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PREFABRICATED STRUCTURAL MEMBERS FOR CUT-AND-COVER TUNNELS

Vol. 1. Design Concepts

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FOREWORD

The report summarizes the results of a study by The Consulting Engineers Group, Inc., to determine the applicability of prefabricated structural members to cut-and-cover tunnel construction. This volume describes design concepts for both the overall tunnel and the structural members. The second volume presents detailed designs for three actual tunnel sites.

Sufficient copies of the report are being distributed to provide two copies to each regional office, one copy to each division office, and two copies to each State highway agency. Direct distribution is being made to the division offices.

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Richard E. Hay /for

Charles F. Scheffey
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Federal Highway Administration

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16. Abstract

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

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

This report is the first of two volumes. Volume 2, FHWA-RD-76-114, "Three Case Studies," summarizes designs and cost considerations applicable to three actual tunnel sites.

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This report contains the results of a study of the use of prefabricated structural members for cut-and-cover tunnels. The study was conducted by The Consulting Engineers Group, Inc., with the assistance of Consultants Ben C. Gerwick, Jr., Frank T. Wheby, and Soil Testing Services, Inc. It was performed under Contract No. DOT-FH-11-8594 with the Department of Transportation. Contract Administrator was Ms. Shirl Vendemia and the Contract Manager was Mr. J. R. Sallberg.

Volume one covers the initial concept development which was designated as Task A of the contract. During the course of the study, many knowledgeable individuals were interviewed, including contractors and engineers working on the New York City, Washington, D. C., Atlanta, Chicago, and Edmonton subway projects. Particularly helpful were Mr. George Tamaro of ICOS of America, Mr. Gilbert Tallard of Soletanche and Rodio, Inc. for their experience in diaphragm wall construction, and Mr. Richard E. Steen of Washington, D. C., who provided valuable insight into the problems that subway construction can cause to nearly all business establishments.

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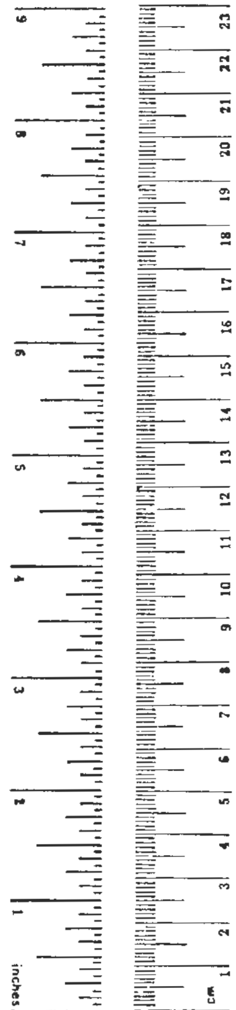
METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

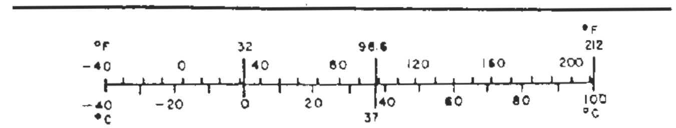
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2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

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

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



I. INTRODUCTION

Construction of the Bay Area Rapid Transit (BART) and the Metropolitan Washington, D. C. subway system have focused attention on the need for improvement in cut-and-cover tunneling techniques in urban environments. These two projects represent the most comprehensive underground transportation systems ever undertaken in this country. During their construction, there has been so much disruption to the surface activities for so long a period of time, that residents, visitors, and businesses in the area have wondered--sometimes very vocally--whether the end product is worth the effort.

This study and other related studies sponsored by the United States Department of Transportation are aimed at improving cut-and-cover tunnel designs and construction methods, particularly methods that could significantly reduce construction costs and urban disruption. The specific purpose of this study is to examine the possibilities of using prefabricated structural members to help accomplish these aims.

The task of developing new construction techniques can be frustrating, if not futile. Construction is an evolutionary rather than revolutionary type of industry. This is not because those in the construction industry are any less creative than those in other industries, but they are restricted by a combination of economic and legal factors to a "slow and easy" approach to change.

There is usually no incentive for consulting engineers or contractors to assume the risks inherent in innovative designs. Owners, especially public agencies, are reluctant to subject themselves to the criticism that could result if an untested construction method does not perform satisfactorily. It is clear that any "innovative concepts" suggesting change in traditional construction materials or methods must be compatible with the evolutionary nature of construction in order to be useful.

Specialization within the general discipline of Civil Engineering may contribute to the slow acceptance of new construction materials and methods. Structures are usually classed as "buildings", "bridges", or "heavy structures" (such as dams and tunnels). The owners, designers and builders of these different classes are usually different, and developments within one class of structure are often not recognized as being applicable to another. Thus, what may be a relatively commonplace method of construction in one type of structure might be considered "innovation" when applied to another.

A similar reluctance to accept construction methods developed in other countries (or even in other states, in some cases) has often slowed the evolutionary process in construction. While there are usually very good reasons why a total concept may not be readily transferable from one class of construction to another, or from one country to another, adoption in a partial or modified form would certainly seem to be practical.

This study is intended to focus on the use of prefabricated structural members in cut-and-cover highway tunnels in urban areas. However, the discussion of materials and methods investigated have broader application than that. The designs of urban mass transit systems, suburban and rural tunnels and highway cuts, subterranean parking structures and other types of underground construction should benefit from the research. While the examples shown may indicate designs consisting totally or mostly of prefabricated elements, the best and most economical solution for any given construction problem may involve a combination of prefabricated members and more conventional in situ methods.

This report suggests several means for using prefabricated structural members in cut-and-cover tunnels. Various shapes and materials are examined and methods of incorporating these shapes into transportation tunnels are described.

While the types of loadings required and design methods are shown in this volume, it is not its purpose to serve as a text on the design of prefabricated structural members. The design sections and appendices contain guidelines and special considerations that may deviate from conventional design methods, but it is necessary to have a fundamental knowledge of both soil mechanics and structural design to fully understand these sections.

Factors which influence costs are described and methods are suggested for reducing costs. However, quantitative cost comparisons with presently used construction methods are not attempted in this volume because of the many site variables. This is done for specific sites in Volume II, "Three Case Studies."

II. POTENTIAL BENEFITS OF PREFABRICATED MEMBERS

The two most serious drawbacks to constructing transportation facilities underground in congested urban areas are: (1) the extremely high cost (in 1975 the Washington Metro subway system is costing upwards of $50 million per mile), and (2) the severe disruption to surface activities. Use of prefabricated structural elements has the potential of reducing both of these.

In order to realize these benefits it is necessary to do more than just substitute the prefabricated element for a conventional member without significant changes in the design and construction process. Several recent studies have suggested, and the investigations in this study have confirmed, that designing the permanent structure to act also as the ground support system during construction is probably the best way to cut costs and reduce construction time. It is the only way that the extensive use of prefabricated elements can make sense. Proof that it works can be found in many projects in Europe and Japan.

In this study, prefabricated wall elements are always assumed to act as ground support walls during construction and as tunnel walls in the permanent structure. In several of the concepts investigated, a permanent deck is installed early in the construction period and all subsequent work is performed under cover. It is obvious that backfilling under such a permanent deck is impractical. Except in those cases of relatively shallow cuts and when utilities need not be permanently located within the excavated area, it is suggested that this space be employed as a utility corridor, with permanent access within the structure, for maintenance of the utilities. It could also be possible, in some cases, to use this created space for commercial purposes. It is a logical place for any ventilation ducts and equipment which may be required for tunnel operation. Elimination of this backfill also has the positive benefit of greatly reducing the loads on the tunnel roof. However, omission of the backfill reduces the overall weight of the tunnel. Where a high water table tends to float the tunnel, other means must be provided to resist this uplift.

Placement of utilities in tunnels or "utilidors" is the subject of other Department of Transportation studies. One such study, "Utility Tunnel Economic Feasibility Analysis in Conjunction with the Construction of Subways by the Cut-and-Cover Method" by the Chicago Urban Transportation District and Consoer, Townsend & Associates has shown a benefit/cost ratio too low to be feasible. However, that study was for a specific project in which the utility tunnel was necessarily a significant add-on cost and the total tunnel design was not changed to take full advantage of the reduced loading effects. (See Fig. 1). Also there was no reduction in the disruption time during construction in the case they studied.

Meaningful cost analyses for general cases is virtually impossible. This "double roof" method of construction may not result in any significant construction cost savings. However, reducing the surface disruption from months or years to weeks, would have a very positive economic impact, especially on businesses in the area. This cost of business disruption and public inconvenience has not been evaluated quantitatively. Even if it could be evaluated, there is a serious question as to who should bear the burden of this cost. Public agency owners are reluctant to pay for it, since the effect is on a very few individual businesses, but to assess the businesses affected for the added cost of faster construction procedures that would reduce their disruption would be unjust.

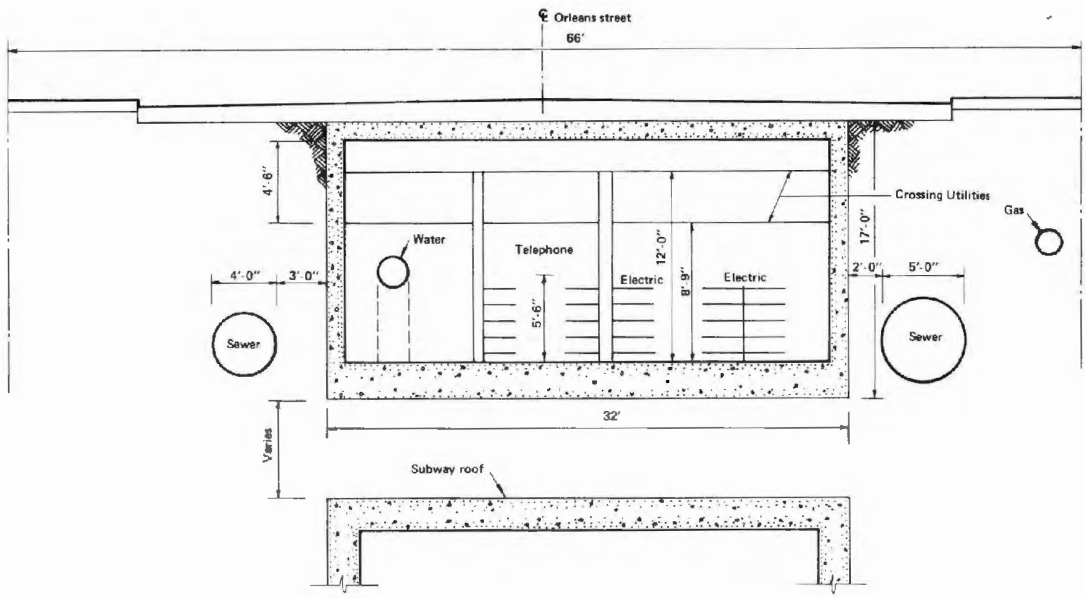


Fig. 1 Utility tunnel section from "Utility Tunnel Feasibility Analysis---"(See text)

Fig. 2 Cut-and-cover tunnel construction may cause severe business disruption

These economic and social impacts of cut-and-cover construction are the subject of other on-going research studies, and it is inappropriate to discuss them further in this report. However, the factors mentioned here do have a substantial effect on the feasibility of using prefabricated members.

III. PREFABRICATED STRUCTURAL MEMBERS

Prefabrication of structural members is not a new phenomenon in the construction industry. Various degrees of prefabrication have been used for years. For example, the practice of fabricating steel beams and columns to the proper length and attaching such items as connection angles and bearing plates before shipping to the job site is so common that it is not usually even included in discussions on "prefabrication". However, the reasons behind this practice are essentially the same as those advanced for the use of larger and more sophisticated prefabrication which is often regarded as a "new" construction technique. These reasons are:

1. Centralization of fabricating equipment allows the use of larger, more costly and more sophisticated equipment with a higher degree of automation.
2. The use of this equipment, plus a more closely controlled production environment produces a more consistent quality.
3. The use of this equipment, plus the lower wage rates for plant labor in comparison with field labor can result in lower unit costs.
4. Construction schedules can be telescoped by prefabricating and stockpiling the structural members.

Cost savings from plant prefabrication are offset to some degree by:

1. The cost of transporting the members to the job site.
2. The often larger equipment required to place the members in their final position.

3. The much higher overhead associated with manufacturing as opposed to construction.

A. PREFABRICATED STEEL ELEMENTS

Steel elements are widely used in cut-and-cover tunnels, with various degrees of prefabrication employed. Following is a list of the types of elements now used, and some possible applications of members not commonly used.

1. Standard rolled sections, particularly wide-flange beams, are the most commonly used members for soldier beams in temporary retaining walls, usually with timber lagging. They are also usually the members employed for struts, although steel pipes are also commonly used.

   Virtually no prefabrication is involved in this use of steel. In fact, the relative simplicity of making field connections and splices by welding, and cutting excess length with cutting torches is often cited as a principal advantage.

   Steel sections are also used in the finished tunnel construction, but extensive prefabrication is seldom employed. For example, sections now being built in the New York City subway use steel rigid frames at about 5 ft on center along the track. Concrete arches span between the frames. This method of construction is essentially the same as that used in the first section of tunnel built around the turn of the century.

   In the ICOS system of diaphragm wall construction, steel soldier piles are designed to carry the lateral loads, with appropriate strutting. A reinforced concrete wall, cast in a

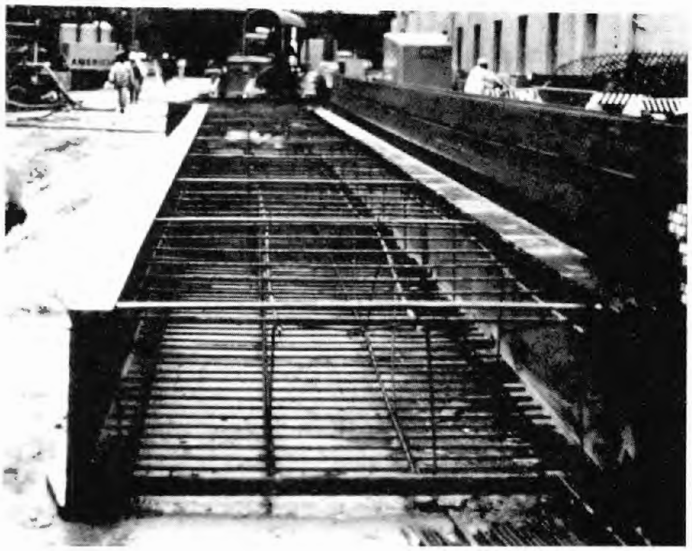


Fig. 3a Prefabricated steel assembly used in subway construction

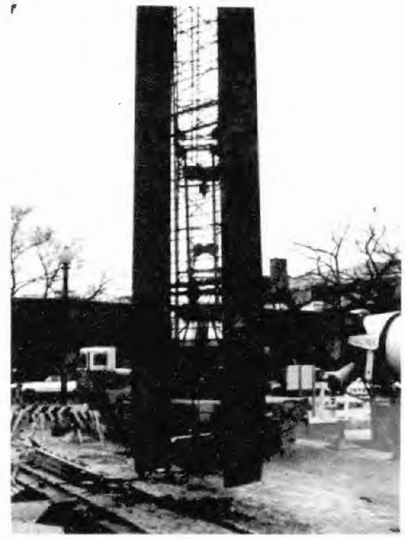


Fig. 3b Inserting assembly into slurry trench

(Photos courtesy ICOS Corporation of America)

slurry trench, spans between these soldier piles. The steel soldier piles and the reinforcing cage for the primary panels are prefabricated as a single unit at the job site and placed in the slurry trench prior to concrete placing.

2. Steel sheet piling is often used for cut-and-cover construction. No prefabrication beyond the initial rolling and cutting to length is usually employed.

3. Steel trusses have received very little consideration in cut-and-cover tunnel work, except for occasional use as a temporary strut or temporary deck support. While the truss is one of the oldest uses of structural steel, it is still one of the most efficient uses of material, if enough depth is available. Its use in buildings (in the form of steel joists) is still very common, but it has lost favor in bridge construction, primarily because of its

lack of esthetic appeal. For cut-and-cover tunnel construction, it may have some excellent applications since it can be concealed and there is plenty of room for structural depth. Some of these possible applications are shown in subsequent sections of this report.

4. Long-span corrugated steel arch sections are a recent development in highway construction (Fig. 4). To date, their use as transportation tunnels has been confined to culvert type applications in rural areas, usually carrying a secondary road under a primary highway. These applications involve relatively shallow cuts and short lengths. Their stability is dependent on the interaction of soil and steel. They are sensitive to unbalanced loading, so backfilling precautions are necessary, and certain minimum cover depths are required. As more experience in the use of these

Fig. 4 Long span corrugated steel arch. (Courtesy Armco Steel Corporation)

sections is gained, extension of their use to urban areas may prove to be an economically viable solution.

Any exposed steel member used for final tunnel construction is subject to corrosion and fire damage. Adequate protection and/or maintenance programs must be considered.

B. PREFABRICATED CONCRETE SECTIONS

Designers and builders of reinforced concrete structures have long looked at precasting as a means of reducing the costs of expensive in-place forming. It has always seemed rather foolish, without a detailed cost analysis, to build a structure out of wood (the form), pour it full of concrete and then tear it down. However, the additional equipment required to lift the heavy precast units into place, the added space requirements for job-site precasting and the technical problems of joining the pieces together have proved to more than offset the savings in forming costs, except for very limited applications.

It was not until the development of linear pretensioning which provided the additional cost savings of less required material, that precasting of large structural elements became economically feasible to any great extent. This development took place in the early 1950's. Since that time, the precast, prestressed industry has grown at a significantly higher rate than construction in general. Even so, at present, precast concrete construction probably accounts for less than ten percent of total volume of construction in buildings and bridges.

