program, which will be described in Section 5.4, is subdivided into work items and work operations. The work items are comparable to the basic activities of this study, and the operations to sub-activities. In preparing an estimate for a cut-and-cover construction project, each work item would be prepared using an appropriate number of operations. The work item is described by a four character code, similar to the code used here to describe an activity. In using this computer system to combine a number of previously determined activity options (or work items) it is possible to identify each work item by its distinct four character code name. The work item costs for an estimate can then be added to arrive at total direct cost (A-K) and total indirect costs (N-Q). This procedure will help to simplify the logistical problems of producing multiple cost estimates to optimize construction operations.

The activities A through K described in detail in Section 2, constitute the work items usually included as contract items. Indirect costs are generally considered those costs not readily identified with a particular direct cost item, such as rental for job office space, a timekeeper's salary or installation of an electrical substation. The particular charges to these accounts will vary slightly depending on the contractor's cost accounting system. Some contractors will charge only equipment operating costs to a direct cost item and consider the difference between purchase and salvage as indirect expense. Others will charge off ownership to individual pay items along with operating costs. This latter procedure will be followed in this study to simplify revisions due to variations of site and construction conditions.

Activity N represents fixed overhead costs such as job bond and personnel relocation costs. Activity O represents time related overhead costs such as supervisory payroll, office rent, and telephone. Plant costs can also be divided into fixed costs (activity P), such as shop erection costs and electrical substation installation, and time related costs (activity Q) such as electric power and plant maintenance costs.

5.1.3 Variation in material quantities - The variations in activities discussed to this point have been major in the respect that they involved changes in the scope of the work. There are other, less obvious differences that affect some activities when site conditions vary. These are mostly differences in material quantities where the scope of work remains essentially the same for labor crews and equipment.

An example of this is the variation of decking due to depth of excavation. A review of dimensions of any of the structures show that the outside wall thickness, and therefore the overall width of structure, varies with depth. Although this difference is not sufficient to warrant increasing the deck beam sizes, each beam increases in weight because it is longer. The extra width also increase the deck timbers required.

Only variations of scope of work are included in the table of Figure 18. It is possible that during the estimating process some of these variations, or some material variations, will be found to be insignificant in terms of overall job costs, and can then be eliminated from consideration. Other variations will probably follow predictable patterns, and having established these patterns can then be varied by inspection and interpolation.

5.2 AVAILABLE COST DATA

The major proportion of cut-and-cover projects in the United States are constructed for public or governmental agencies. This means that there is a public announcement made, public bidding, and award of contract to the lowest qualified bidder. Detailed unit bid prices are usually published within a few days after the bids have been opened. It is safe to estimate that in the last ten years, in the United States alone, over a hundred major cut-and-cover projects have been bid and let using similar procedures. It would seem from this that the cost of doing this type of work would be well known and documented.

Strangely enough, nothing could be further from the truth. A review of the unit prices published from such bids indicate variations where high bids are sometimes 200% or more of the low bid and unit price spreads exceed that percentage by many times. A statement made earlier said that no two heavy construction projects were identical, and apparently, very seldom do two contractors bid such a project with similar cost projections. To add to the complication the actual cost of a project is seldom the same as the amount estimated by the successful bidder. Two reasons for this are, that the most optimistic contractor is low bidder, and few projects end up as "typical" or "class-book" examples. To put it in terms of our current study, the deviations from typical situations would have to be suitably accounted for.

There are additional practical considerations that reduce the usefulness of published unit bid prices. Since the design of ground support system, bracing system and decking system is often left to the option of the contractor, payment for these activities are seldom made as separate items and may be lumped together in the unit price for excavation. (Decking and ground support are sometimes bid on a square yard basis.) This simplifies payment for the owner who need not guess at what unit price items to include for those unspecified items, but it does not aid the researcher trying to separate the cost of these major items. To add to the unit price confusion ground water is often lumped into the potpourri of "excavation."

Nor do contractors help to simplify the situation; their contribution to complexity is known as "unbalancing the bid." To understand what is meant by unbalancing a bid, it is necessary to understand the estimating and bidding procedure. To compare bids of various contractors and provide for fluctuations of quantities of work, most contracts are bid on unit prices. The owner specifies the approximate quantities of work to be performed and the contractor states the price he wants to be paid for each unit of work. The cumulative total represents the total bid price. The direct cost of an item of work consists of the labor, materials, equipment and sub-contractor payments that contribute directly to the particular pay item cost. In addition to direct costs there are other costs that are general in scope, and not easily definable to individual direct cost items. These include general and administrative costs, plant set up costs, time oriented costs that are sustained regardless of which work items are in progress, and contingencies and profit. Collectively these amounts can equal 40% to 100% of the definable direct cost items. The percentage is lower in building and other above grade construction and higher in underground construction where there are more intangible conditions.

When preparing a bid, the contractor must spread these general costs, contingencies and profit over the contract pay items; if they are spread proportionately the bid is considered "balanced," if not, it is "unbalanced." The theories and strategies used by contractor in unbalancing bids are complex. Reasons for unbalancing can be either legitimate or devious, depending on motive and extent. Actually, spreading general costs proportionately does not truly balance them, since the general costs themselves are not generated equally proportionate to all direct costs. When unit prices include furnishing materials such as cement, reinforcing steel, structural steel and miscellaneous iron, the general costs attributed to these items should be lower than those where considerable labor, equipment and supervision are involved. The same is true of items performed by subcontractors requiring only a minimum of supervision. Instead of limiting unbalancing to these items, the contractor may favor early completion items with a large portion of general costs, to increase payments at the beginning of the job and reduce financing costs. Items that are likely to overrun in quantities may be bid high; underruns bid low.

The purpose of including such a discussion here is not to attempt to pass judgement on current bidding procedures, but merely to describe factually the situation that exists in competitive bidding, and to explain the reluctance to rely only on published unit prices.

It must be noted that this discussion concerns the major costs of a project. Very often there are a large number of unit prices of small quantity items that may represent only 5% to 20% of the total cost of the project in aggregate. This may include items done by subcontractors such as utility work, electrical installation, mechanical installation and paving items. These items are less likely to be unbalanced, and published unit prices, viewed prudently, will yield reasonable answers.

5.3 PRODUCTION COST DATA

If published unit prices cannot be relied on for complete cost data the obvious solution is to acquire this cost data at its source, from on-going or recent completed projects. This is not easy. A contractor is usually reluctant to divulge his actual costs. His past production records are part of his assets, which he relies on for bidding future work. Several contractors have been contacted and have agreed to allow use of cost data from current and recently completed cut-and-cover projects. These projects include BART and WMATA rapid transit line sections and stations in urban areas utilizing soldier pile and lagging, and S.P.T.C. wall ground support. The methods to be employed for incorporating this contractor supplied data into the evaluation process will be discussed more fully in Section 5.4. Contractor supplied data will not be identified by source.

It is anticipated that the most useful and easily assimilated data will be basic information on size and composition of work crews, type of equipment and actual work hours, amount of materials used, and actual production rates for various work items. Other useful data are the number of supervisory and administrative personnel, and type of plant required. Use of these basic data makes it easier to compare and combine information from different projects.

The information received from contractors will be added to existing in-house data contained in approximately twelve detailed pre-bid estimates of recent cut-and-cover projects covering highway tunnels and sections of the BARTD and WMATA rapid transit systems. Useful basic resource data can be abstracted for use in this study as required. These estimates were prepared using the Jacobs computer estimating program, in the same manner, and degree of detail, that a knowledgeable contractor employs when estimating a multi-million dollar project. This proprietary program will be used in preparing the estimates indicated in the tabular format of Figure 18.

5.3.1 Composition of total contract costs - The analytical expressions described in Section 5.1 express total cost as the sum of individual activity costs. Section 5.4 will show how each activity is the sum of individual operations which in turn represent the sum of costs of basic resources that perform the operation. The total of all basic activities A through K represent the total direct cost of a project or those costs of resources that can be directly attributable to a particular work item (or activity). Indirect costs N through Q are more general in nature consisting of plant and overhead costs. These contain certain fixed costs, such as bond cost, and others that are time oriented, such as supervisor payroll, varying directly with the time (duration) required to perform the work. The chart on Figure 20 shows the relationship of direct costs, general costs, contingency costs and contractor's markup that constitutes the total cost to the owner for the construction work performed.