The use of precast concrete elements for cut-and-cover tunneling has been limited, especially in the United States. A few U. S. contractors have experimented with precast slabs for temporary decking in place of the conventional timber. These experiments have usually taken place during times when timber prices (which have been quite cyclic the last several years) were high. There appeared to be very little advantage to using concrete, and several disadvantages including lack of flexibility and no salvage value.

A cut-and-cover tunnel section of Interstate 95 adjacent to the Delaware River in Philadelphia uses precast, prestressed concrete box beams for the roof members.

In the Edmonton, Alberta subway system, the Jasper Street station was constructed using precast, prestressed channels placed on load-bearing cast-in-place tangent piles (Fig. 5). The tangent piles served as both the temporary earth support and permanent tunnel walls. All excavation was performed under the prefabricated roof, after the street above was restored. The space between the street and the tunnel roof is left open for use as an underground pedestrian walkway.

The Yerba Buena Island structure of the San Francisco-Oakland Bay Bridge Reconstruction uses precast channel slabs post-tensioned together for the tunnel roof (Fig. 6). The tendons are tensioned by jacking the precast units apart from the center.

Outside of North America, precast and prestressed members have been used for both the wall elements and the horizontal elements in cut-and-cover tunnels. Some examples are the Moscow Metro, Stuttgart subway, pedestrian tunnels in London, A13 Motorway extension in Paris, and a transportation tunnel in Kyushu, Japan.

Two French companies, Soletanche and Bachy, market proprietary precast systems. The Soletanche Panasol systems include both continuous

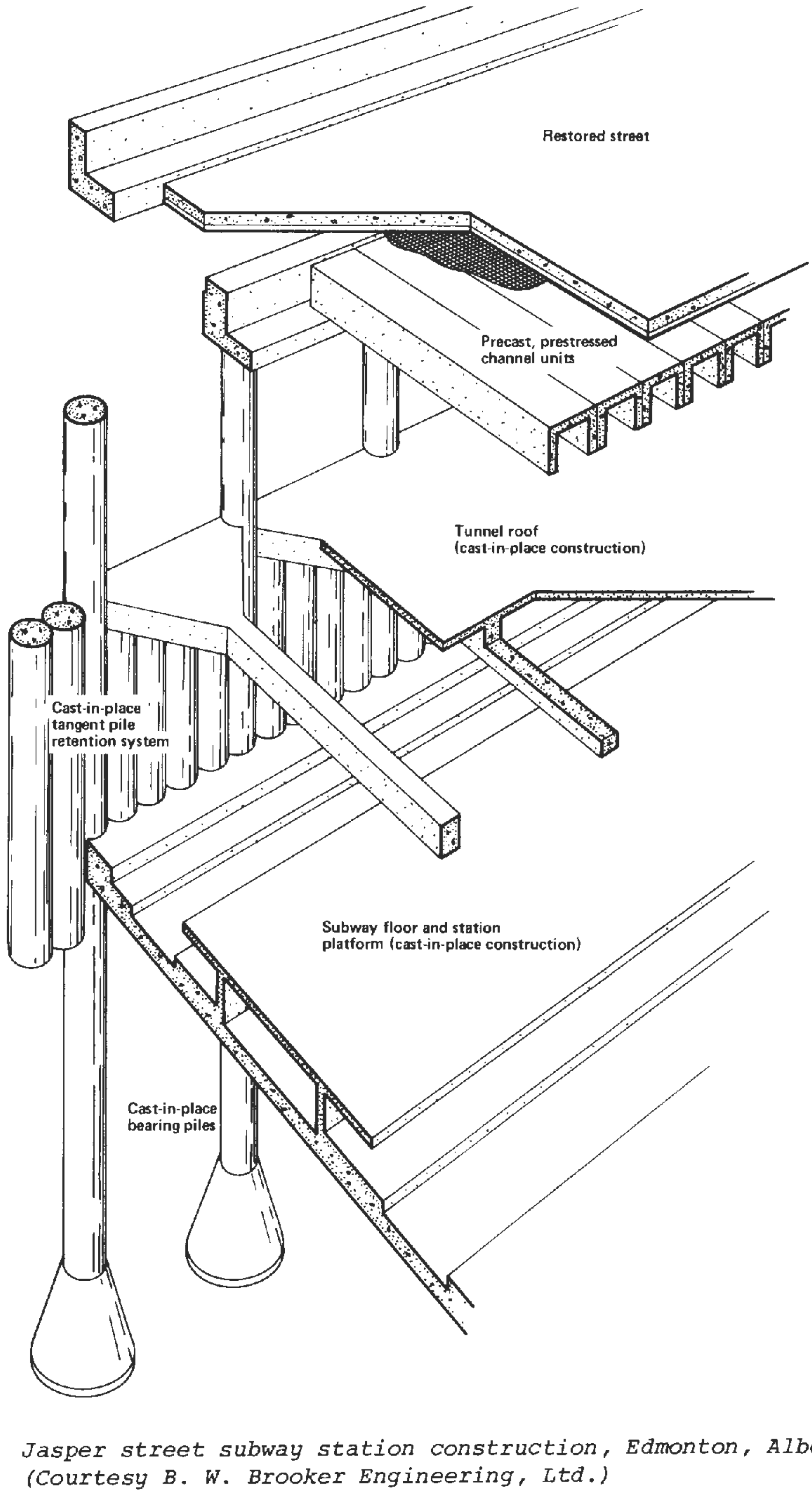


Fig. 5 Jasper street subway station construction, Edmonton, Alberta. (Courtesy B. W. Brooker Engineering, Ltd.)

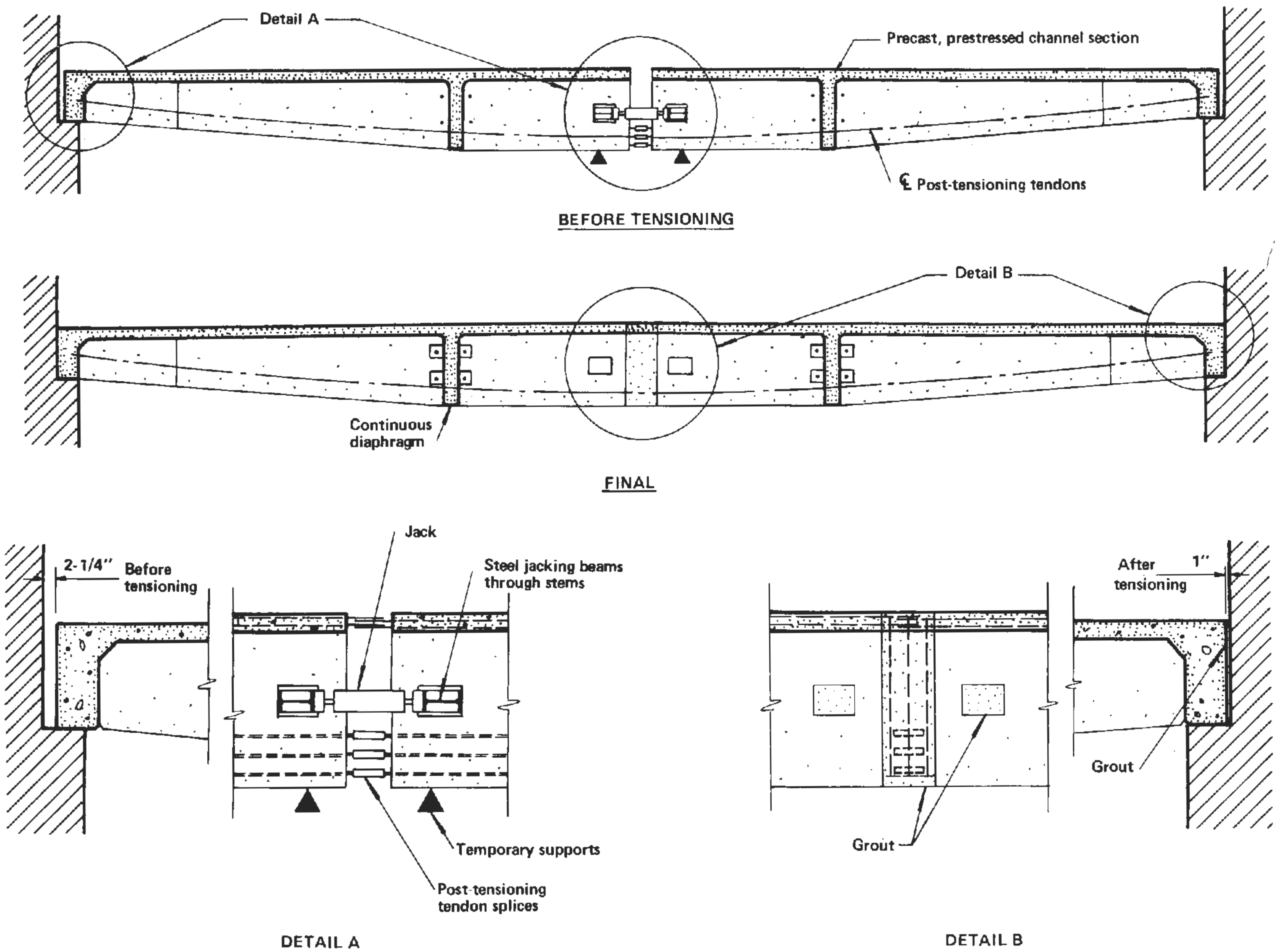


Fig. 6 Yerba Buena Island tunnel structure

reinforced concrete sheet pile sections, and a T-beam and precast panel system (Fig. 7). The Soletanche company has marketing and construction divisions in North America. The company is involved in slurry trench, cast-in-place diaphragm wall construction and grouting, as well as prefabricated panel walls. The Bachy "Prefasil" system is structurally similar to the Panasol sheet pile system, except for the method of connecting the adjacent units.

1. Slurry wall construction:

One of the key elements in the use of prefabricated members for the walls of cut-and-cover tunnels is the slurry wall or diaphragm wall method of construction. This method of constructing

Fig. 7 Installing precast concrete wall panel in slurry trench. (Courtesy Soletanche and Rodio, Inc.)

walls below ground is only about 25 years old, but is rapidly gaining favor for basement construction of large buildings, and has had some use in this country in subway construction. The first slurry walls were constructed in the U. S. just ten years ago, but in Europe and Japan, the slurry wall has reached the status of being considered the "normal" way to build underground walls.

In most applications of diaphragm wall construction, a continuous trench is excavated in the presence of a fluid, usually a clay slurry. The walls of the trench are prevented from collapsing by the hydrostatic pressure of the fluid (specific gravity only slightly greater than water) and the presence of an impervious cake which forms on the face of the excavation. The most commonly used fluid is a bentonite slurry. A properly proportioned bentonite slurry will penetrate into surrounding permeable soils to form a permanent watertight barrier.

A permanent concrete wall is formed by either (1) placing reinforcing cages in the trench and then placing the concrete by tremie or (2) suspending precast concrete units in the trench. When the latter method is used, portland cement is usually mixed with the bentonite to form a setting slurry which can transmit the lateral loads to the wall and vertical loads to soil at the bottom of the trench. (In some methods, the constructor displaces the original bentonite slurry by a setting material or a grout.)

In "conventional" cut-and-cover tunnel construction, a temporary ground support system is constructed usually using either steel sheet piling or steel soldier piles and timber lagging, and the permanent structure is built within this temporary support system. The slurry wall constructed by either of the methods mentioned above has several advantages over the conventional methods:

a. The concrete wall can usually be designed to act as both the temporary ground support and the permanent structure.

b. The slurry wall can be built in almost every type of soil and ground water condition, with the exception of solid rock. (Rock excavation for slurry trenches is possible, and has been done, but it is very slow and expensive.

c. The wall thus formed is very rigid, and with proper design of struts and tiebacks, ground movements behind the wall are almost negligible. In urban areas, this can eliminate the need for expensive underpinning of adjacent structures.

2. Plant vs. site prefabrication of concrete members.

Job-site precasting has been attempted by many general contractors on building and bridge projects. It has rarely proven economical when compared with plant-produced products. The primary reasons for this are:

a. In the United States, manufacturing plant labor rates are typically about one-half of field labor rates.

b. Construction unions at a job site demand a strict separation of work according to craft skills. This seriously restricts efficient utilization of manpower. Central precasting plants when unionized normally operate with only one trade jurisdiction.

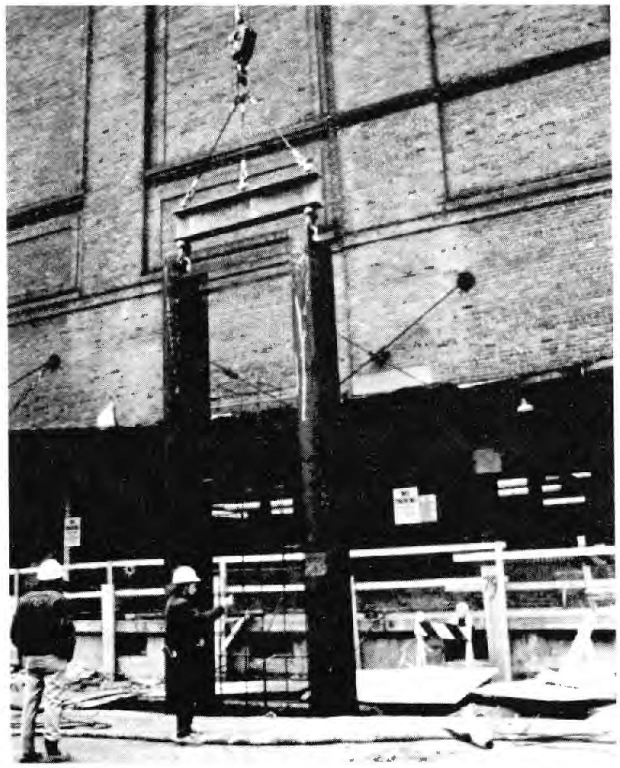


Fig. 8 Slurry wall construction near 8-story building on Washington, D. C. Metro subway system. The building footings were within seven feet of the slurry wall. No underpinning of the structure was necessary and no noticeable damage occurred. (Courtesy ICOS Corporation of America)

c. Most job sites lack adequate space to manufacture and store the products. This is especially true of tunneling projects in urban areas.

d. Accelerated curing procedures are difficult to install and operate at a job site. Accelerated curing allows a much faster turn-over of the precasting beds, hence more efficient use of labor and lower investment in forms. The same is true of much other equipment used in a modern precasting plant. Also, this equipment is often expensive, and cannot be economically written off on one project, unless the job is quite large.

Conversely, plant prefabrication may impose some constraints on the members which can be used, because of weight, length, and width restrictions for over-the-road hauling. In most states, these restrictions can be waived, within limits, through special permits.

3. Standard shapes of precast, prestressed concrete elements.

For large projects, special shapes can be economically produced in a precasting plant. Since cut-and-cover tunnel construction is substantially different in many respects from buildings and bridges, special shapes may often be necessary, and the designer should not be limited to the standard sections commonly available. However, as a reference guide, these standard shapes are shown here.

a. Bridge sections: The most commonly used bridge sections are those developed by a joint committee of the American Association of State Highway Officials and the Prestressed Concrete Institute in the 1950's. These AASHO-PCI sections are shown as Figs. 9 and 10. Many states have developed their own standard sections, which are typically more efficient than the AASHO-PCI sections. Some of these are shown in Fig. 11. Other sections are produced by manufacturers of precast, prestressed concrete products for use on bridges. Many of these sections are shown in the recently published PCI "Short Span Bridge" Manual. Excerpts from that publication are included in Fig. 12.

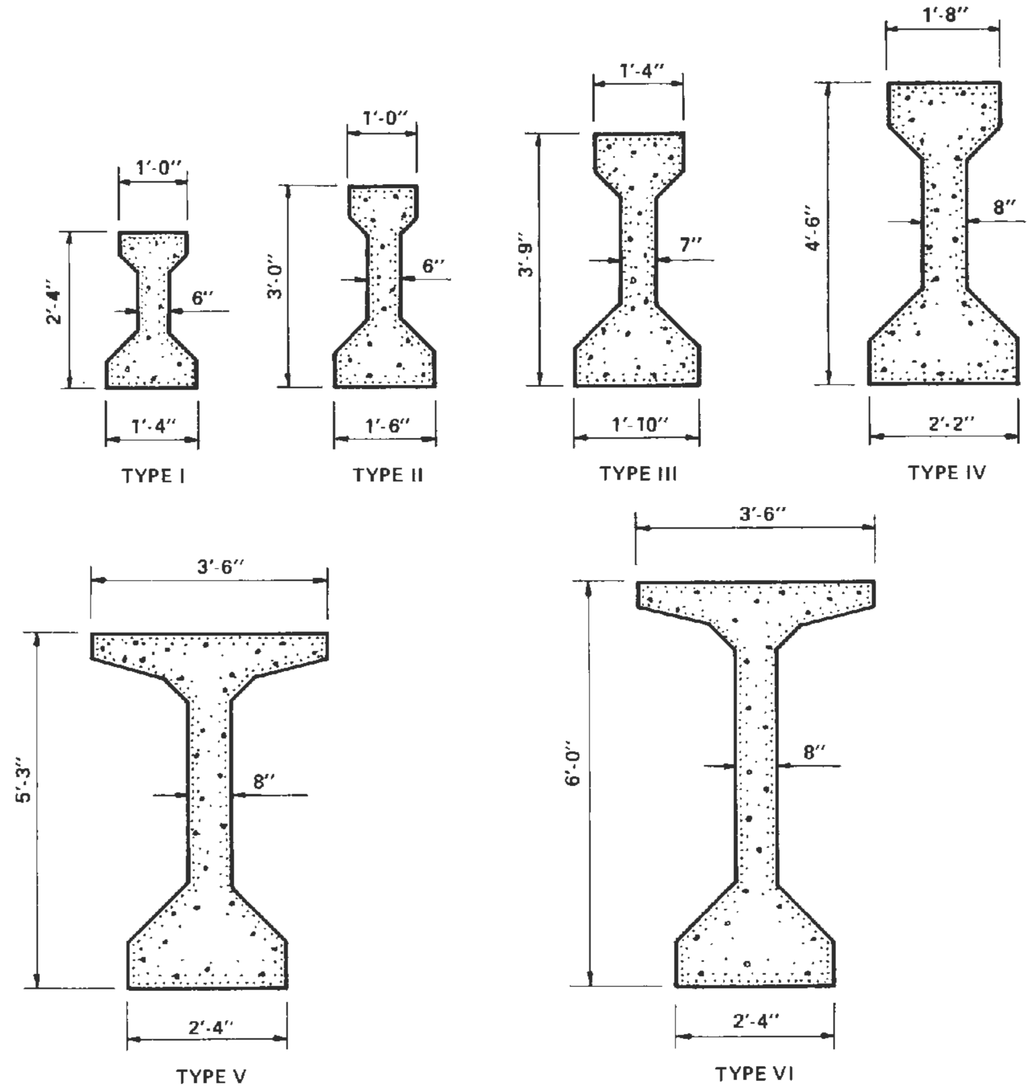


Fig. 9 AASHO-PCI standard bridge girders

b. <u>Building sections.</u> During the development of the prestressing industry, many different building sections were produced by different precasting plants across the country. In 1970, the Committee on Standardization of the Prestressed Concrete Institute selected the most commonly used and most efficient of these sections for publication in the PCI Design Handbook

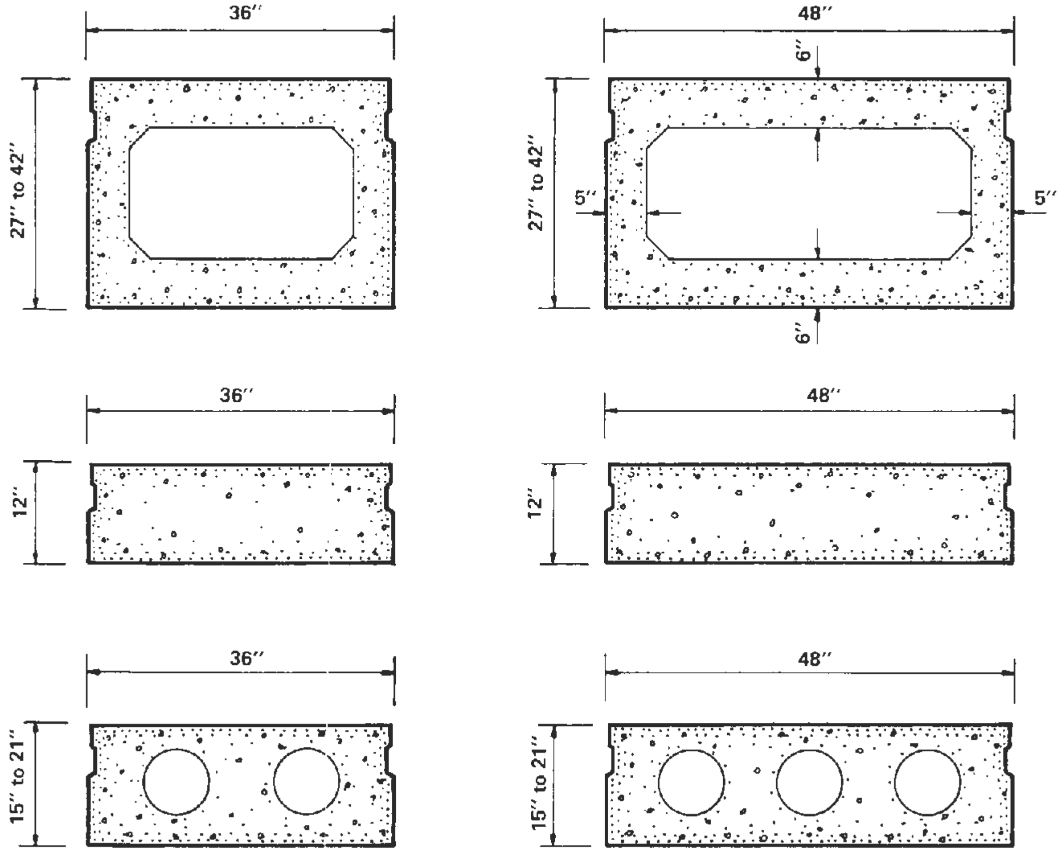


Fig. 10 AASHO-PCI standard box beams and bridge slabs.

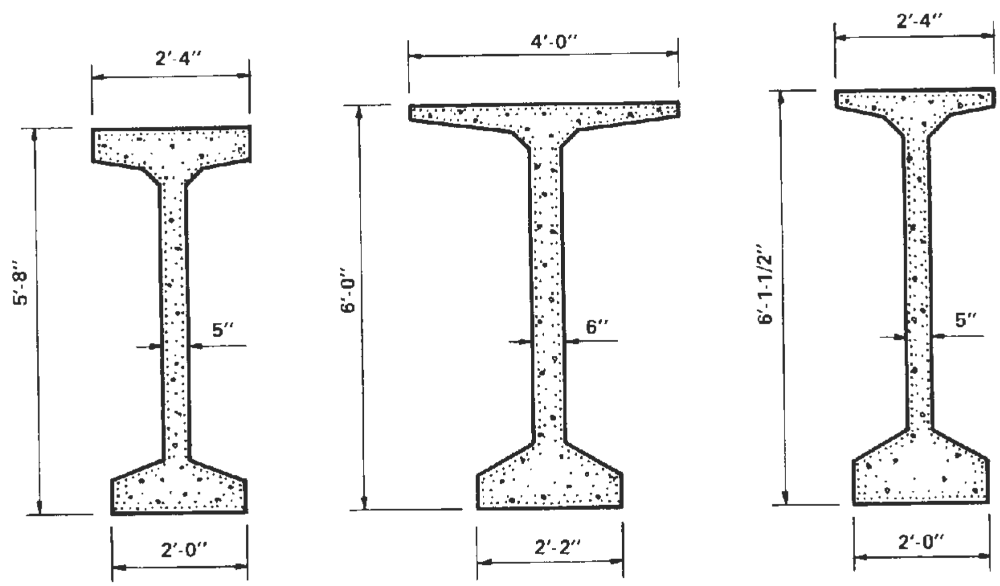


Fig. 11 Typical state standard bridge girders.

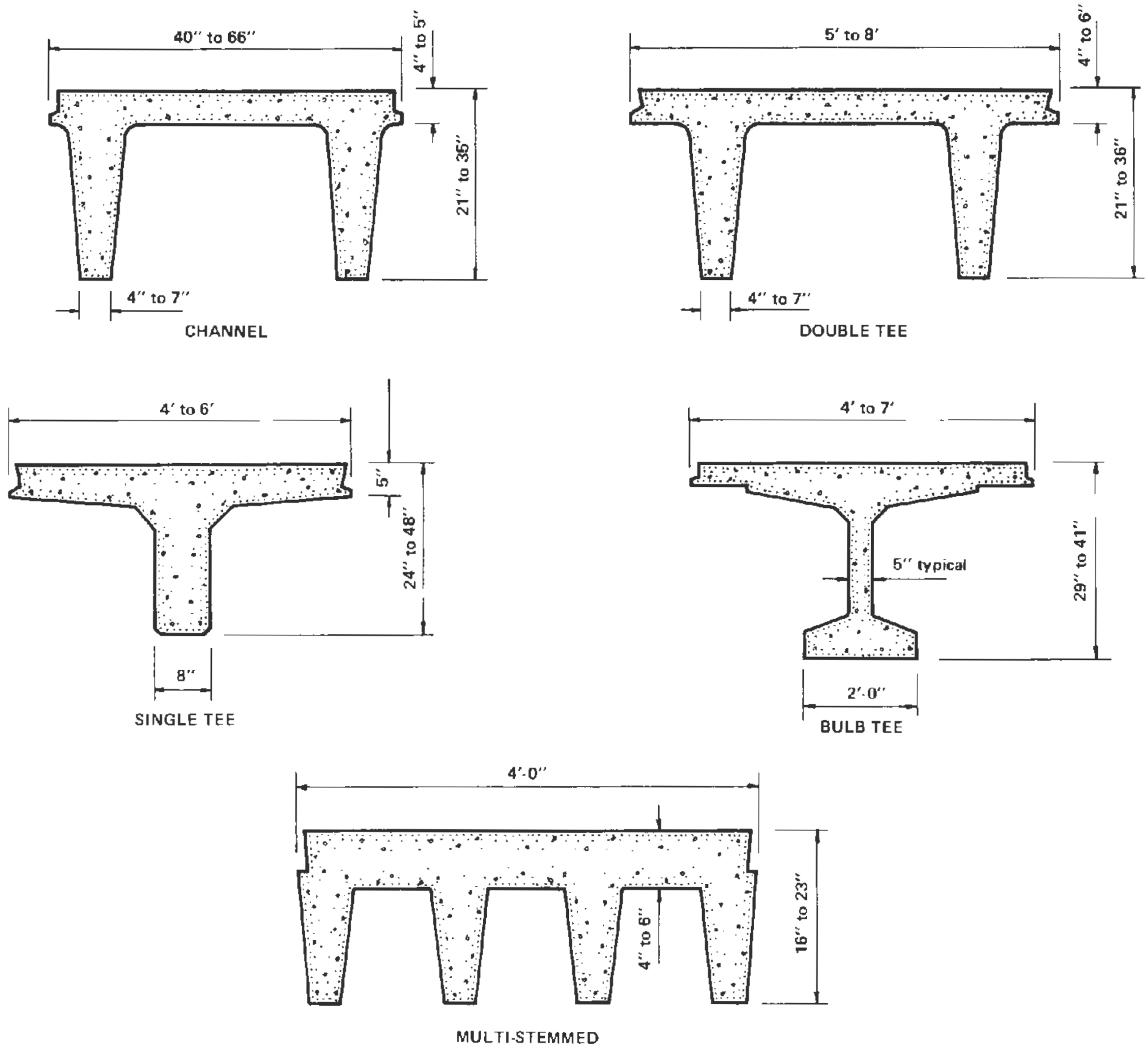


Fig. 12 Typical precast, prestressed concrete bridge sections produced by U. S. and Canadian manufacturers. (Courtesy Prestressed Concrete Institute)

as "industry standards". While the industry is still not standardized, the Handbook sections, with some minor variations are the most readily available in most parts of the U. S. Some of these sections are shown in Fig. 13.

c. <u>Piling:</u> These sections, shown in Fig. 14, are manufactured in most plants in coastal states, and in some inland states

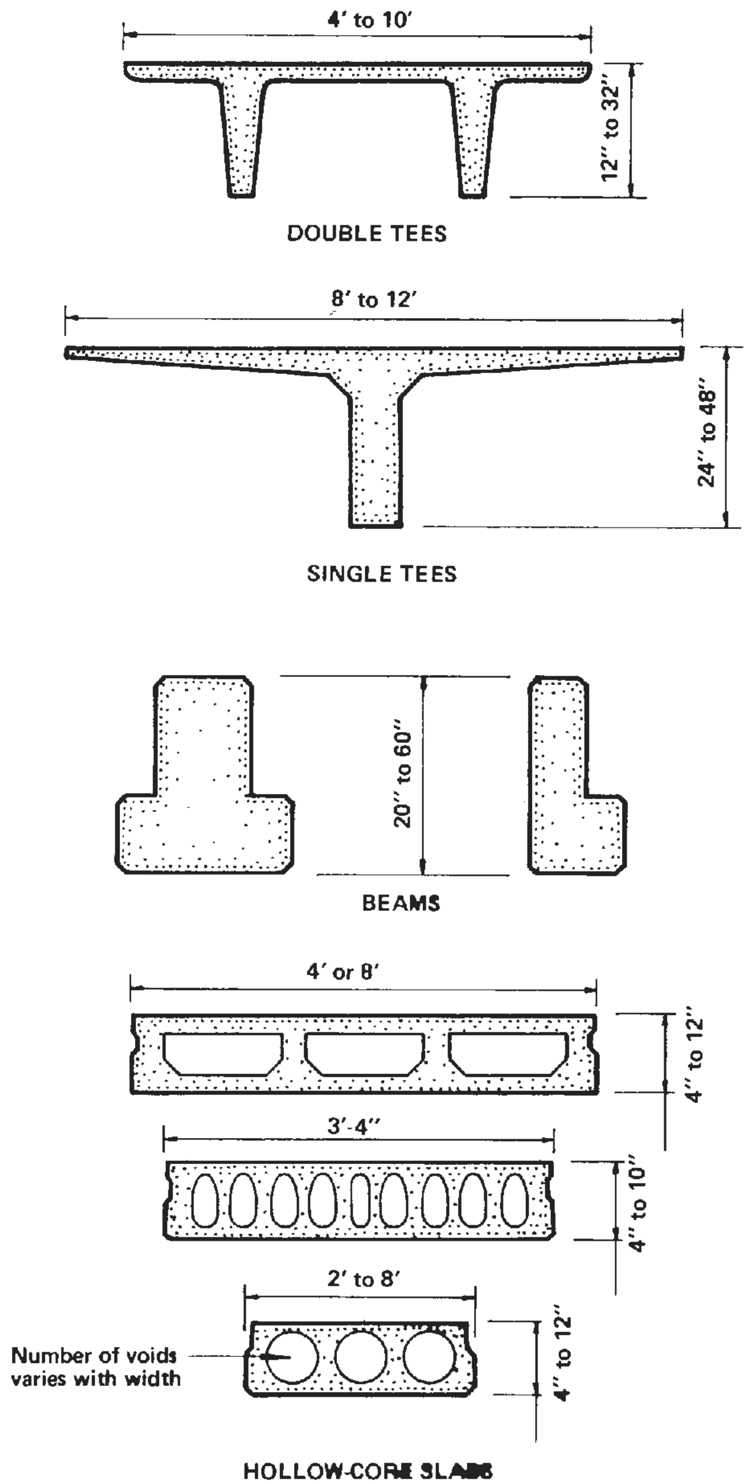


Fig. 13 Typical precast, prestressed concrete sections used in buildings. (Courtesy Prestressed Concrete Institute)

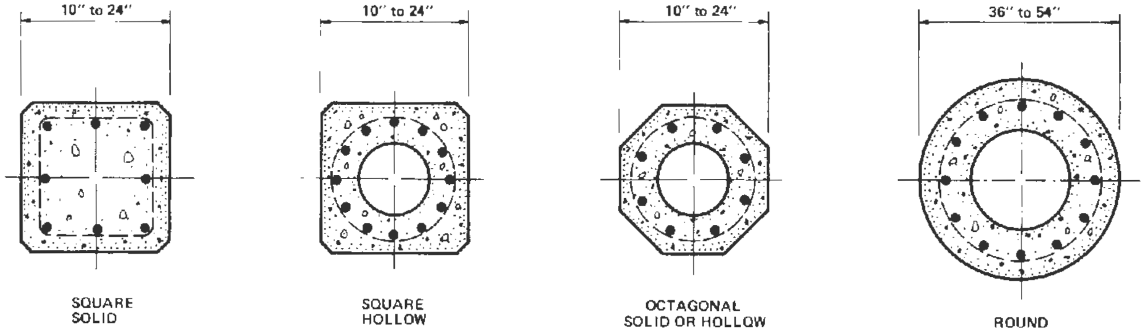


Fig. 14 Typical precast, prestressed concrete piling

where highway departments have adopted them for bridge foundations. Typically, only a few of these sections will be available from any one plant.

d. Columns: Until the adoption of ACI 318-71, most precast columns were designed as reinforced concrete columns. The 1971 edition of the ACI Code waives the minimum reinforcing requirements of columns when they are prestressed. Prestressed concrete columns rely almost completely on the strength of the concrete to carry the compressive loads, with the prestressing being effective in resisting moments. Precast, prestressed columns are usually much less expensive than precast, reinforced columns, especially when advantage is taken of the plant casting environment to achieve concrete strengths of 7000 to 10,000 psi.

C. PREFABRICATED TUBES

Totally prefabricated tubes have been frequently used for transportation tunnels under water. These tube sections are usually built in drydock or on special barges, barged to their location and then sunk to a previously prepared bedding on the floor of the waterway.

Extension of this method to underground tunnels has been employed at two sites in the Netherlands, where the high water table and poor soils enabled similar construction methods.

One project was in Rotterdam, through the old section of the city, where foundations of adjoining buildings were very precarious. The following sequence of construction was followed:

1. A sheet pile wall was driven on each side.
2. Longitudinal wales and transverse struts were placed. Rails were installed on each wale.
3. Soil was excavated by a hydraulic dredge traveling on a rail mounted carriage. The excavation was now full of water, forming a canal.
4. Select rock backfill was placed on the bottom and screeded to exact grade by a screed running on the same rails.
5. Precast tunnel segments were cast at one end of the canal, and transported on the rails to their final location, and assembled in place.
6. Finally, the canal was backfilled.