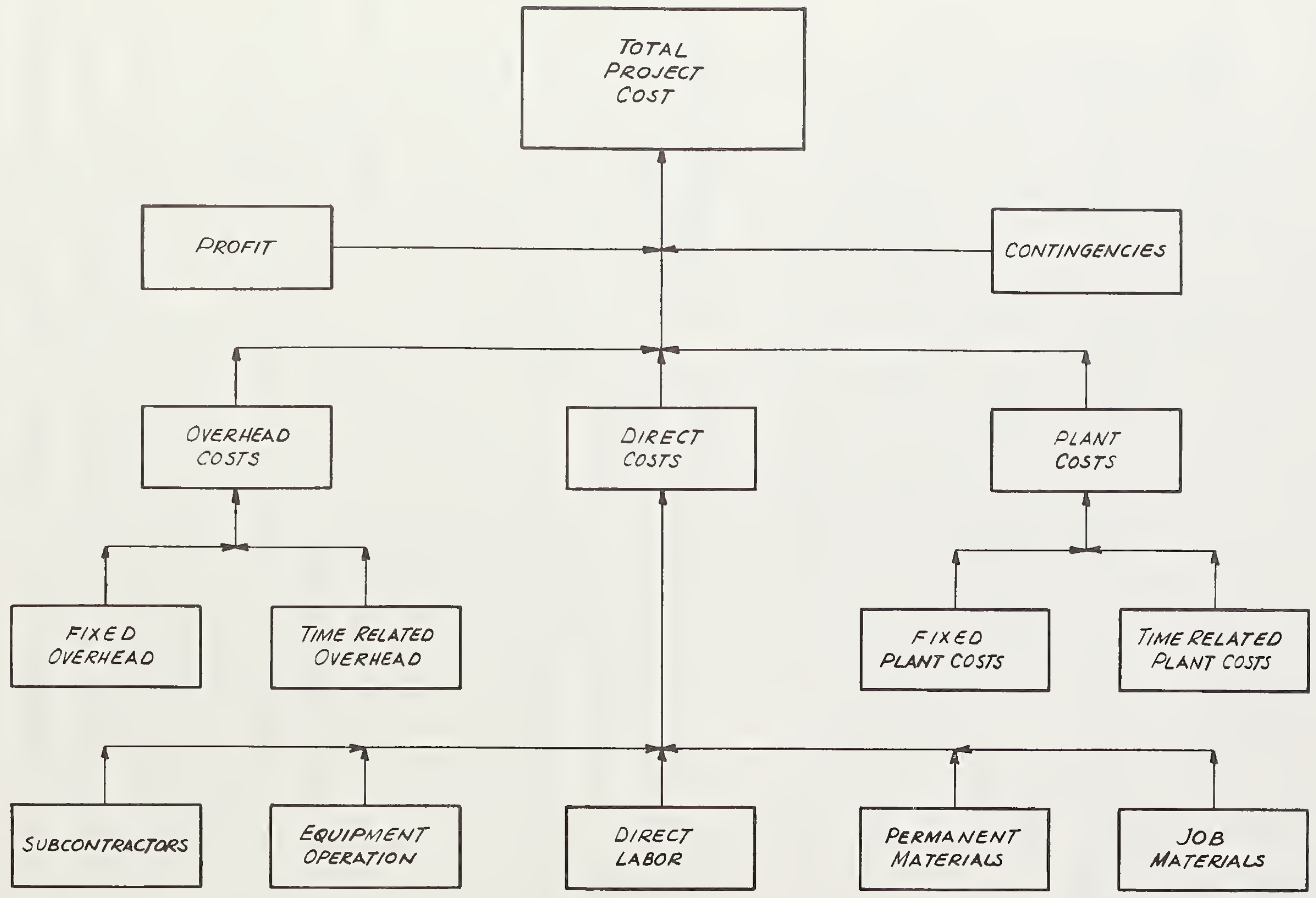

TOTAL PROJECT COST
PROFIT
CONTINGENCIES
OVERHEAD COSTS
DIRECT COSTS
PLANT COSTS
FIXED OVERHEAD
TIME RELATED OVERHEAD
FIXED PLANT COSTS
TIME RELATED PLANT COSTS
SUBCONTRACTORS
EQUIPMENT OPERATION
DIRECT LABOR
PERMANENT MATERIALS
JOB MATERIALS

Figure 20

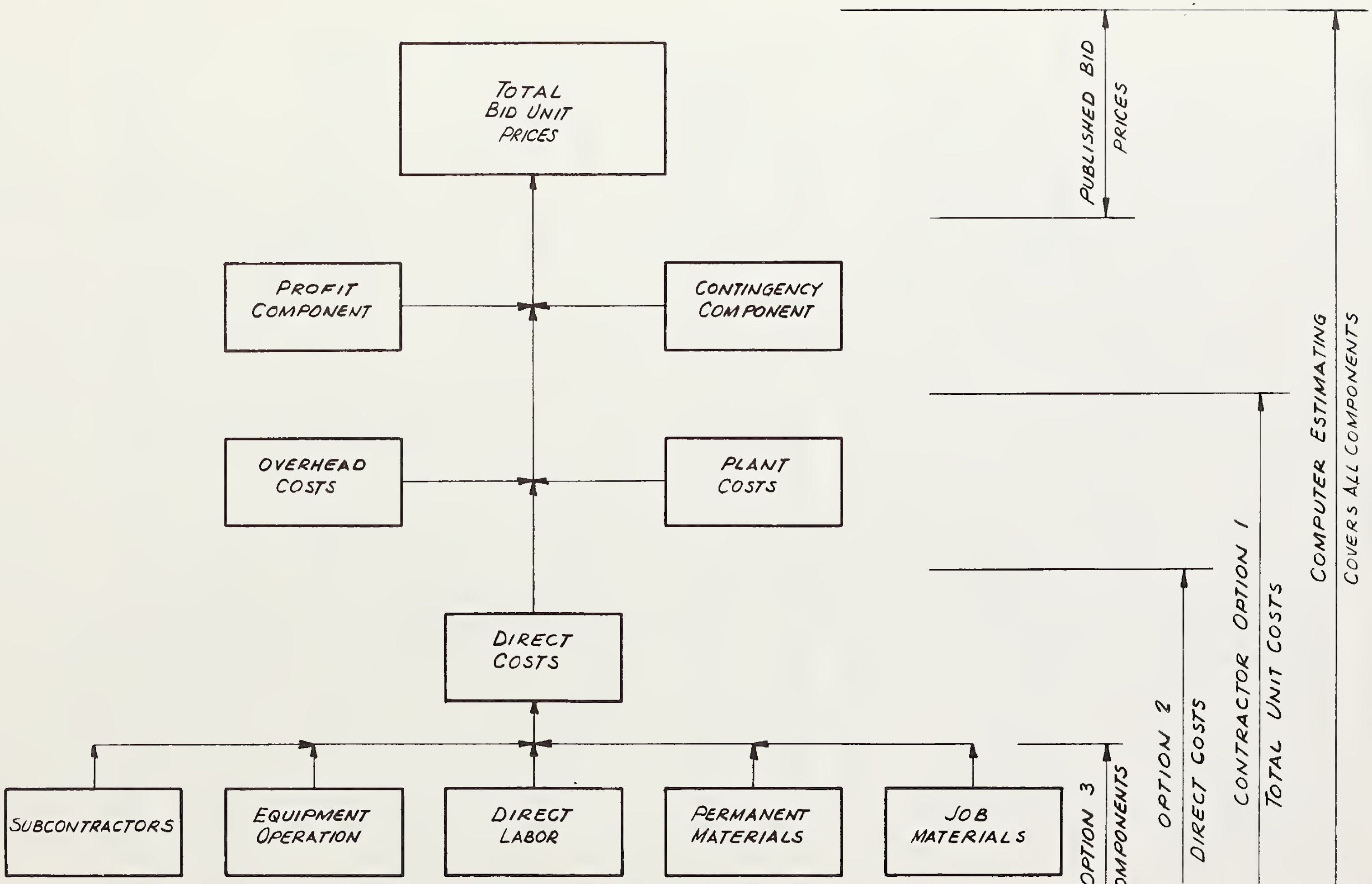
5.3.2 Comparing and combining cost figures - If actual costs, estimated costs and published unit bid prices are to be used effectively, it is important to know the composition of each set of numbers. To compare or combine costs it is necessary that they include the same basic components. It has been shown that published unit prices contain not only direct costs, but a proportion of indirect costs and contractor markup (contingencies and profit). Cost data received from contractors follow several options depending on the cost account system used and the generosity of the contractor.