In Amsterdam, the tube sections averaging 10 meters high and 40 meters long were built on the ground directly above their final position. The units are constructed with a sort of false bottom. Each rests on cutting edges that extend below the floor and create a hollow area beneath. This is shown in Fig. 15. Within the hollow area are powerful water jets to wash away the ground on which the unit is resting, while high pressure air

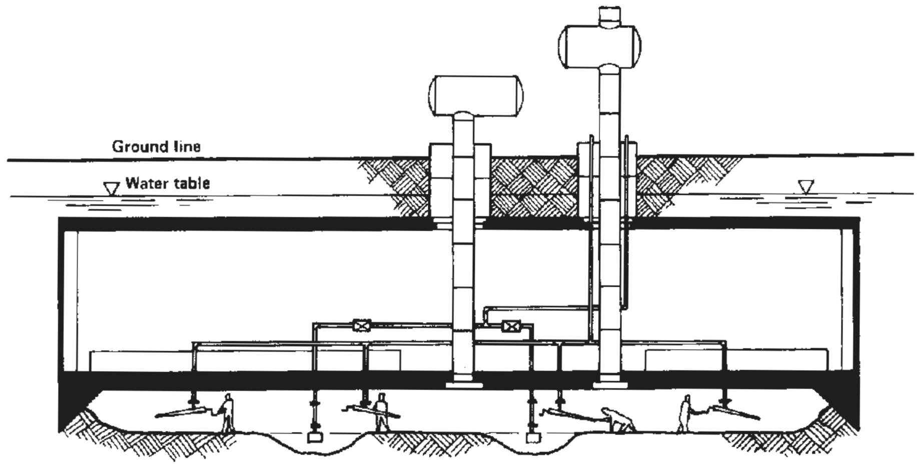


Fig. 15 *Longitudinal section (schematic) of "sunken tube" section on the Amsterdam Metro. (Courtesy Civil Engineering magazine)*

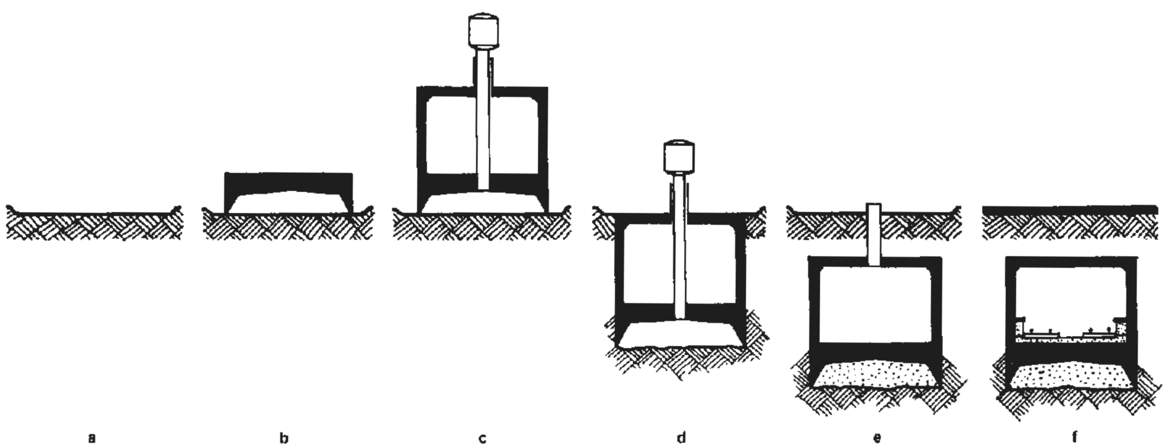


Fig. 16 *Construction procedures of the "sunken tube" section on the Amsterdam Metro. (Courtesy Civil Engineering magazine.)*

is pumped into this chamber to keep out ground water.

The construction procedure is illustrated in Fig. 16.

1. (Fig. 16a) Ground was excavated to the water table, which was approximately 3 ft below the surface.
2. (Fig. 16b) The caisson floor and cutter shield were cast.
3. (Fig. 16c) The caisson walls and roof were cast, and the shafts, airlocks, and equipment installed.
4. (Fig. 16d) Soil below the caisson was excavated as shown in Fig. 15, allowing the unit to settle into position.
5. (Fig. 16e) The work chamber was filled with concrete for ballast and airlocks were removed.
6. (Fig. 16f) Finally, the adjoining sections were connected.

These construction methods might have application in a few cities in the United States, such as those on the Gulf Coast or Atlantic Seaboard. Since the applicability is so limited, the use of totally prefabricated tubes was given very little consideration in this study. The use of prefabricated designs in which the vertical and horizontal members were considered separate elements appears to be much more applicable for the vast majority of sites.

IV. METHODS OF USING PREFABRICATED MEMBERS IN CUT-AND-COVER TUNNELS

Figs. 17 through 28 illustrate some of the ways that prefabricated structural members can be used in cut-and-cover tunnel construction. It should be recognized that any of the members shown could be easily used with other types of members. For example, prefabricated wall elements could be used with cast-in-place roofs or prefabricated roof members could be used with cast-in-place wall construction.

Fig. 20 shows a continuous bearing wall of precast prestressed concrete units of design similar to that shown in Fig. 17. The wall units are placed in a slurry trench prior to excavation. The roof units in this case are concrete box beams designed by standard methods and similar in section to those shown in Fig. 10. For fill depth greater than about 8 or 10 ft, these box beams must be extremely deep and heavy, but not necessarily outside the feasible range.

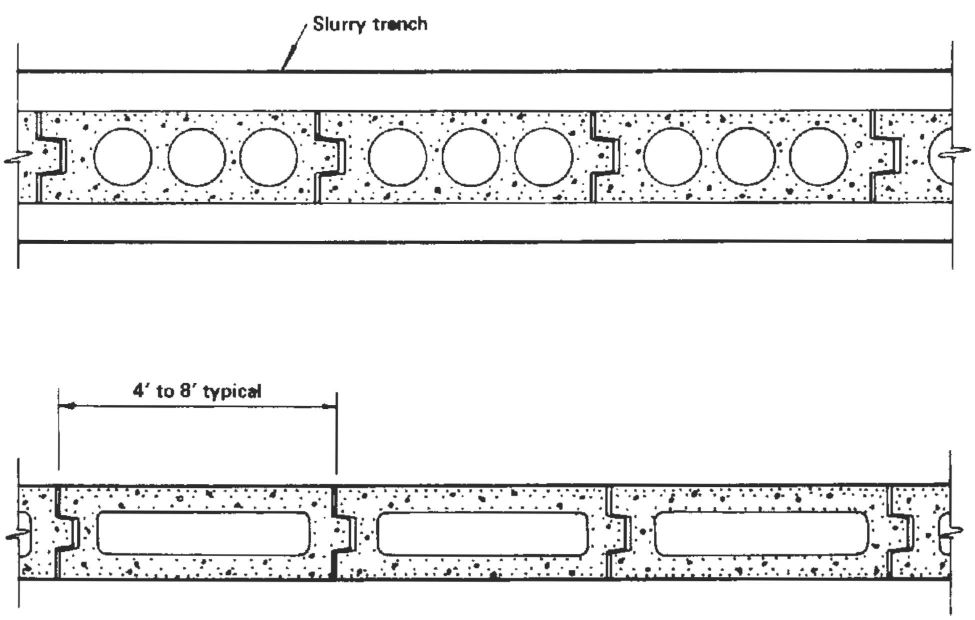


Fig. 17 Continuous load-bearing sheet pile walls.

This method would require temporary strutting and street decking (if necessary) much the same as used in conventional construction.

Fig. 21 is similar to 20 in general concept except that in this case the roof member is a precast concrete ribbed arch section. The two-hinged arch is an exceptionally efficient section for this application. The usual objection to the two-hinged arch in building and bridge construction is that large abutments or tension ties are required to resist the thrusts generated at the spring line of the arch. For tunnel applications such as illustrated here the earth behind the tunnel walls create a natural resistance to this thrust. Again, temporary struts and decking would normally be required if this method was used in congested urban areas.

Fig. 22 illustrates the concept of the utility corridor as discussed in Section II of this report. By eliminating the backfill over the roof beam the structural members directly over the tunnel can be greatly reduced in size. However, a secondary roof is required to carry the street traffic. This upper level roof is shown here as concrete box beams, but

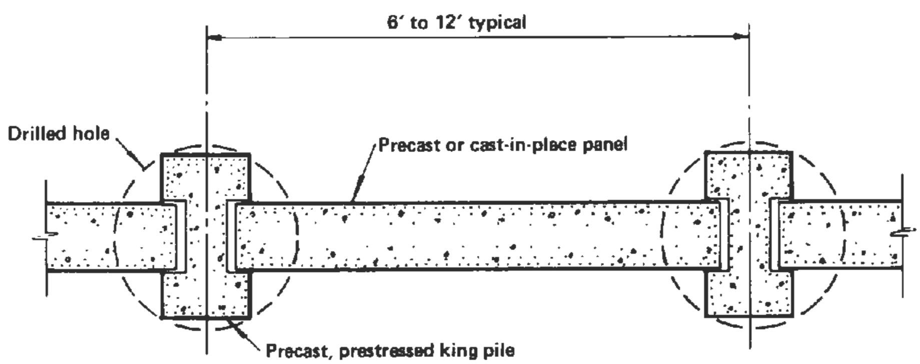


Fig. 18 King pile wall system.

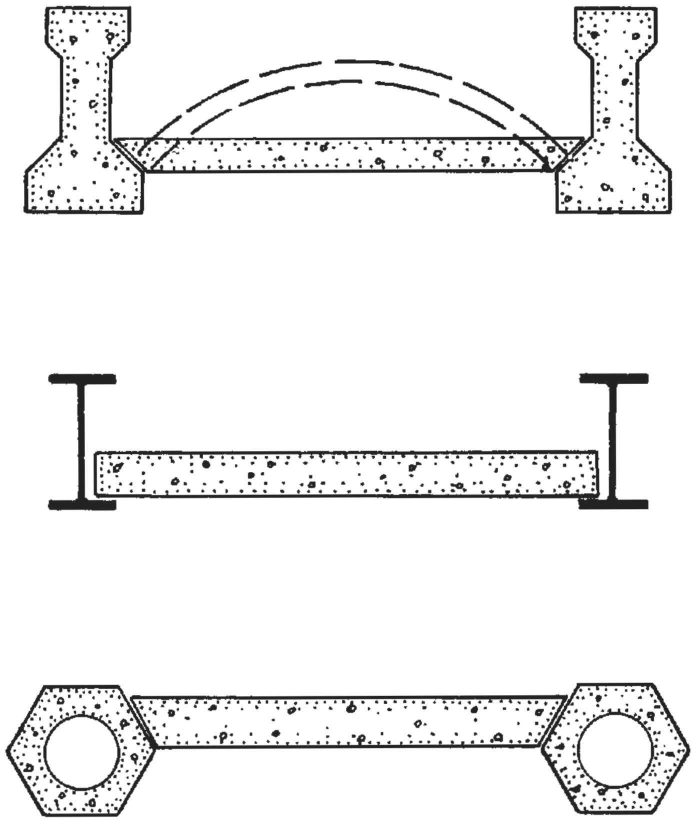


Fig. 19 Variations of king pile wall system.

could be a wide variety of structural members as illustrated in Part III of this report. This upper roof member could eliminate the need for temporary decking and can be designed to serve as a strut during construction. It is anticipated that much of the excavation and placement of the lower horizontal members would be done under cover after the upper members are placed.

Figs. 23 and 24 illustrate how concepts similar to those shown for the single span, single level condition could be adapted for multi-span or multi-level type tunnels.

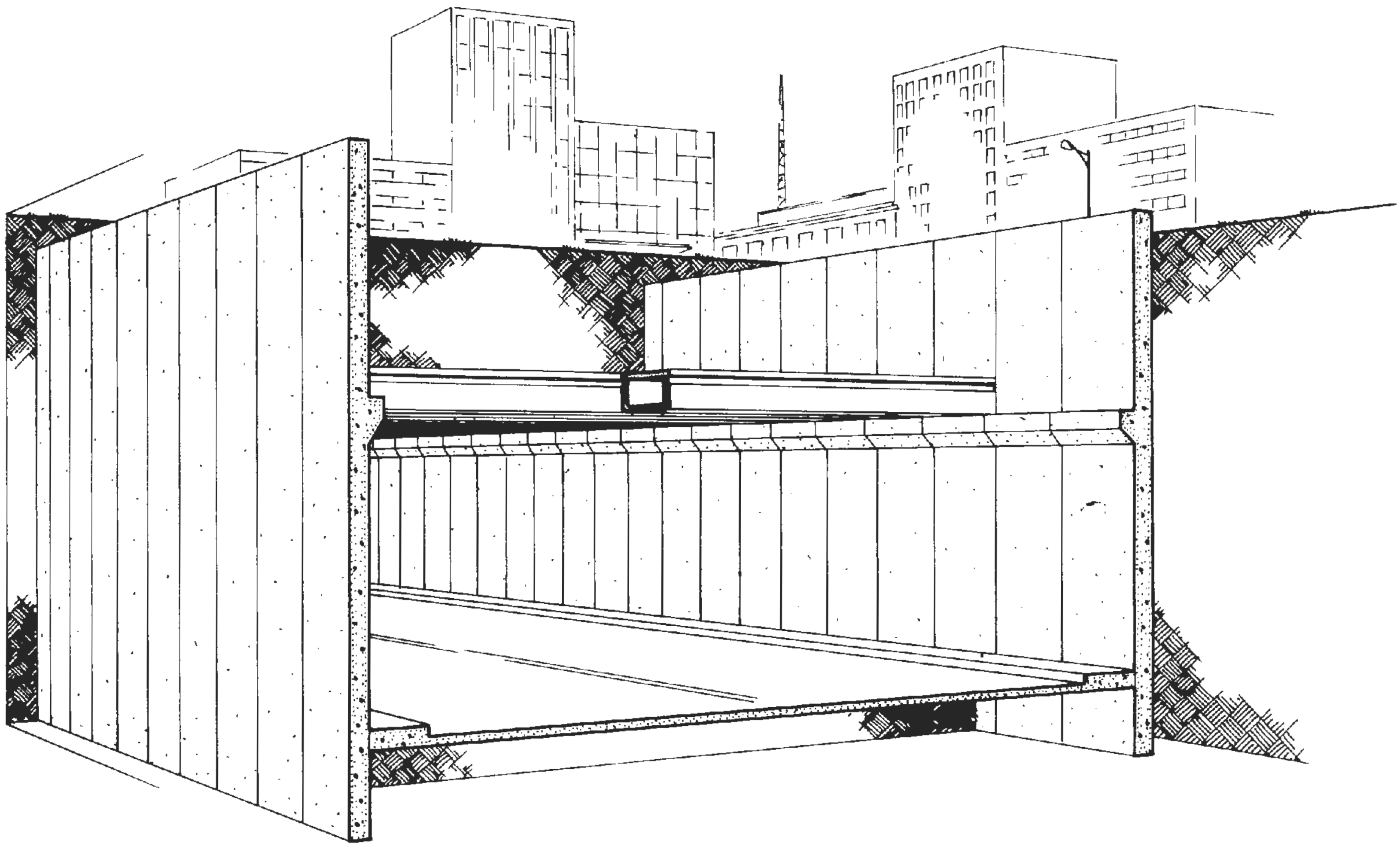


Fig. 20 Continuous precast concrete bearing walls with box girder tunnel roof.

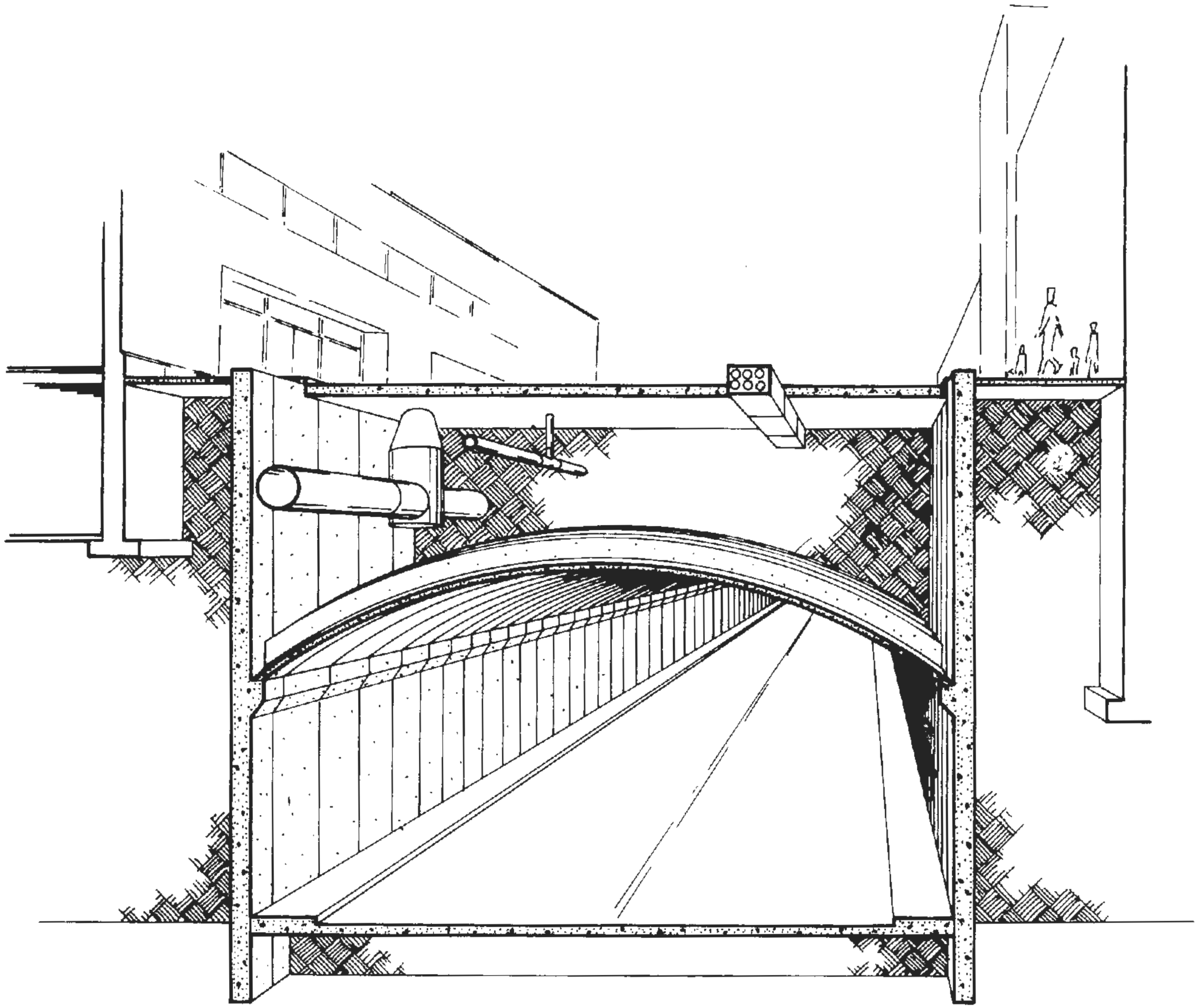


Fig. 21 Precast concrete ribbed arches used as tunnel roof.

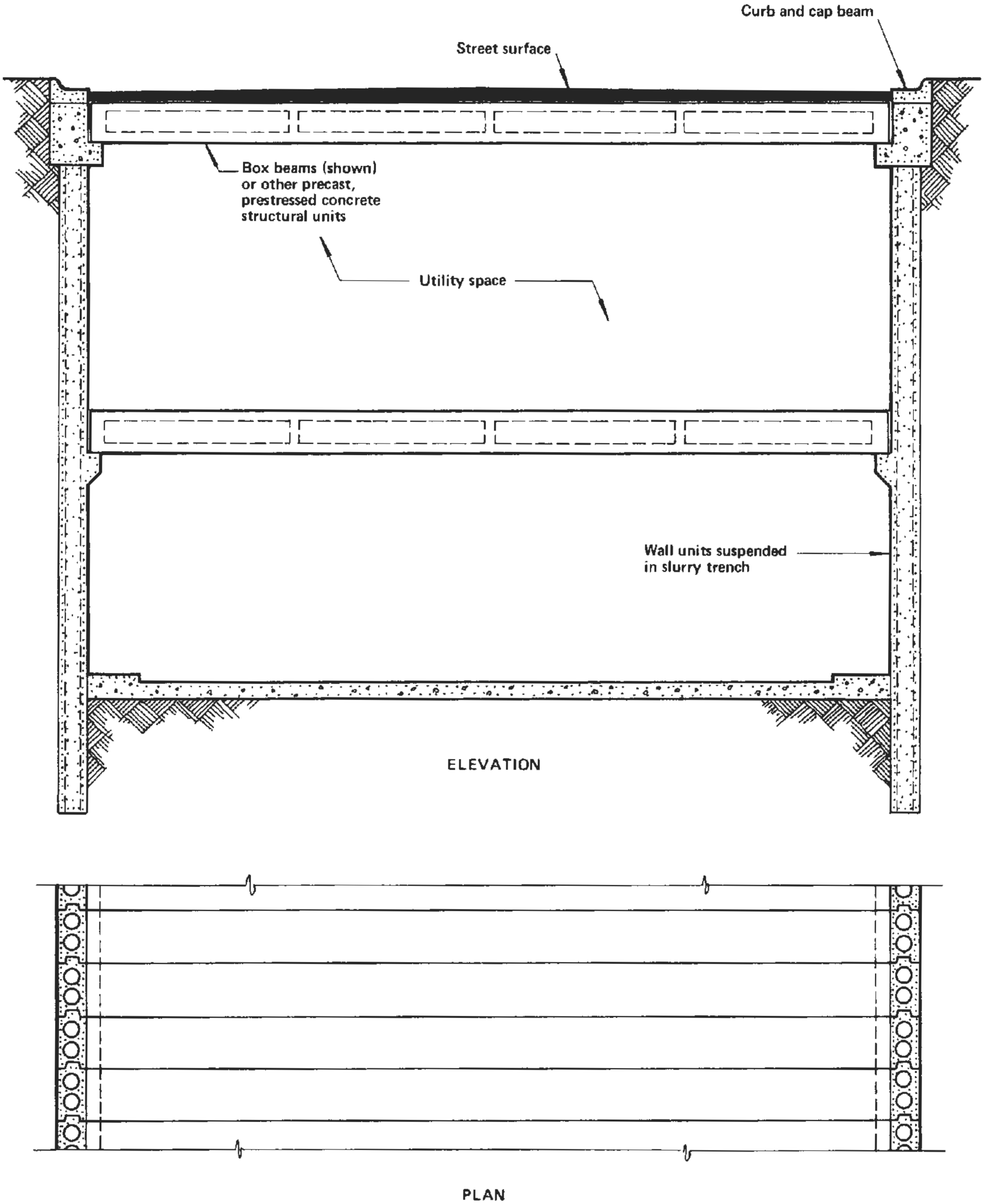


Fig. 22 Schematic of system using space above tunnel roof as utility corridor.

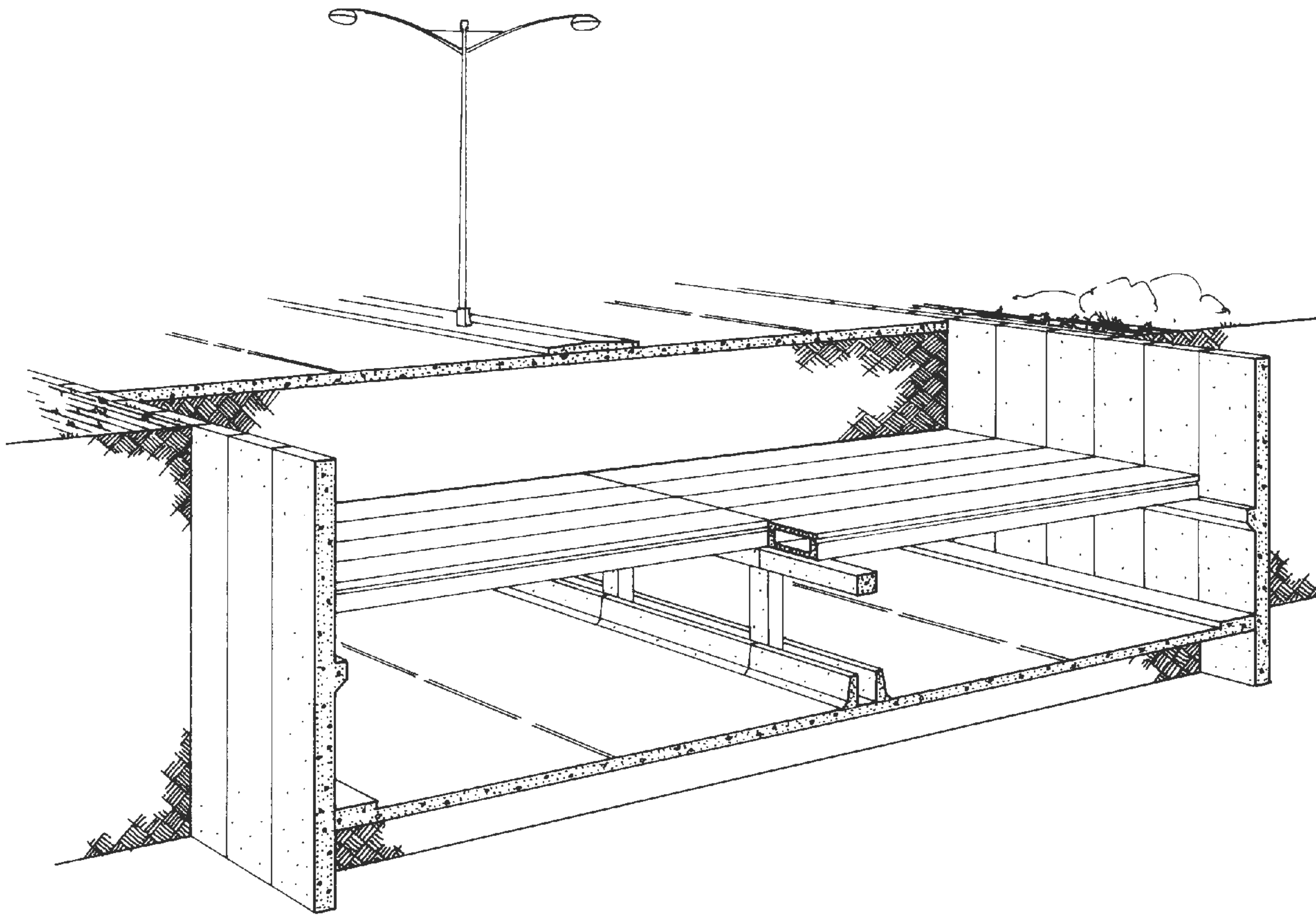


Fig. 23 Multi-span variation.

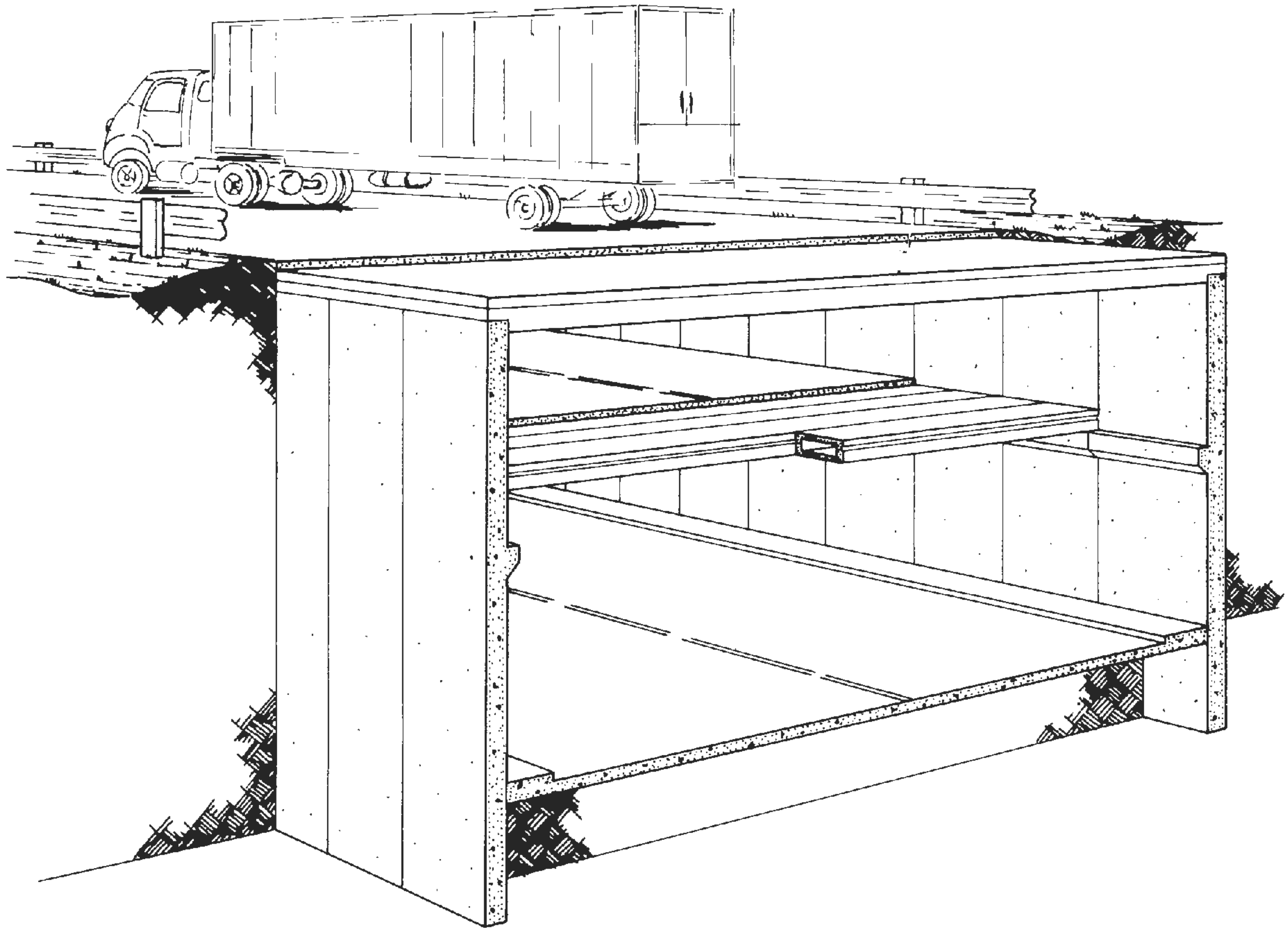


Fig. 24 Two-level highway tunnel.

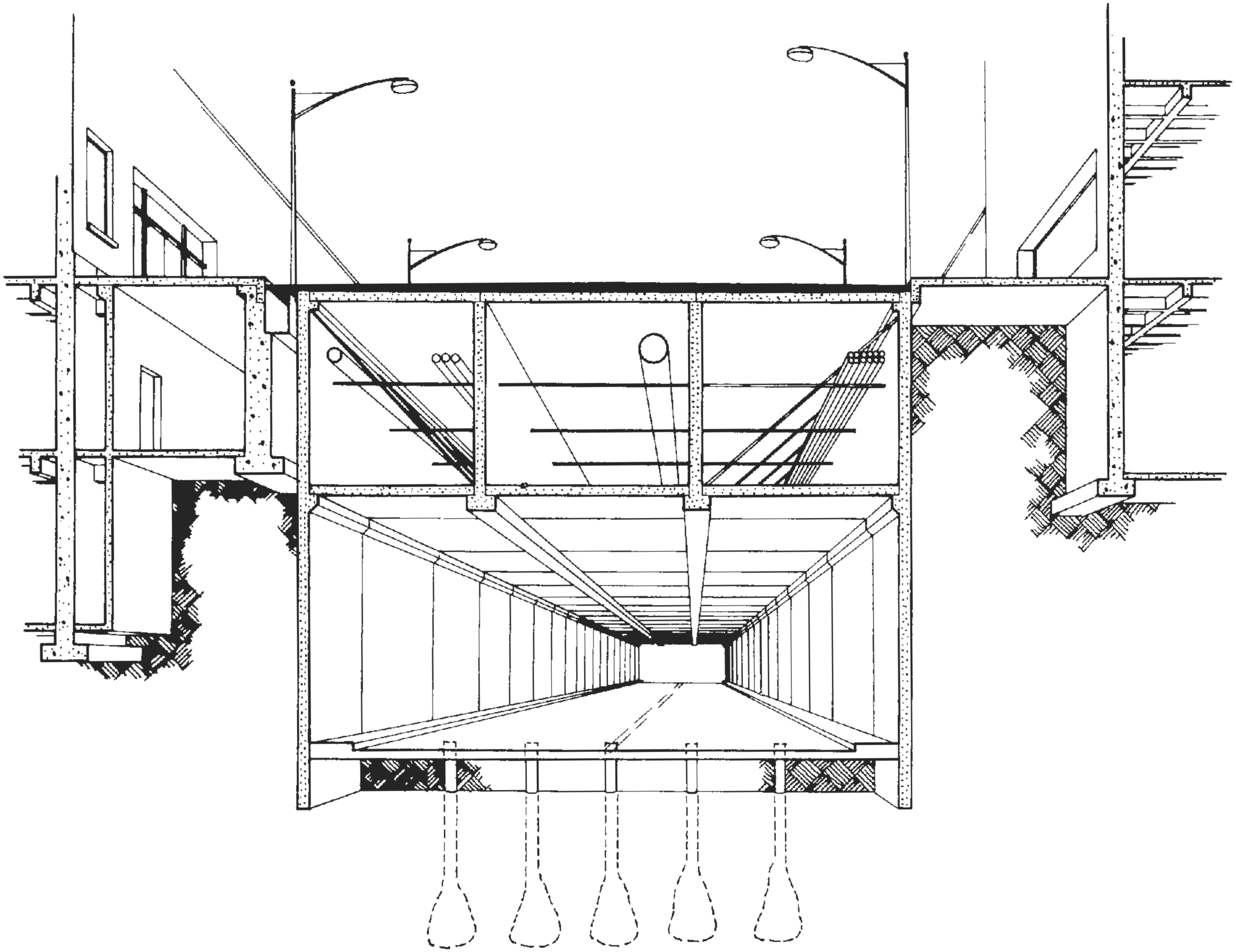


Fig. 25 Two structural levels tied together to act as a modified Vierendeel truss.