Cost option 1 - Total unit costs - This is the closest to unit bid prices except that it does not include the contractors markup. The units might be comparable to the bid price units but in all likelihood would be divided into work operations: installing lagging, installing bracing, etc. Total unit costs would include a proportion of indirect costs.

Cost option 2 - Total unit direct costs - This is similar to option 1 except it does not include indirect costs. In each case it must be ascertained which basic components (such as equipment ownership costs) the contractor includes as direct costs and which are included in the indirects.

Cost option 3 - Basic cost components - This is most similar to the information used in the computer estimating program. It consists of production rates, work crew sizes and rates, equipment rates and material prices, at least for direct costs, and hopefully, for indirect costs.

A comparison of each of these cost data systems is shown in Figure 21. This chart indicates how contractor costs can be compared to bid unit prices by making reasonable assumptions for missing components. The figure also illustrates the impossibility of trying to reverse the procedure and breakdown bid prices to meaningful components with any degree of certainty.



UNIT PRICES — COST COMPARAISON CHART

Figure 21

5.4 ESTIMATING PROCEDURES

To properly evaluate the cost of performing a proposed cut-and-cover construction contract for bidding purposes, a contractor must follow a logical sequence of activities to build the project on paper. He must decide on a design of ground support, and if necessary, decking. He must select his work crews, decide whether to purchase or rent equipment, acquire material quotes, and contact suitable subcontractors for proposals. He must inspect the job site, consider who is available for supervisory positions and plan a sequence of job events that will complete the project on schedule. This cannot be done by simply entering a series of unit prices on a bid sheet. The unit bid prices, whether they constitute the low bid or the high bid are arrived at through a slow and sometimes tortuous estimating process incorporating experience, optimism and caution. Right or wrong, optimistic or pessimistic, the prices represent the value of the cost of construction to men who are willing to back their decision with their money, and reputations. It is this type of detailed procedure that will be used for the preparation of estimates needed for value analysis.

5.4.1 Definition of computer estimating program procedures -

a. Purpose. The purpose of a construction cost estimate is to prepare a detailed analysis of the cost to construct a project, using the methods and restraints that are either specified in the project documents or developed as part of the estimating process. An estimate is also used to compare the cost effectiveness of alternate methods of construction.

b. Work operation concept. The successful completion of a construction project is dependent on the efficient execution of a multitude of individual activites, tasks or work operations. A work operation is defined as a construction task that is identifable as an entity by the nature of the work, quantity measurement, and cost allocation. The total cost of construction of a project is the sum of the costs for each work item.

c. Resource group concept. A work operation is performed by a balanced group of resources: labor, equipment and materials; e.g. for the work operation of erected forms the necessary resources may be carpenters, crane operator, laborers, a crane and form materials. The estimating process involves selecting the most suitable resource group to perform a construction operation using the most efficient construction method.

d. Work study estimating. Work study estimating requires that an estimator make a detailed analysis of the resources needed to perform a construction operation using a particular method. Such an analysis will determine the operation work quantity, unit of measure, and the combined production rate of the various resources to be used to perform that operation.

e. Productivity estimating. Productivity estimating is the method whereby the resources required to perform a work operation are expressed as the number of resource units per unit of work quantity, e.g. two manhours per cubic yard of concrete. This method of estimating is commonly used in building work.

f. Unit price estimating. Unit price estimating is similar to productivity estimating, except that the resources required are expressed in terms of dollars per unit of work quantity, rather than in terms of resource units per unit of work quantity.

g. Lump sum estimating. Lump sum estimating is the method whereby a lump sum of money in included in the estimate to represent the total cost of performing a particular operation or where a resource quantity is entered as a total number of hours. This method is generally used where it is difficult to express a meaningful work quantity.

The estimate is comprised of a sum total of work items (or activities) each consisting of a logical sequence of individual operations, which may be estimated by one or another of the procedures listed above as most appropriate to the particular operation.

5.4.2 Estimating resource library The basic building blocks of each work operation are retrieved from a common library of resources prepared for the particular estimate. Each labor category is represented, together with all types of construction equipment and materials likely to be used on the project. The individual resources can be combined to form typical group resources. Individual work operations draw on this library of resources for those units needed. Each unit is specified at an appropriate rate. Labor rates include base salary, labor burdens, vacation, etc. Equipment rates can include operation, maintenance and ownership costs. Material rates are determined by the time and location of the contract work.

5.4.3 Estimating rates for cut-and-cover study - The rates included in the estimating resource library will be delineated in the Appendix of Volume II of this report to allow comparison with projects in another time and space frame. The labor rates will be the actual rates for each trade category in effect in mid 1974 in Washington, D.C., as recorded by The Master Builders Association. Equipment and material rates used will also be specified for future comparison. In utilizing the results of this study, it will be possible to apply a factor for each of these categories applicable to the time and location of the project being considered based on actual costs then in effect. Construction cost indexes supplied by the U.S. Department of Commerce are useful for this purpose.

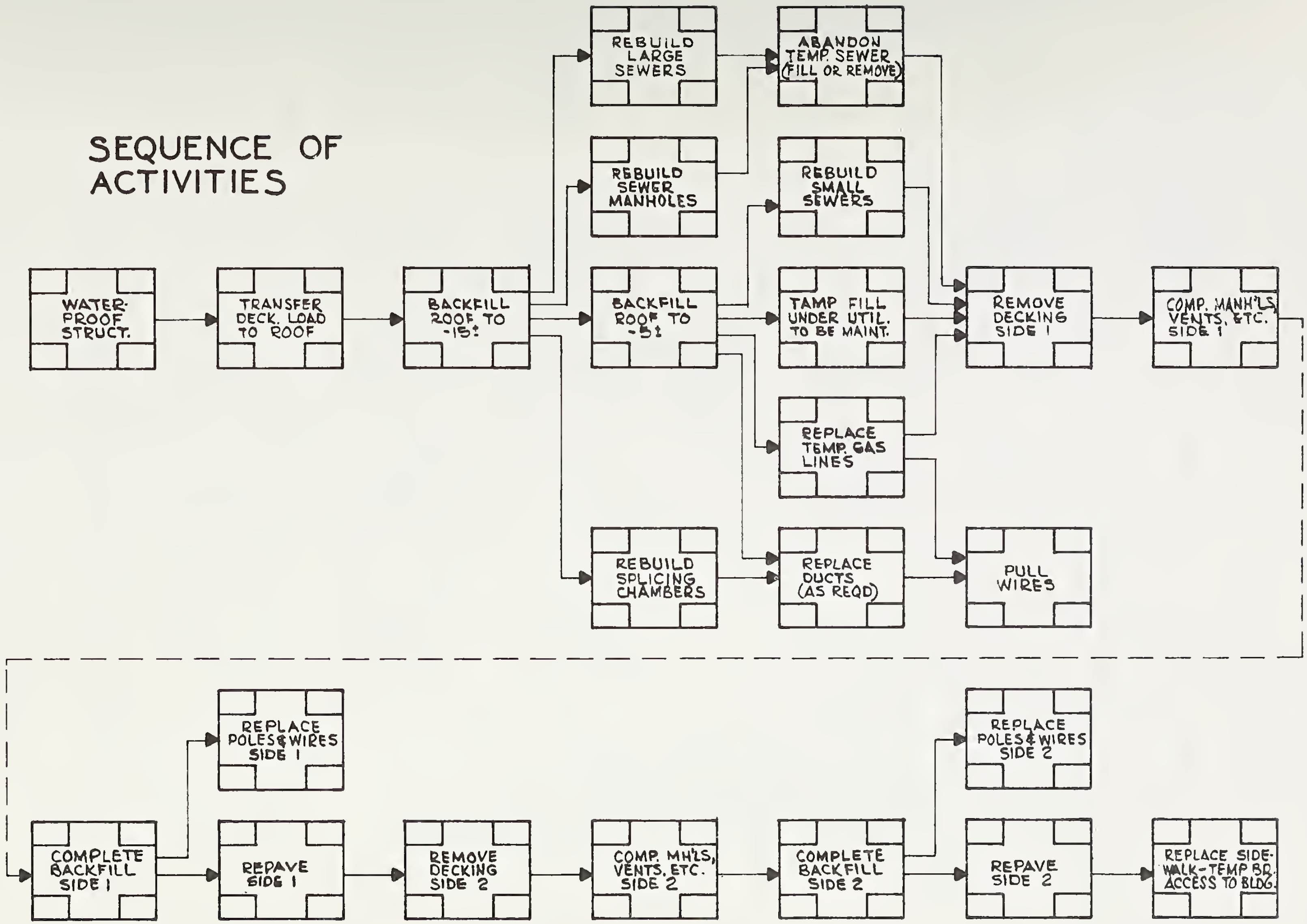
5.5 ACTIVITY NETWORKS

While the total cost of a project is the arithmetic sum of all its parts, the same is not true of the time required to complete the project. Operations and activities may be performed simultaneously, or may overlap when the start of one is dependent on partial completion of another. The more operations that can be performed efficiently at one time, the shorter the job duration. There are time related overhead and plant costs that may amount to more than 15% of total costs. For this reason, determining the activity and project durations is important in estimating as well as being necessary for proper construction planning. Activity networks graphically represent the activity sequence and are used to determine project duration. Using either a Critical Path Method (CPM) or Precedence Diagram Method (PDM), the contractor can tell which items must be performed at a particular time, and which have "float" time and can be delayed without extending the job duration. The network is most useful for monitoring the work during construction. Delay of an operation beyond its float time may place it on the critical path and change the remaining network.

On a complex project such as those considered for this study, a typical network would contain several hundred activities. This would be impractical for the number of possible alternate estimates required and this type of detail not necessary for the purpose of determining project duration. Simplified precedence networks of the type shown in Figure 22 and later in Section 5.6 are sufficient for establishing an approximate time schedule and presenting it graphically. This type of network was chosen primarily because it is easier to follow in this presentation than the more common "arrow" or "i-j" networks. The network in Figure 22 shows typical activities for the restoration stage of the project. It shows in detail the interrelationships of sub-activities of backfill and restoration depicted in the last sketch of Figure 9. The networks in Section 5.6 will show entire projects with this restoration portion simplified for ease of presentation.

Activity networks are a tool for the estimator. The dates, durations, lead and lag times cannot be included till they have been determined during the course of preparing an estimate. They will then be used to determine time oriented costs.

SEQUENCE OF ACTIVITIES



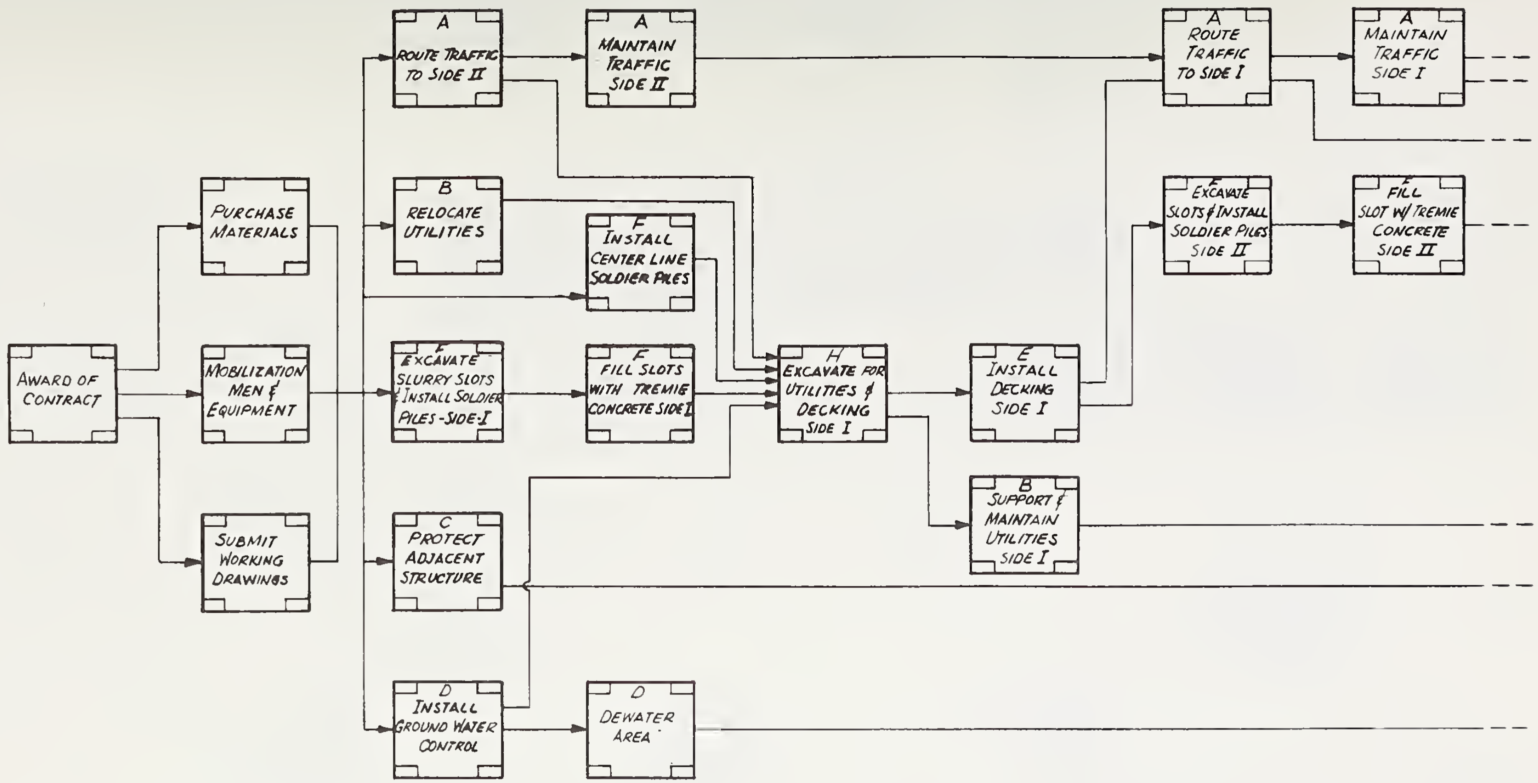
K. RESTORATION (INCLUDING BACKFILL, UTILITIES & REPAVING)