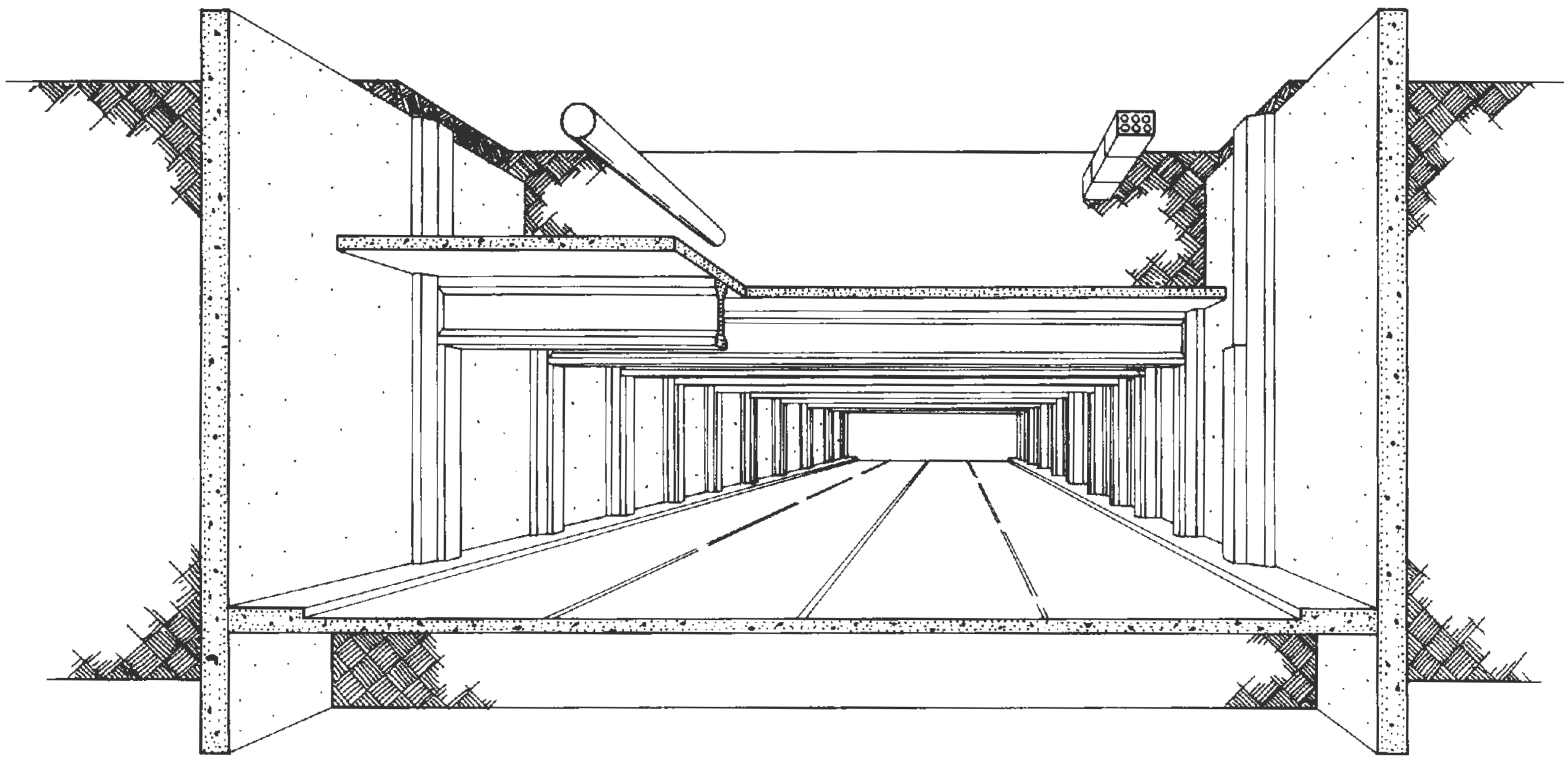


Fig. 26 King pile wall system with spaced girders as tunnel roof.

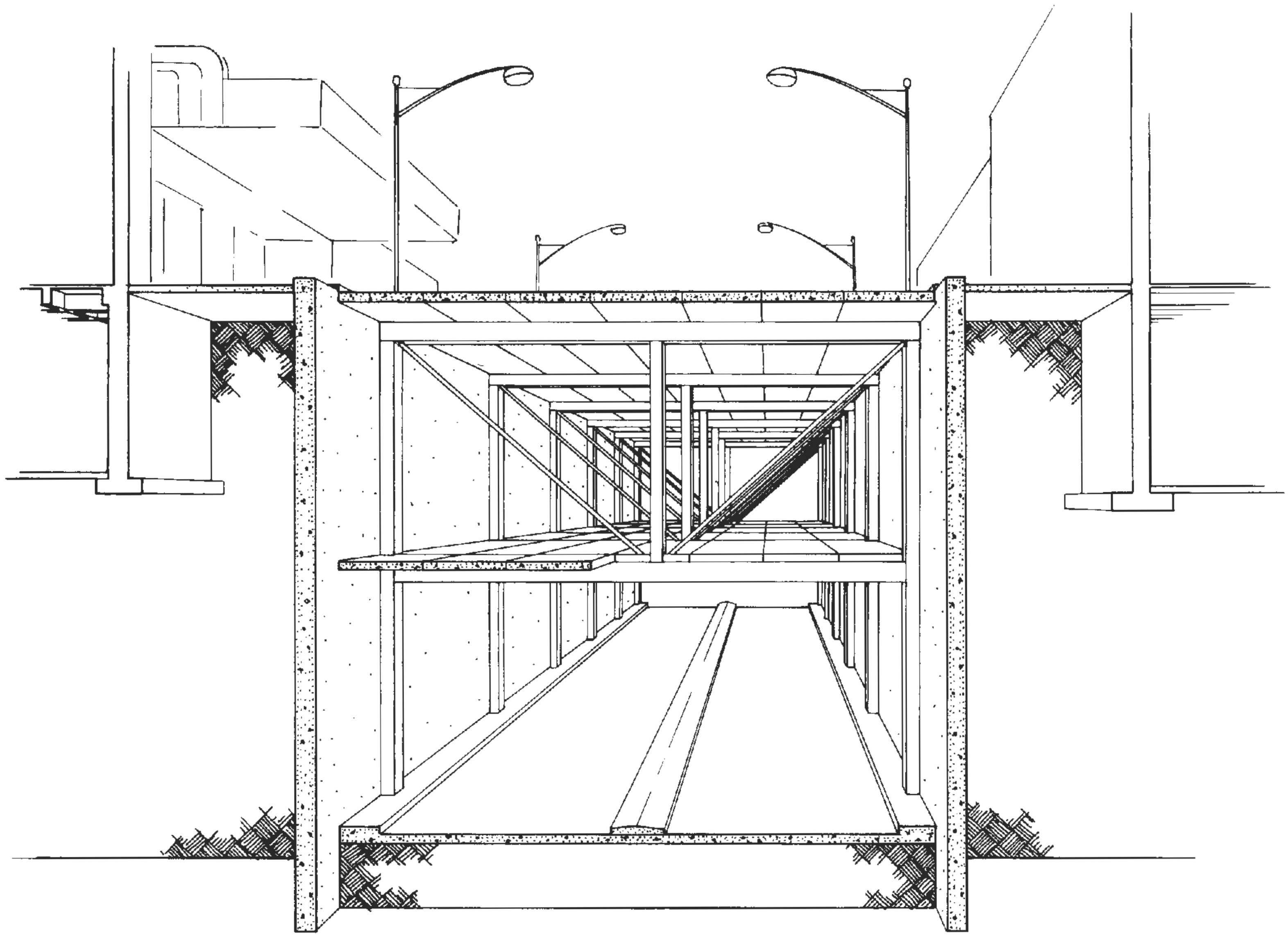


Fig. 27 Space between tunnel roof and street used for a concrete truss. Horizontal support members at tunnel roof and street levels form chords of the truss.

Fig. 25 also uses the continuous sheet pile bearing wall similar to those shown in the previous illustrations. The horizontal structural system is unique in that the lower roof and the upper roof are tied together and designed to act as a unit, in a modified Vierendeel truss. This construction offers some attractive possibilities for stage construction that would allow the street to remain partially open at all times during construction. A step by step procedure is described in Section XI.

Fig. 26 illustrates the use of a king pile wall system of construction similar to that shown in Fig. 18. In this figure the pile section shown is hexagonal but could be several different shapes, as shown in Figs. 18 and 19. The piles are spaced at approximately 8 to 10 ft on center and are placed in drilled holes using standard, readily available caisson drilling equipment. It is suggested that a setting slurry similar to the types discussed in Section VIII of this report could be used to facilitate the drilling of the hole if soil conditions were such that it was necessary. This setting slurry would also serve to maintain alignment of the piles during excavation. Precast concrete panels are placed between the piles in a slurry trench which is dug after the piles have been set and the slurry around them has hardened. Tremie concrete placed in slurry trenches to form cast-in-place diaphragm walls could also be used. Roof members shown in this illustration are precast, prestressed I-girders with a cast-in-place roof deck. Again, if deep fill is required, these I-girders would necessarily be very deep and heavy.

The utility corridor idea could also have application with the king pile type of construction. Elimination of the backfill would reduce the required size of the horizontal members, but obviously would require twice

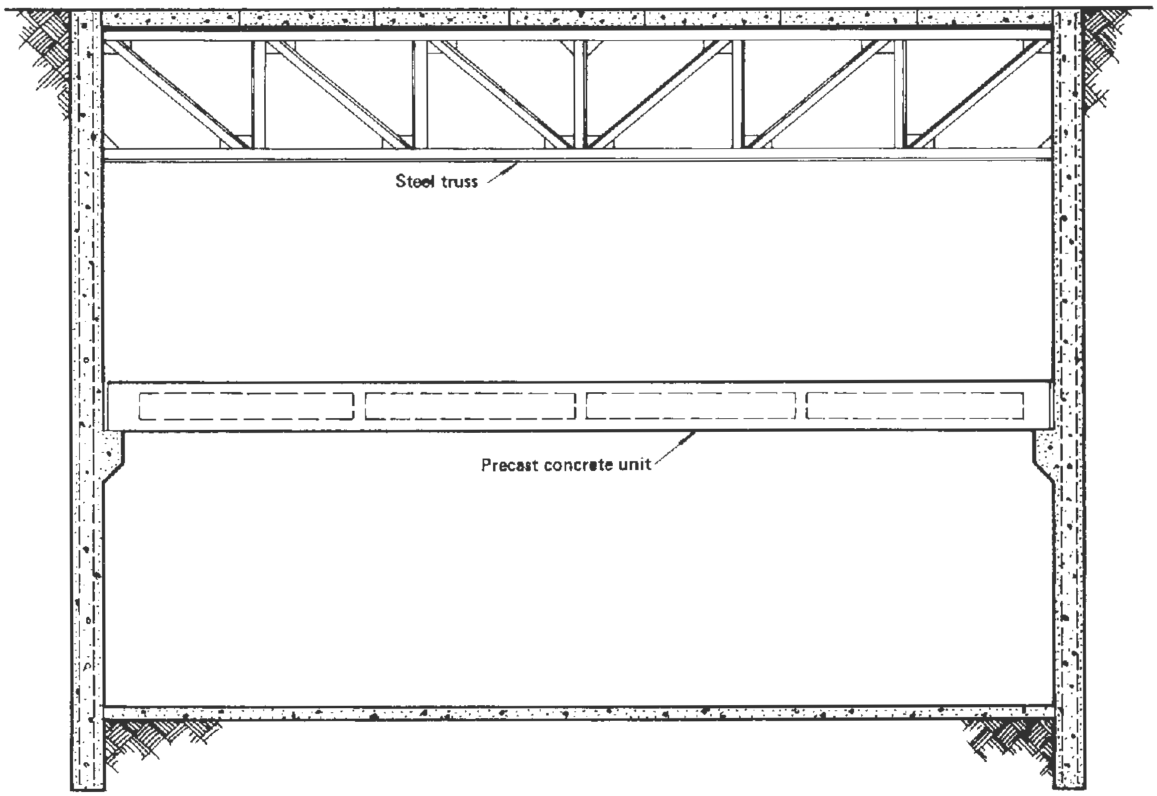


Fig. 28 Steel truss used to support upper level.

as many of them. It is also possible to design a truss in which the horizontal members at the street level and at the tunnel roof level act as the chords. This is illustrated in Fig. 27.

Use of a steel truss for the horizontal members in conjunction with either a continuous or king pile wall system is another possibility (Fig. 28). The web members of this steel truss could also be used to support utilities within the space. Although this is not a true utility corridor concept, it would be possible to provide access to the space between the street level and the tunnel roof level so that utilities could be maintained without disrupting street level traffic. Steel trusses are easily

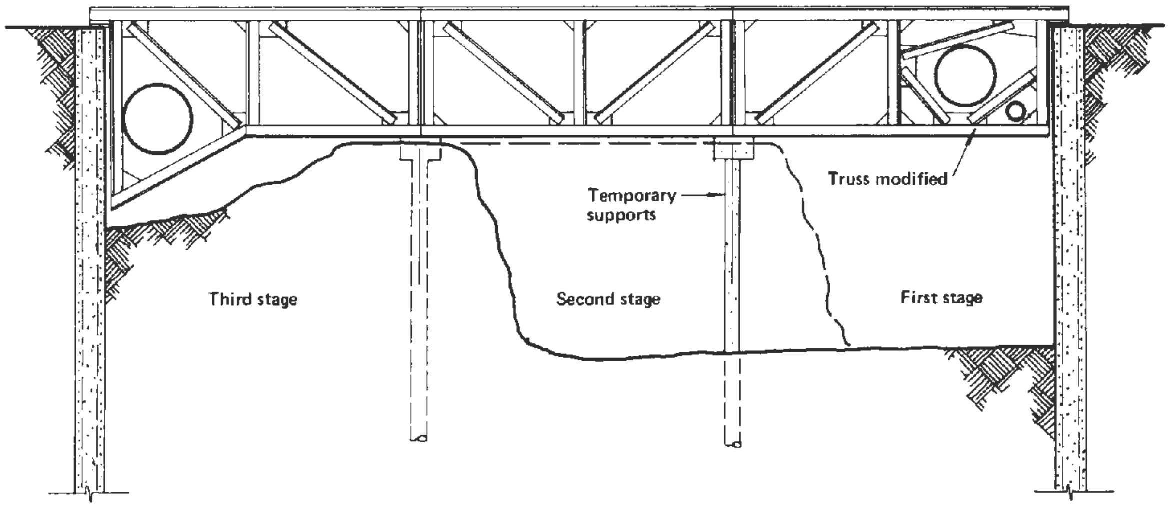


Fig. 29 Stage construction and field modification of steel trusses to accommodate existing utilities.

field modified if necessary to provide space for utilities to pass through. They can also be fabricated in sections and field connected easily with no loss of integrity, opening the way for stage construction allowing the street to remain partially open during placing of the steel members. This concept is illustrated in Fig. 29, and is discussed further in Section XI.

V. LOADS ON TUNNEL STRUCTURE

A. VERTICAL LOADS

The Standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials (AASHTO Specifications) provides specifications and guidelines for loading conditions to be imposed on highway bridges and culverts. These loads often do not precisely apply to cut-and-cover tunnel construction, especially those constructed of prefabricated structural elements in the manner suggested in this report. This section will cite the various provisions of the AASHTO Specifications (Eleventh edition, 1973, with 1974 and 1975 Interims), and suggest how these provisions might be applied to the construction methods recommended in this report.

1. Earth loads. Section 1.2.2(A) of the AASHTO Specifications allows the weight of earth above a culvert to be taken as 70 percent of its actual weight. This reduction apparently is in recognition of soil arching. For those structures which have permanent walls extending up to or near the ground surface, it is doubtful that any arching beyond the walls is possible, however, arching within the walls would reduce the moments in horizontal members supporting the backfill. The magnitude of the reduction should be judged on the basis of soil properties, presence of water and span length. Total weight should always be used for shear on horizontal members and for the vertical load-bearing members, haunches, or other connections between the horizontal and vertical members, except at internal supports.
2. Dead load of the structure. Use actual weights of the members.
3. Utilities. When there is backfill over the tunnel roof, the

weight of utilities can normally be neglected. With no backfill, a reasonable analysis should be made of the utilities being supported.

4. Permanent live loads. It is assumed that most cut-and-cover tunnels in urban areas are placed under streets. Loading assumptions can be based on standard HS-20 truck loading. Loading criteria in the AASHTO Specifications in general assumes that members span in the direction of traffic. Most of the schemes suggested in this report have the flexural members spanning transverse to the direction of traffic. Therefore, it was necessary to develop loading criteria based on the assumed intent of the AASHTO Specifications. The following shows the development of the equivalent uniform load used in the sample designs in this report.

   a. For "double roof" systems, or shallow cuts (no fill).

      (1) Determine the number of traffic lanes which may be supported (Par. 1.2.6, Interim 1974, AASHTO Specifications)

         36' to 47' - 3 lanes

         48' to 59' - 4 lanes

         60' to 71' - 5 lanes

      (2) Determine reduction in load intensity (Par. 1.2.9)

         3 lanes - design for 90%

         4 or more lanes - design for 75%

      (3) Determine impact factor (Par. 1.2.12)

      (4) Determine sum of axle loads which will be supported by the member.

(a) For adjacent members, or those spaced at 6 ft or less, use Par. 1.3.1(c). This allows a load distribution of from S/5.5 to S/7.0. For this study an average value of S/6.0 is used. (S = beam spacing). The maximum axle load, P, is 32,000 lb.

(b) For members spaced more than 6 ft apart an analysis based on Fig. 30 was used:

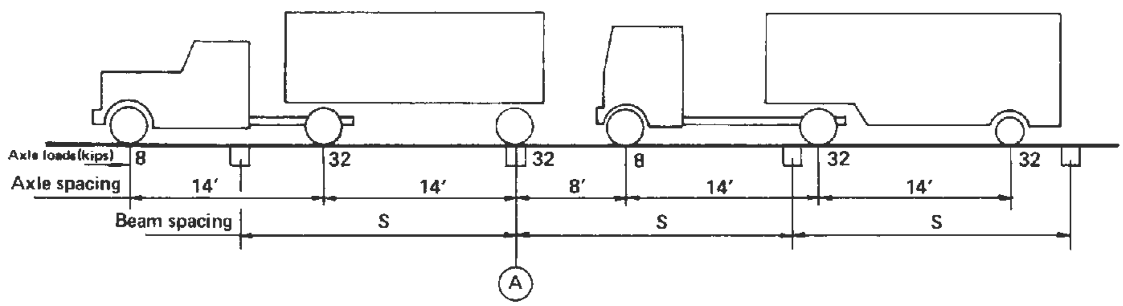


Fig. 30 Assumed truck load distribution to structural members spanning transverse to direction of traffic.

Using this assumption, the maximum load on a beam (at point A) is:

For $S \leq 8$ ft

$$P = 32{,}000 \text{ lb}$$

For $8 < S < 14$

$$P = 32{,}000 + \frac{(S-8)}{S}(8000)$$

For $14 < S < 22$

$$P = 32{,}000 + \frac{(S-8)}{S}(8000) + \frac{(S-14)}{S}(32{,}000)$$

For spacing greater than 22 ft, additional axles can receive partial support from the member and additional terms should be added.