Figure 22

5.6 USE OF THE ESTIMATING PROGRAM FOR REMAINING TASKS

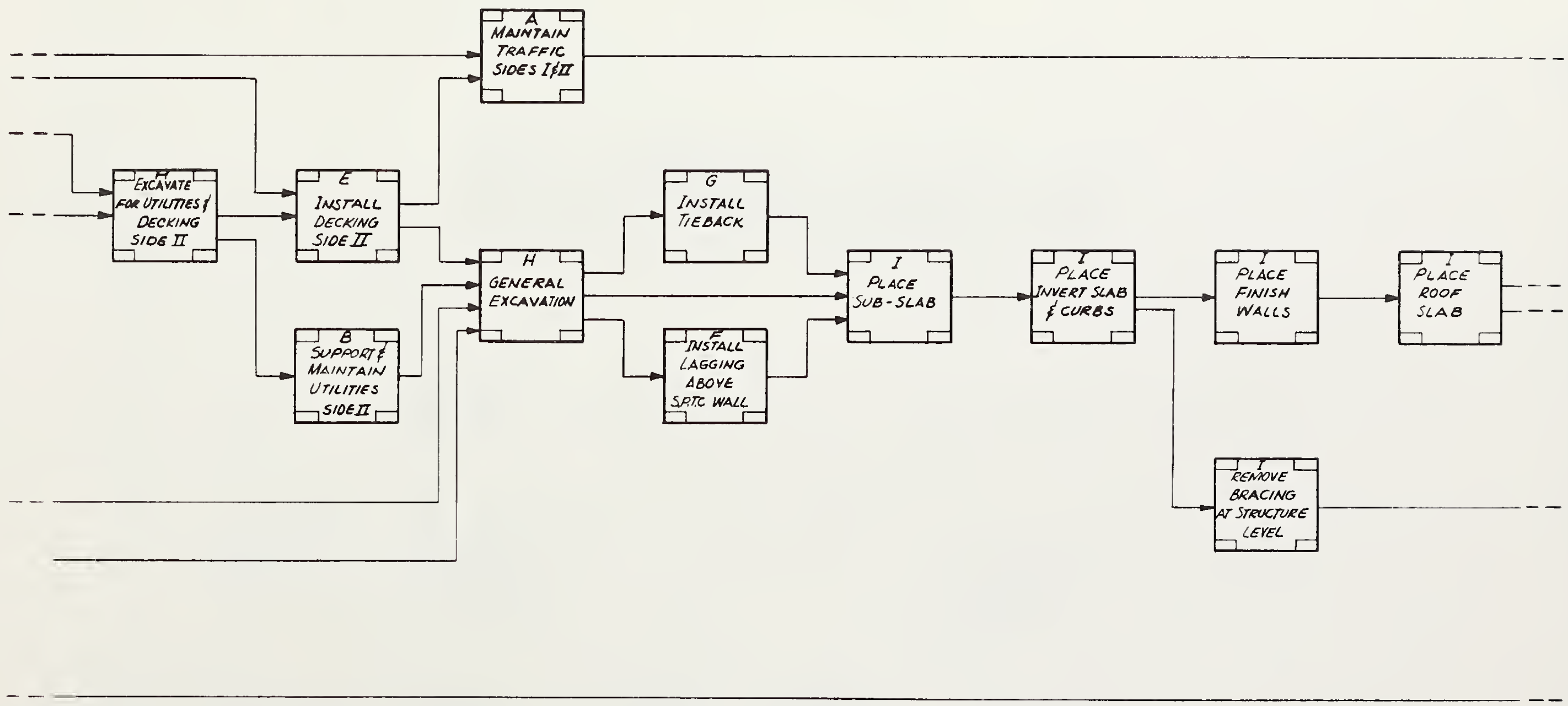
The use of the Jacobs computer estimating program will aid in evaluating the alternate construction method variations shown in Figure 18. Without it the preparation and comparison of so many estimates would be extremely time consuming unless it were simplified with the sacrifice of much useful detail. Once the multiple estimates have been established, extending them to new situations will be relatively simple using unit price variations developed by computer. After the optimization evaluation has been completed the method will be used to develop comparable costs for the under-the-roof construction, and partially inverted construction, required for testing the optimization process.

In order to achieve the utmost benefit of the computer program for estimating various alternates, three cut-and-cover tunnel systems have been chosen of the 96 possible alternates described in Section 5.1, to begin the evaluation process. These three systems, to be described independently, represent a maximum variety of basic activity, site and construction variables. Not only do these present at least one variety of each important variable, they are sufficiently representative to establish a proven rate libra _y that can be used for any combination of factors. The three estimates will include all three types of structures, three ground support systems, two bracing systems, both length and width possibilities of decking, and all major variations of utility handling. For options such as depth variation and ground water control, maximum requirement conditions will be used. Many variations with less than maximum requirement conditions need only have one or more operations dropped to be applicable to the new condition. Each of the three basic estimate situations will be described, expressed in terms of activity, site and construction variables and graphically illustrated by an activity network.



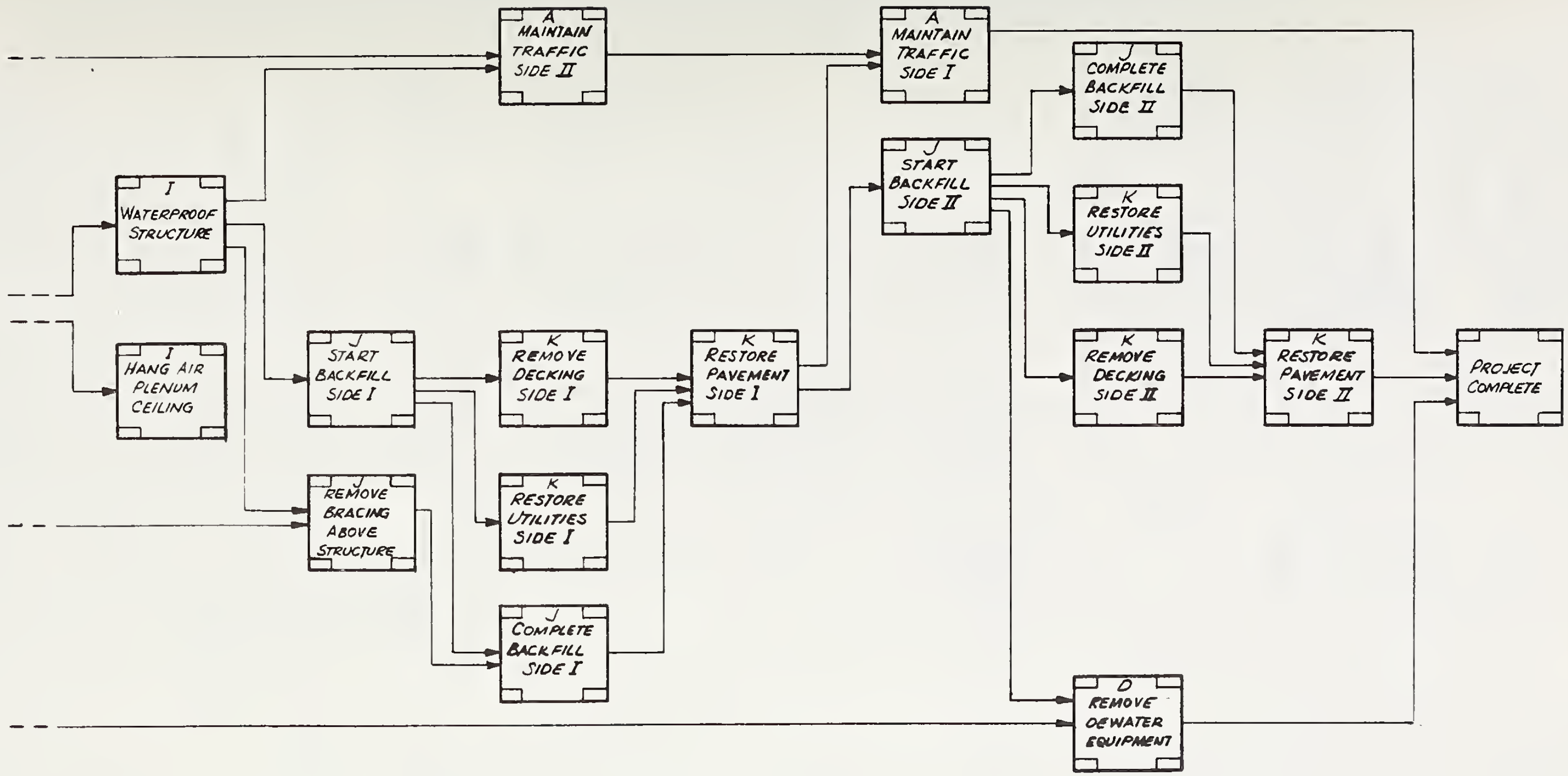
ACTIVITY NETWORK - PROJECT 1KY
STRUCTURE: FOUR LANE HIGHWAY TUNNEL
GROUND SUPPORT: CAST-IN-SLURRY WALL
BRACING: TIEBACKS

Figure 23 - Sheet 1



ACTIVITY NETWORK - PROJECT 1KY
STRUCTURE: FOUR LANE HIGHWAY TUNNEL
GROUND SUPPORT: CAST-IN-SLURRY WALL
BRACING: TIEBACKS

Figure 23 - Sheet 2



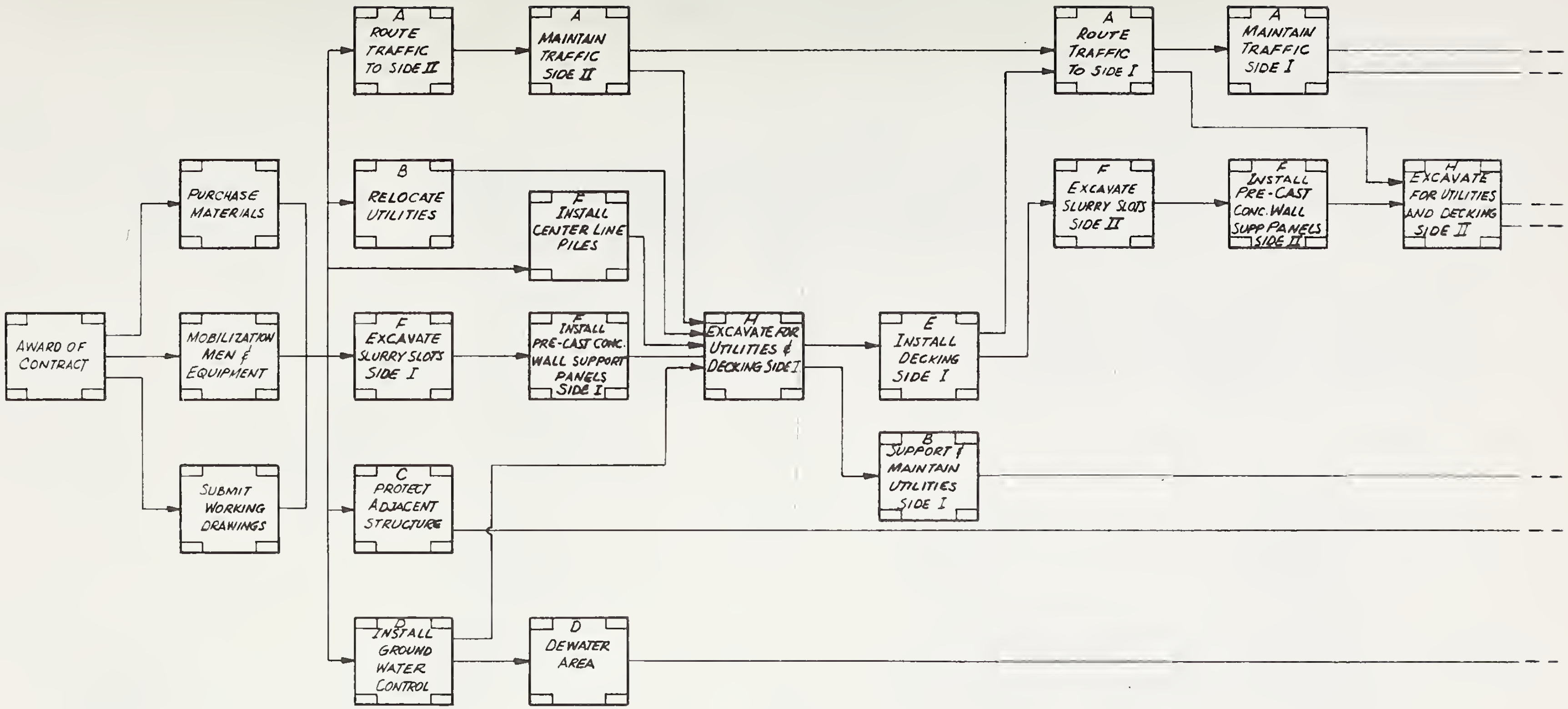
ACTIVITY NETWORK - PROJECT 1KY
STRUCTURE: FOUR LANE HIGHWAY TUNNEL
GROUND SUPPORT: CAST-IN-SLURRY WALL
BRACING: TIEBACKS

Figure 23 - Sheet 3

5.6.1 Highway tunnel estimate - This estimate (1KY) will be based on use of an S.P.T.C. cast-in-slurry ground wall support and tieback bracing. It is excavated 70 feet deep (21.3 m) in wet soil. The structure is 2,000 feet long (610 m) and requires a wide deck with center support. Because of the diaphragm wall, maximum relocation of utilities is required. Special plant costs reflecting the need for a recirculating slurry plant, and special slot digging equipment will be included in the ground support item. The analytical expression for this set of conditions, based on the table in Figure 18 is: 1A00 + 1BD0 + 1CD0 + 1DDY + 1E00 + 1FBY + 1GFY + 1HFQ + 1IBR + 1JFQ + 1K00 + 1N00 + 1OKQ + 1P00 + 1QKQ. The activity network presenting a graphic representation of this set of variables is shown in Figure 23.

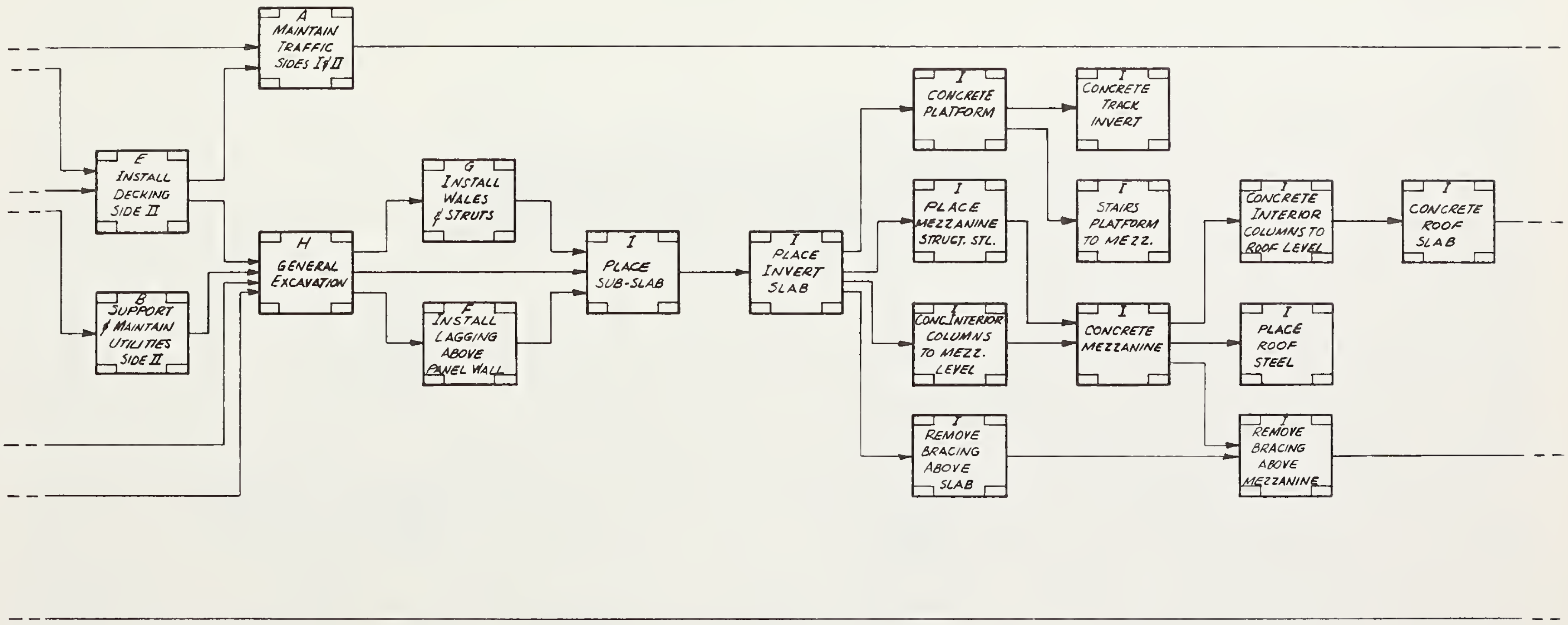
5.6.2 Rapid transit station estimate - This estimate (2LY) will be based on use of a precast concrete groundwall support and internal bracing. It is excavated 70 feet deep (21.3 m) in wet soil. The structure is 700 feet long (213 m) and requires a wide deck with center support. Because of the diaphragm wall, maximum relocation of utilities is required. Special plant costs will reflect the need for a slurry plant, casting yard, and special heavy slab handling equipment, and will be included in the ground support item. The analytical expression for this set of conditions, based on the table in Figure 18 is: 2A00 + 2BD0 + 2CD0 + 2DDY + 2E00 + 2FCY + 2GEQ + 2HEQ + 2ICR + 2JEQ + 2K00 + 2N00 + 2OLQ + 2P00 + 2QLQ. The activity network presenting a graphic representation of this set of variables is shown in Figure 24.