(5) The equivalent uniform load is:

$$w = \frac{P \times \text{no. of lanes} \times \text{reduction factor} \times \text{impact factor}}{\text{span} \times \text{spacing}}$$

For span and beam spacings within the range appropriate for cut-and-cover tunnels, equivalent uniform loads are tabulated in Table I.

b. For sections with fill on the roof, the live loads are distributed in accordance with Par. 1.3.3. The equivalent uniform load is computed as follows: Steps 1 through 3 are the same as in a. above, except impact factors may be assumed as for culverts (Par. 1.2.12).

The axle loads are considered as distributed longitudinally over a length equal to 1.75 times the depth of fill, with a maximum length of distribution of 14 ft.

$$w = \frac{P \times \text{no. of lanes} \times \text{reduction factor} \times \text{impact factor}}{\text{span} \times S_1}$$

P = 32,000 lb for HS-20 loading

S_1 = 1.75 x depth of fill, maximum 14 ft

impact factor = 1.20 for fill depth = 1'1" to 2'-0"

= 1.10 for fill depth = 2'-1" to 2'-11"

= 1.0 for fill depth 3'-0" or greater

w need not be greater than that calculated in a.(5). For spans within the range appropriate for cut-and-cover tunnels, equivalent uniform loads are tabulated in Table 2.

Table 1

Equivalent Uniform Live Load (psf) - HS-20 Loading
Cut-and-Cover Tunnel Sections without Backfill
Beams Spanning Transverse to Traffic Direction

BEAM SPACING (ft)	SPAN (ft) 42	44	46	48	50	52	54	56	58	60	62	64
2	446	424	405	430	411	395	379	365	351	423	409	395
3	446	424	405	430	411	395	379	365	351	423	409	395
4	446	424	405	430	411	395	379	365	351	423	409	395
5	446	424	405	430	411	395	379	365	351	423	409	395
6	446	424	405	430	411	395	379	365	351	423	409	395
7	382	364	347	368	353	338	325	313	301	363	350	339
8	334	318	303	322	309	296	284	273	263	318	307	296
9	305	291	277	294	282	270	260	250	241	290	280	271
10	281	267	255	271	259	249	239	230	221	267	258	249
11	260	247	236	250	240	230	221	212	205	247	238	230
12	241	230	219	233	223	214	205	198	190	229	221	214
13	225	215	205	217	208	200	192	184	178	214	207	200
14	211	201	192	204	195	187	180	173	167	201	194	188
15	211	201	191	203	195	187	179	173	166	200	194	187
16	209	199	190	201	193	185	178	171	165	198	192	185
17	206	196	187	198	190	182	175	168	162	196	189	183
18	202	192	184	195	187	179	172	165	159	192	185	179
19	198	189	180	191	183	175	169	162	156	188	182	176
20	194	184	176	187	179	172	165	159	153	184	178	172
21	189	180	172	183	175	168	161	155	149	180	174	168
22	185	176	168	178	171	164	157	151	146	176	170	164

Table 2

Equivalent Uniform Live Load (psf)- HS-20 Loading
Cut-and-Cover Tunnel Sections with Backfill
Beams Spanning Transverse to Traffic Direction

Equivalent Uniform Load (psf)

Fill Depth (ft)	SPAN (ft) 42	44	46	48	50	52	54	56	58	60	62	64
2	446	424	405	430	411	395	379	365	351	423	409	395
3	392	374	358	381	366	352	339	327	315	381	369	357
4	294	281	268	286	274	264	254	245	236	286	276	268
5	235	224	215	229	219	211	203	196	189	229	221	214
6	196	187	179	190	183	176	169	163	158	190	184	179
7	168	160	153	163	157	151	145	140	135	163	158	153
8 or more	147	140	134	143	137	132	127	122	118	143	138	134

Example 1: 60 ft span, no backfill, 4 ft wide adjacent roof units at street level. Determine equivalent uniform live load.

(1) For 60 ft span, assume 5 traffic lanes

(2) Reduce load to 75%

(3) Impact factor (AASHTO Specifications, Par. 1.2.12)

$I = 1 + 50/(L + 125) = 1 + 50/(60 + 125) = 1.27$

(4) $P = 32{,}000 \times 4/6 = 21{,}333$ lb

(5) $$w = \frac{21{,}333 \times 5 \times 0.75 \times 1.27}{60 \times 4} = 423 \text{ psf}$$

Example 2: 56 ft span, no backfill, beams spaced at 20 ft o.c. Determine equivalent uniform live load.

(1) For 56 ft span, assume 4 traffic lanes

(2) Reduce load to 75%

(3) Impact factor = $1 + 50/(56 + 125) = 1.28$

(4) $$P = 32{,}000 + \frac{(20 - 8)}{20}(8000) + \frac{(20 - 14)}{20}(32{,}000) =$$ 46,400 lb

(5) $$w = \frac{46{,}400 \times 4 \times 0.75 \times 1.28}{56 \times 20} = 159 \text{ psf}$$

Example 3: Same as example 1 except assume 7 ft of fill on tunnel roof.

(3) Impact factor = 1.0

(5) $$w = \frac{32{,}000 \times 5 \times 0.75 \times 1.0}{60 \times (7 \times 1.75)} = 163 \text{ psf}$$

5. Temporary Live Loads. In urban construction sites, working space for construction equipment is often limited. Efficiency can sometimes be gained if the equipment which places the prefabricated members can be supported on the members previously placed. Construction equipment which is working adjacent to the excavation may also induce a relatively large surcharge on the ground support

Fig. 31 Construction equipment and supplies causes temporary live loads on Cut-and-cover tunnel structures.

system.

It is common practice to use a conservative uniform load of approximately 400 psf to account for these temporary live loads. However, if the permanent structure acts also as the ground support system and roof deck, such a conservative assumption may unnecessarily penalize the structure, adding to the cost. A better approach would be for the designer to determine the actual loading conditions during construction, and perhaps specify certain limitations of temporary loading. Such restrictive specifications, however, are difficult to enforce in the field and the designer should try to anticipate all logical loading combinations, with an allowance for error. Use can also be made of temporary shores or heavy timber distribution blocks.

B. DESIGN OF LATERAL PRESSURES FOR DIAPHRAGM WALLS USED IN CUT-AND-COVER TUNNEL

Walls of cut-and-cover tunnels which serve as both the temporary ground retention system and the final structure must be designed for combinations of horizontal pressures due to earth, water and surcharge loading for the various stages of construction, as well as for the completed, permanent structure. Lateral pressure supported by the retention system depends upon the method of excavation support; i.e., bracing levels, installation procedure (whether preloading is done or not), whether ground movements are permitted to mobilize active pressures and stages in excavation, in addition to the soil parameters.

For diaphragm walls, the lateral pressures associated with surcharge and water pressures are generally the same for temporary support during construction and for the long term condition of the completed structure. Methods of determining these pressures are available in geotechnical literature. Some of the most applicable methods are briefly reviewed in the following paragraphs.

1. Surcharge Loading. Surcharge loadings adjacent to excavations vary according to the type of loading, such as loads from foundations for structures, traffic, storage of construction materials, and construction equipment. These surcharge loading conditions, can be approximated by one of the following:

   a. point loading

   b. uniform line loading

   c. uniform area loading

   The determination of lateral pressures resulting from each of the three loading conditions are described below. The lateral pressures due to surface loading are in addition to the earth and water pressures.

a. Point Loading. Point loadings can be assumed for isolated footings for a structure or heavy object situated on a small base. Fig. 32 shows the analysis of pressure distribution on the wall due to a point load. This figure is from the Design Manual of the Department of Navy entitled "Soil Mechanics, Foundation and Earth Structures" (NAVFAC DM-7,

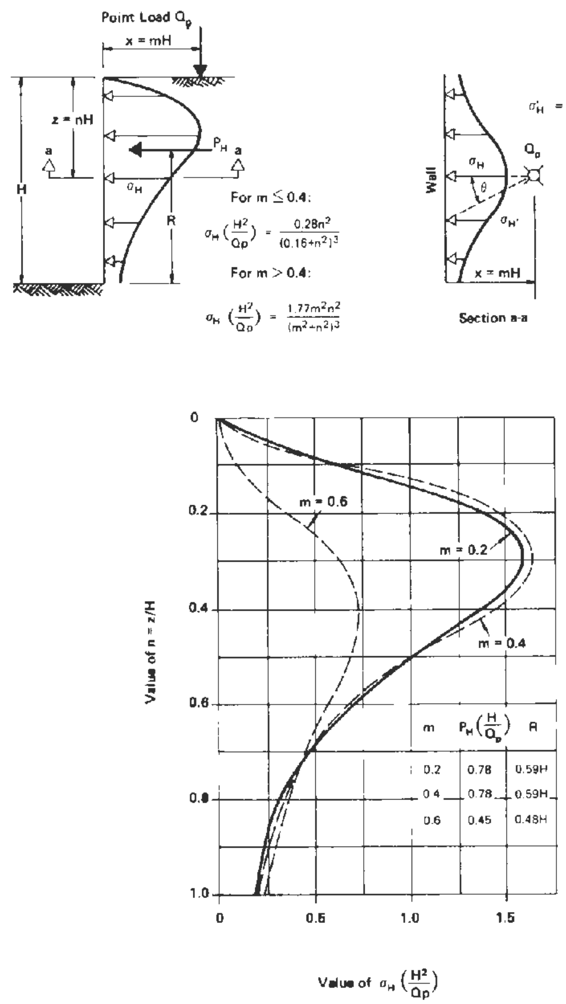


Fig. 32 Lateral pressure due to a point surcharge load. From "Soil Mechanics, Foundations and Earth Structures" (NAVFAC DM-7, March 1971)

March 1971). The pressure distributions shown are based on Boussinesq Theory, modified by experiment.

b. <u>Line Loading.</u> Line loadings can be assumed for a continuous strip footing or railroad running parallel to the excavation. Fig. 33 presents the pressure distribution on the wall resulting from a line loading. This presentation is also from NAVFAC DM-7 (1971) and is based on Boussinesq Theory, modified by experiment.

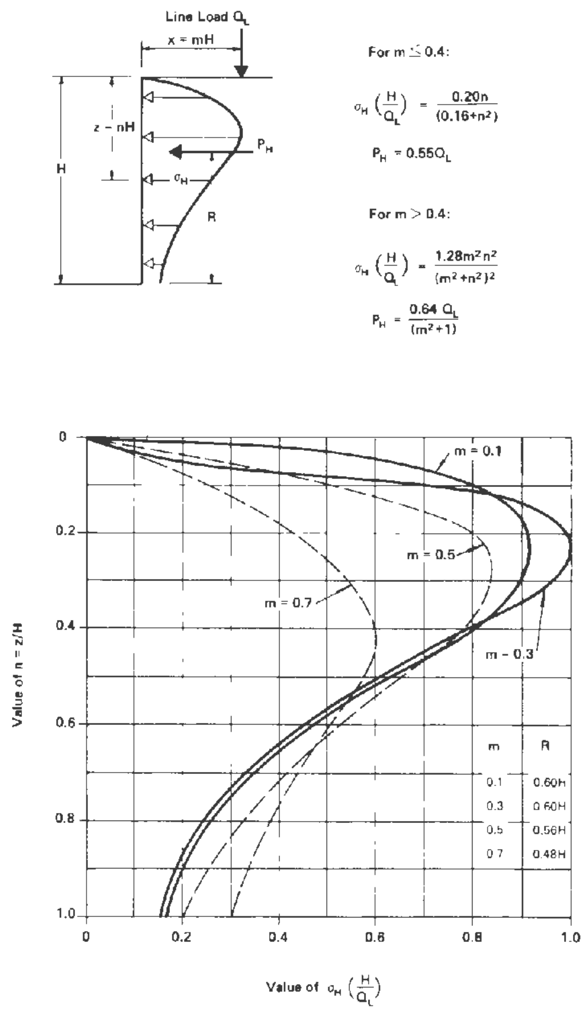


Fig. 33 Lateral pressure due to a line surcharge load. From "Soil Mechanics, Foundations and Earth Structures" (NAVFAC DM-7, March 1971), Dept. of the Navy.

More exact solutions for the lateral pressures resulting from infinite strip loads (both uniform loads and embankment type loading) can be found in the publication entitled "Vertical Stress Tables for Uniformly Distributed Loads on Soil" by A. R. Jimikis (Engineering Research Publication No. 52, Rutgers University).

c. Uniform Area Loading. Uniform surcharge loading can usually be assumed for area wide storage of construction materials or traffic in nearby streets located relatively close to the wall. Usually an average uniform pressure over the area adjacent to the wall can be determined from the weight of the objects and the area covered. Surcharge loading of this nature may be assumed to be of infinite extent behind the wall to simplify the calculations for lateral pressures.

The lateral pressure due to a uniform surface loading of infinite extent is as follows:

$$\sigma_h = k_o \, q$$

where σ_h = the horizontal pressure

k_o = coefficient of earth pressure at rest

q = magnitude of surcharge loading

For relatively stiff retaining structures, such as those employing braced diaphragm walls, the lateral earth pressure coefficient should be equal to the at rest coefficient, k_o, since there should be relatively little wall movement, insufficient to mobilize fully active earth pressures. For evaluation of k_o, refer to paragraph 3-a(2).

2. Water Pressure Loading. For relatively impermeable diaphragm walls, the water pressure loading can be significant, especially if the water table is high. In many cases, water loads are greater than the loads due to earth pressures.

Loads due to water pressures are usually considered in granular soils, sands, gravels, and silts. However, for granular soils, effective unit weights should be used in the earth pressure calculations below the water table. Water pressures must then be added to the loading resulting from earth and surcharge pressures. In cohesive soils (clays), the effects of water pressures are normally accommodated through the use of total unit weights in calculations of earth pressures.

In the computations for water pressures on diaphragm type walls, the maximum probable water level should be used. The hydrostatic water pressure at any level is equal to the unit weight of water (62.4 pcf) times the depth below the maximum groundwater table.

3. Earth Pressure Loading. The magnitude and distribution of earth pressures are dependent on a number of factors; i.e., soil conditions, stiffness of the system, lateral movements, and construction procedures, such as preloading of struts or tiebacks. Earth pressures for temporary construction support and the completed permanent structure may be different due to the movements involved and should be considered separately.

   a. Temporary Construction Support.

      (1) Strut Loads. The conventional approach for development of the earth pressure diagram for internally braced excavation

walls is to use the apparent earth pressure diagram recommended by Peck (Peck, Hanson, Thornburn, Foundation Engineering, 2nd Ed., 1974, pp. 460-461). The apparent pressure diagram was developed for computation of the strut loads for flexible walls. Fig. 34 shows Peck's apparent pressure diagrams for both sand and clay soils and these diagrams should be used for the design of the temporary struts or tiebacks. The recommended procedures for distributing the pressures to the individual struts is illustrated in Fig. 35.

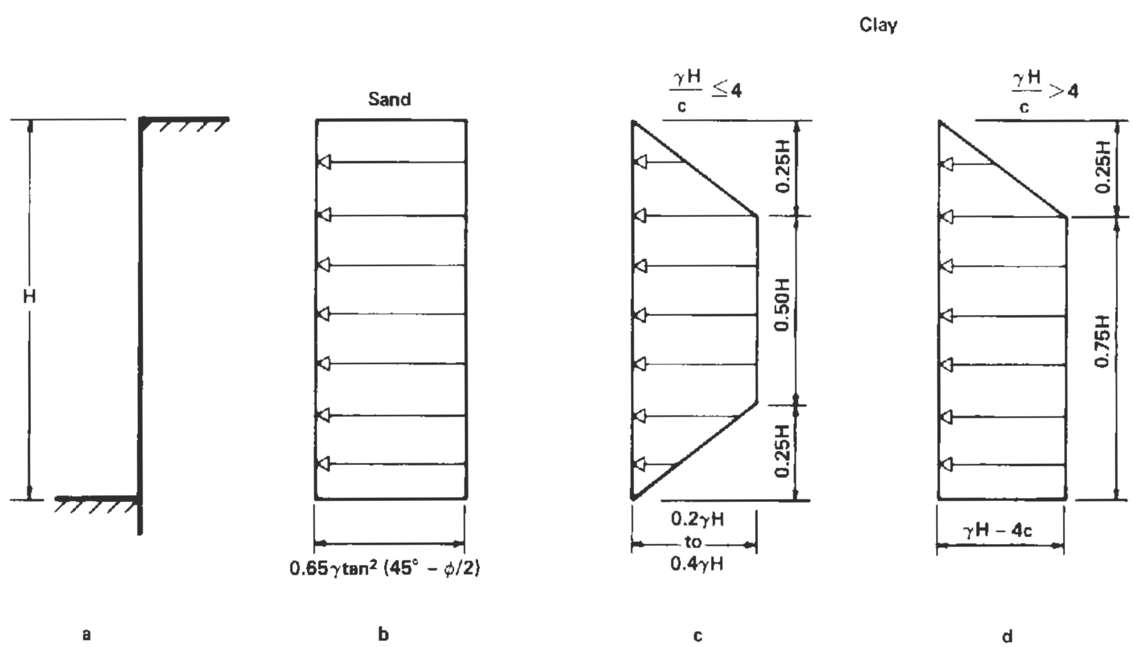


Fig. 34 Peck's apparent pressure diagram. From "Foundation Engineering" by Peck, Hanson, and Thornburn, 2nd Ed., 1974.

(a) Sands. Fig. 34 for sand applies to predominantly cohesionless soils, including non-cohesive silt. Unit weight, γ refers to the moist unit weight (γ_m) of the soil above the ground water table and the submerged unit weight ($\gamma_{submerged} = \gamma_{saturated} - \gamma_{water}$) below the water level. Hydrostatic water pressures must be treated separately. Typical values of the moist unit weights for granular soils would be in the range of 105 to 135 pcf, with saturated unit weights in the range of 115 to 140 pcf. The actual values depend on the material gradation and tightness of packing. The angle of internal friction, ϕ, generally varies from about 28° for loose, granular soils to approximately 43° for very dense materials.