5.6.3 Rapid transit line structure estimate - this estimate (3GY) will be based on the use of soldier piles and lagging for ground wall support and internal bracing. It is excavated 70 feet deep (21.3 m) in wet soil. The strucutre is 2,000 feet lone (610 m) and utilizes a narrow deck without center support. The soldier pile ground support permits minimum utility relocation. Overhead and plant costs are low compared to other systems, but require drilling equipment for placing



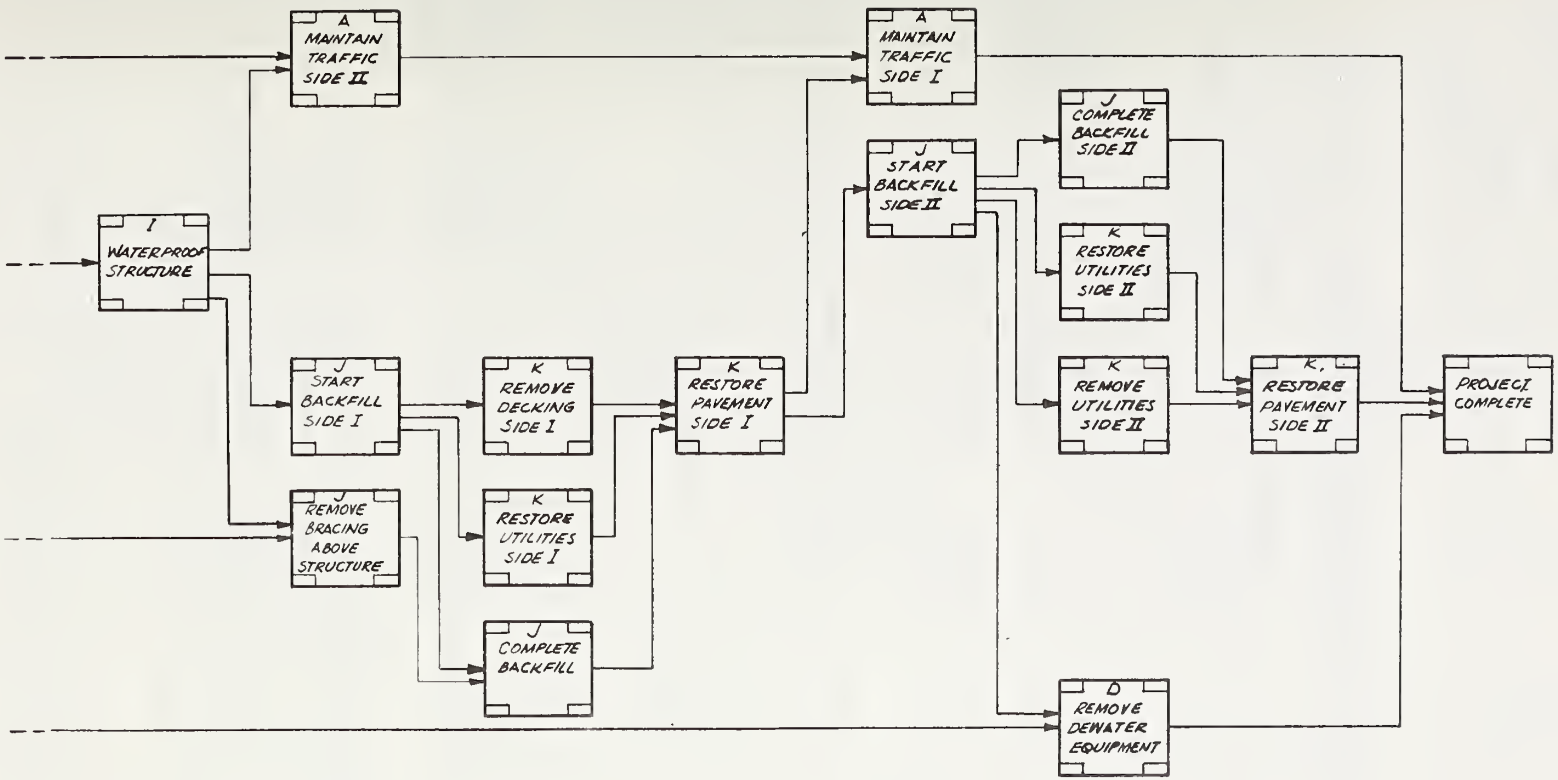
ACTIVITY NETWORK - PROJECT 2LY
STRUCTURE: RAPID TRANSIT STATION
GROUND SUPPORT: PRECAST CONCRETE PANELS
BRACING: WALES AND STRUTS

Figure 24- Sheet 1



ACTIVITY NETWORK - PROJECT 2LY
STRUCTURE: RAPID TRANSIT STATION
GROUND SUPPORT: PRECAST CONCRETE PANELS
BRACING: WALES AND STRUTS

Figure 24 - Sheet 2



ACTIVITY NETWORK - PROJECT 2LY
STRUCTURE: RAPID TRANSIT STATION
GROUND SUPPORT: PRECAST CONCRETE PANELS
BRACING: WALES AND STRUTS

Figure 24 - Sheet 3

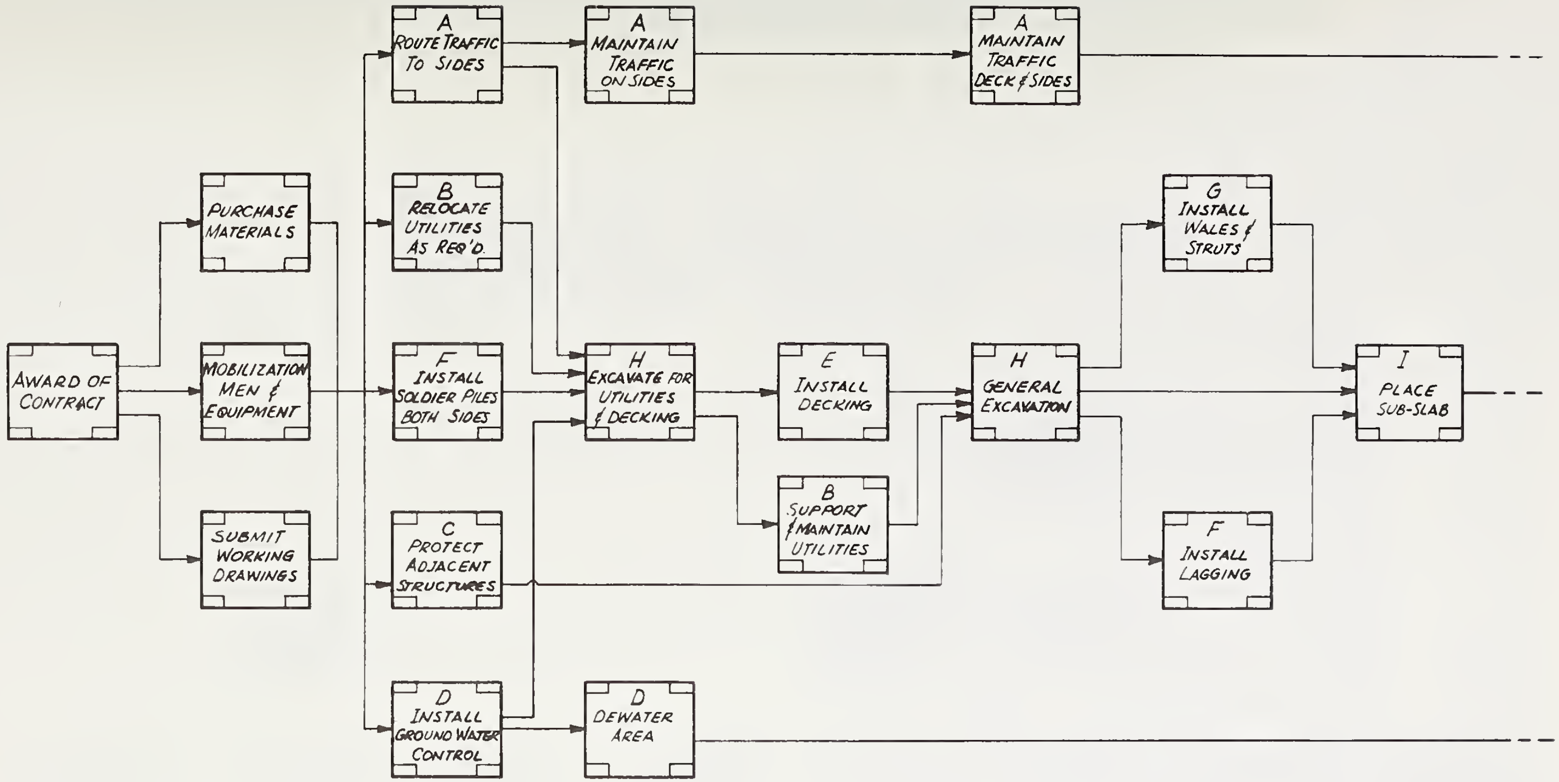
piles. The analytical expression for this set of conditions based on the table in Figure 18 is: 3A00 + 3BA0 + 3C00 + 3DAY + 3E00 + 3FAQ + 3GEQ + 3HEQ + 3IAR + 3JEQ + 3K00 + 3N00 + 3OGQ + 3P00 + 3QGQ. The activity network presenting a graphic representation of this set of conditions is shown in Figure 25.

5.6.4 Variations of basic estimating conditions - The three basic estimates described will be produced concurrently using the computer program based on a common input resource library. Once these three estimates have been proven satisfactory and free from technical computer language errors, they will be utilized to produce new variable estimates rather than building entire new estimates from basic data. In some cases variations in quantities will suffice, in others operations or whole items will need to be changed. These changes will be made by computer or manually depending on the complexity of the changes required.

The first three estimates will be analyzed to determine a range of percentage of total cost for all items of direct and indirect costs. Minor variations of change, such as material quantity changes that indicate an insignificant effect on overall costs, may be eliminated from consideration. Quantification of the cost of construction disruption will be considered in this later portion of the study. The range of permissible data input in the estimating program is sufficient to admit any reasonable expression of disruption evaluation that may be developed.

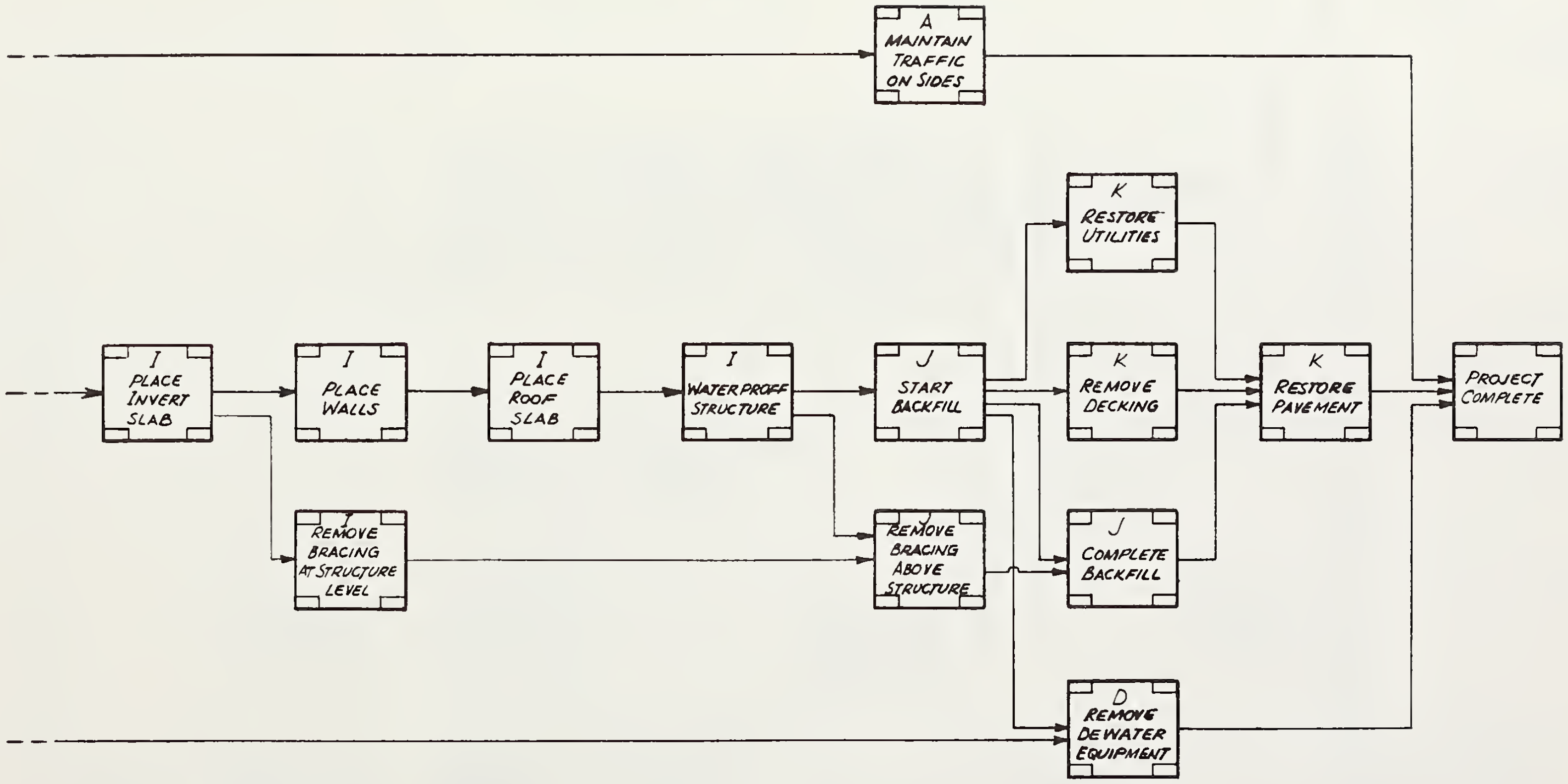
5.6.5 Evaluating current production cost data - Previous paragraphs of this section have emphasized some of the problems of obtaining and effectively evaluating production cost data from active construction projects. Any work plan used to obtain such data must provide for three basic requirements.

a. Ability to relate all cost components (labor, material and equipment) to a common base or price index.



ACTIVITY NETWORK - PROJECT 3GY
STRUCTURE: RAPID TRANSIT LINE SECTION
GROUND SUPPORT: SOLDIER PILES AND LAGGING
BRACING: WALES AND STRUTS

Figure 25 - Sheet 1



ACTIVITY NETWORK - PROJECT 3GY
STRUCTURE: RAPID TRANSIT LINE SECTION
GROUND SUPPORT: SOLDIER PILES AND LAGGING
BRACING: WALES AND STRUTS

Figure 25 - Sheet 2

b. Ability to identify, evaluate and compare similar work types and quantities.

c. Ability to relate and compare relevant requirements of different projects on the basis of components, units or categories of cost.

The basic cost estimates in Section 5.4 will provide needed information, detail and format by which these requirements can be met. An additional potential use of the multiple estimates to be obtained would be to compare unit prices from previous projects. This would entail the development of appropriate yearly cost indexes to arrive at a common basis for comparison. By this means it may be possible to synthesize past construction cost data to extend cost curves for major activities based on such parameters as depth of excavation, type of ground support, type of structure, etc.

The results obtained from the multiple estimates, together with conclusions and recommendations, are included in Volume II of this report.

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