(b) Clays. The earth pressure computation for temporary

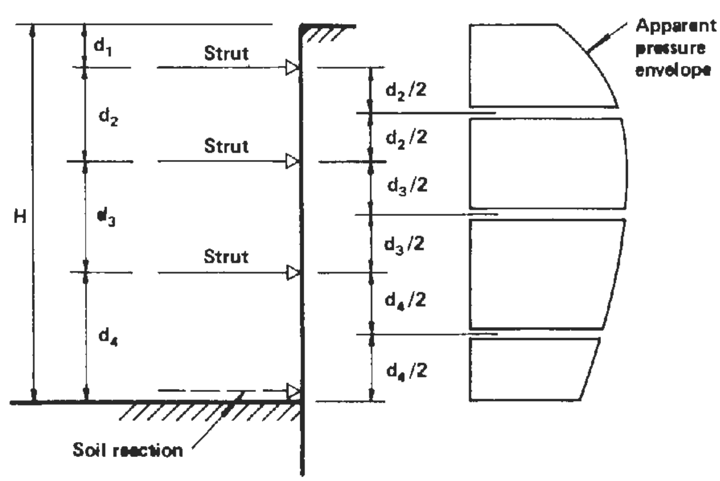


Fig. 35 Distribution of lateral loads to struts from apparent pressure diagram. From "Foundation Engineering", by Peck, Hanson, and Thornburn, 2nd Ed., 1974

restraint of clays or other predominantly cohesive soils is based upon the total unit weight of the soil, without regard to hydrostatic conditions. The range of total unit weights for cohesive soils is 115 to 135 pcf, depending on the consistency and grain size distribution, with the softer, more plastic materials having the lower unit weights. The undrained shear strength, c, for cohesive soils can be summarized as follows, depending on the consistency:

Consistency	Undrained Shear Strength, c, (psf)
Very soft	<250
Soft	250 - 500
Medium	500 - 1,000
Stiff	1,000 - 2,000
Very Stiff	2,000 4,000
Hard	>4,000

(2) Wall Design. For the analysis and design of the diaphragm wall itself, it is recommended that a triangular pressure diagram be used. The horizontal pressure will depend on the coefficient of earth pressure used. The wall can be considered semi-rigid, so lateral movements will be small, not large enough to mobilize the fully active condition. Since the active earth pressure is less than the earth pressure at rest (no lateral movement), it is suggested the coefficient of earth pressure be taken as $k' = (k_a + k_o)/2$, where k_a is the coefficient of active

earth pressure and k_o is the coefficient of earth pressure at rest. The horizontal earth pressure at the base of the cut is then equal to k' times γH (see Fig. 36).

For granular soils, $k_a = \tan^2 (45^o - \phi/2)$

$$k_o = 1 - \sin \phi$$

and γ represents the γ_m (moist unit weight) above the water table and γ_{sub} (submerged unit weight) below the groundwater level.

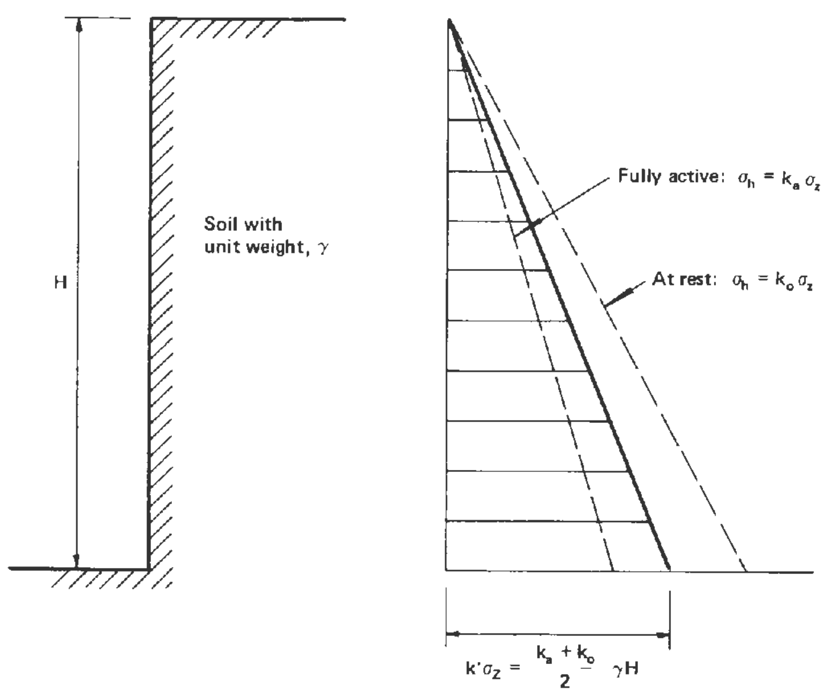


Fig. 36. Recommended earth pressure diagram for the analysis and design of diaphragm walls.

For cohesive soils,

General Case - where ϕ and c are the friction angle and cohesion intercept respectively:

$$k_a = \tan^2 (45^\circ - \phi/2) - \frac{2c}{\gamma H} \tan (45^\circ - \phi/2)$$

Special Case - Where $\phi = 0^\circ$ and c = undrained shear strength

$$k_a = 1 - \frac{2c}{\gamma H}$$

k_o has a value of 0.5 for normally consolidated clays and a value in the range of 0.5 to 2.0 for overconsolidated clays. γ represents the total unit weight.

(3) Required Depth of Embedment of the Wall. The depth of penetration of the wall below the base of the cut should be sufficient to prevent a failure about the base and also prevent excessive movement. The embedment necessary to provide a support point below subgrade should be analyzed. (See Fig. 37).

Passive resistance provided by the subsoils can be expressed as $\sigma_p = k_p \gamma d$

σ_p = passive pressure at depth d below the cut

k_p = coefficient of passive resistance

γ = unit weight

Since the movements required to mobilize the full passive pressures are relatively large, it is recommended that a factor of safety of 1.5 be applied to the passive

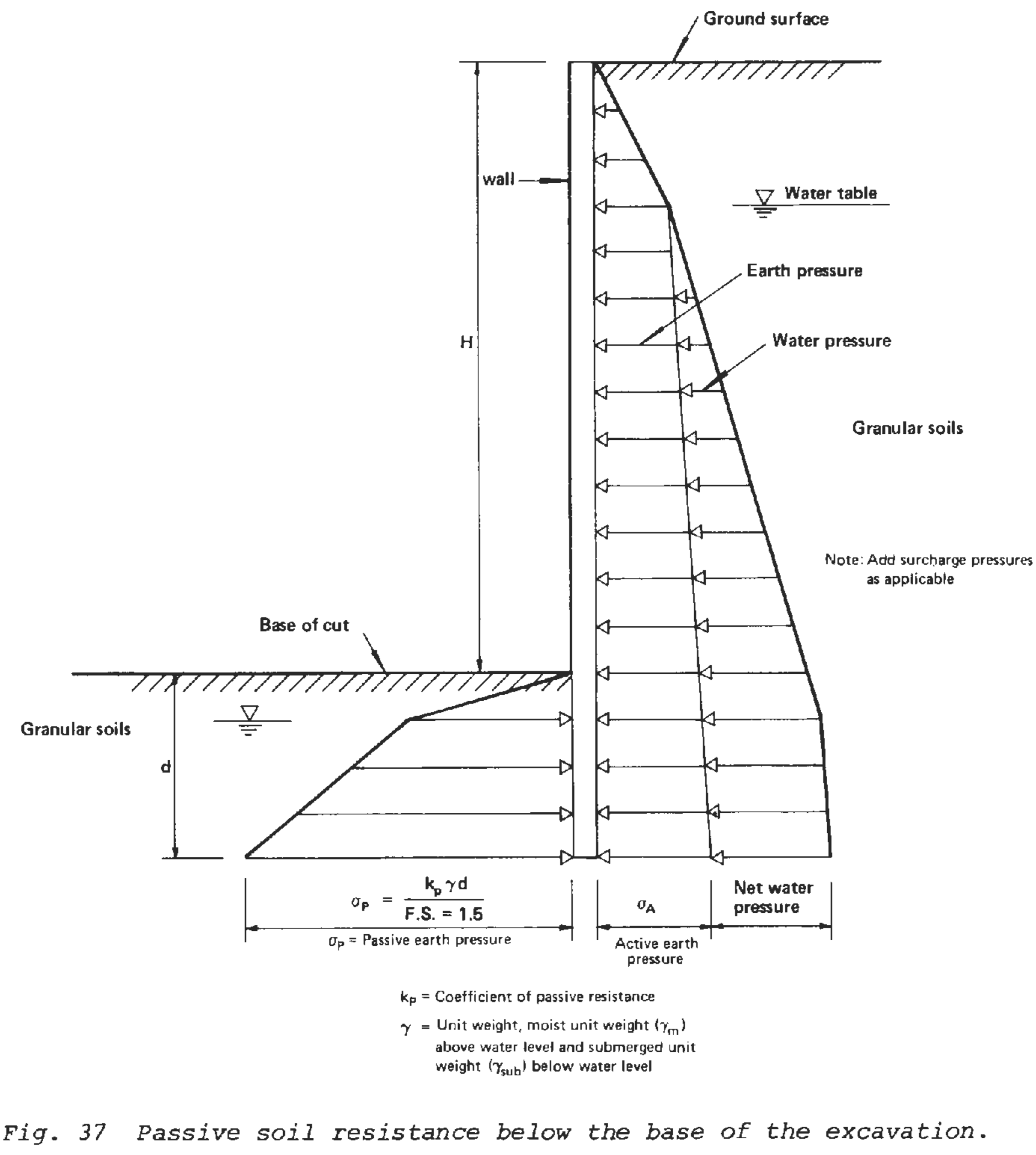


Fig. 37 Passive soil resistance below the base of the excavation.

resistances.

For sands, $k_p = \tan^2 (45^{\circ} + \phi/2)$

However, friction between the wall and the soil usually develops and the resultant of the passive pressure acts at angle, δ, with the horizontal. The increased k_p due to the wall friction (δ) are shown in Fig. 10-3 in NAVFAC DM-7 (1971).

For clays, the passive pressure (σ_p) at depth, d, below the base of the cut is equal to

$$\sigma_p = \sigma_v \tan^2 (45^{\circ} - \phi/2) + 2c \tan (45^{\circ} + \phi/2)$$

Where $\sigma_v = \gamma d$, and ϕ and c are the friction angle and cohesion intercept, respectively.

In the special case where $\phi = 0$ and

c = undrained shear strength,

$$\sigma_p = \sigma_v + 2c$$

In cohesive soils, wall friction is generally ignored.

b. <u>Completed Permanent Structure.</u> For the completed, permanent structure, deflections of the structure walls are restrained and, therefore, the wall movements will be insufficient to mobilize the fully active earth pressures. For this reason, the exterior walls should be designed for the more conservative condition of at rest earth pressure, in addition to the water pressure (for the case of granular soils) and lateral pressure due to surcharge loadings. Refer to Section 3-a(2) for the computation of the earth pressures at rest. The roof of the structure should be designed for the combination of the vertical overburden pressure plus surcharge as described in Part A of this section. Fig. 38 shows the pressure diagrams for the design of the permanent structure.

4. <u>Alternate, Refined Structural Analysis of Diaphragm Walls.</u>

The above mentioned method of analysis of the earth retention system represents the conventional approach. A more exact method of analysis would be to take into account the soil-structure

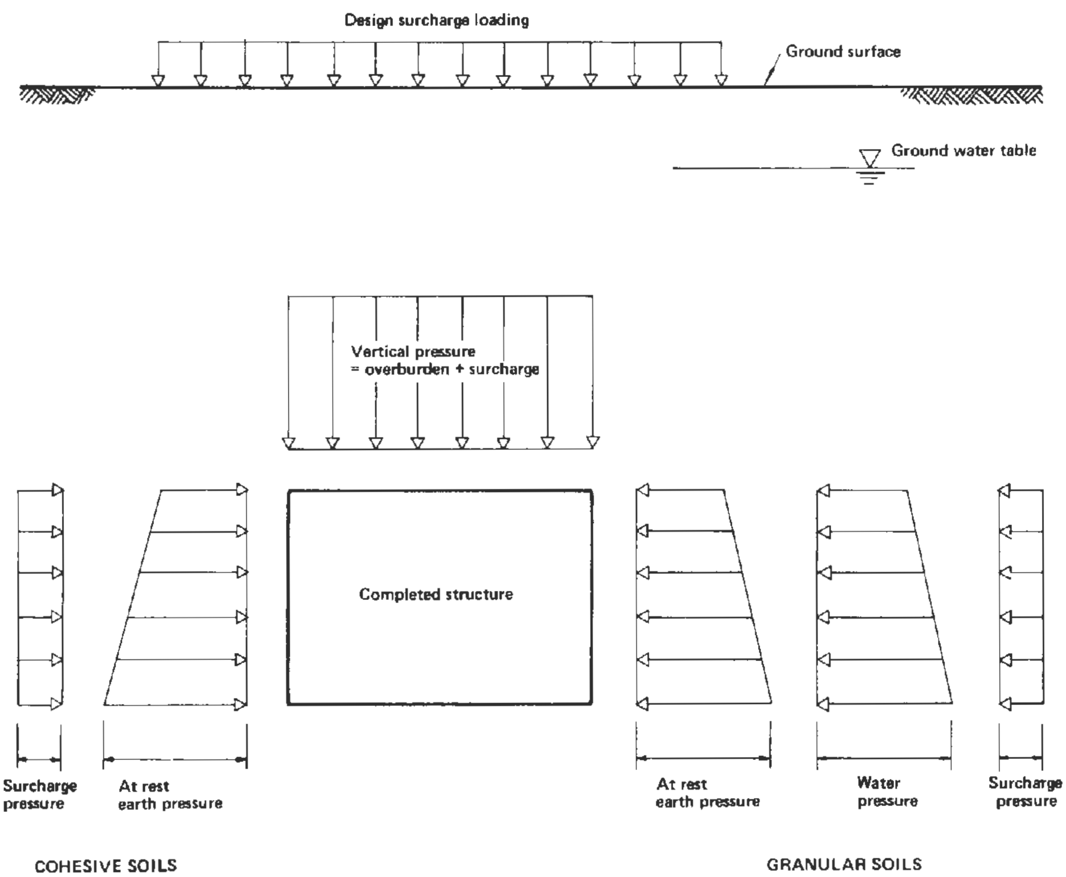


Fig. 38 Design pressures for permanent, completed structure.

interaction. Such analyses may employ a discrete element or finite element approach through the use of computers. The discrete element approach involves analysis of the wall as if it is a continuous beam which rests on a bed of springs which simulates a passive soil resistance. (Fig. 39). This analysis is based on the theory of subgrade reaction, in which the lateral soil resistance depends on the displacement:

$$k_h = \frac{\text{pressure}}{\text{displacement}} = \frac{p}{y}$$

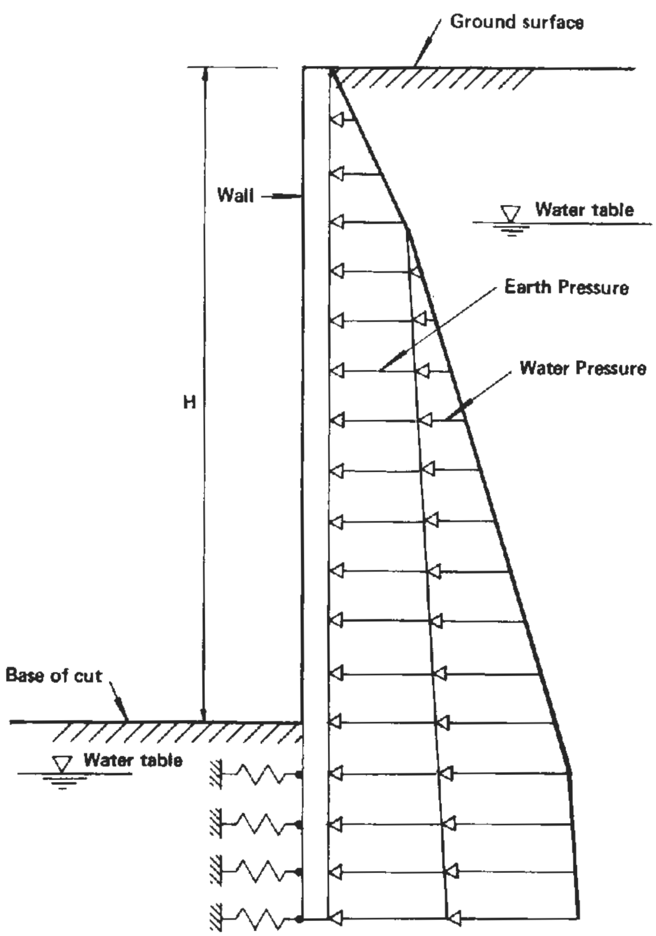


Fig. 39 Discrete element analysis

where k_h is the modulus of horizontal subgrade reaction. This subgrade modulus can be input into special computer programs or readily available structural analysis programs such as STRESS* or ICES-STRUDL* by examining the pressure/displacement relationship:

$$y = \frac{PL}{AE} = \frac{pL}{E}$$

Therefore:

$$\frac{p}{y} = \frac{E}{L} = k_h$$

For input into the analysis program, the supporting element, or "spring", would have an area equal to the vertical spacing of the springs times the unit width of wall and values of length, L, and modulus of elasticity, E, to satisfy the above equation. For example, assume the horizontal subgrade modulus at the support area being considered is determined to be equal to 150 tons/cu ft. A one foot width of wall is being analyzed and the supports, or springs, are spaced vertically at 1'-0". If the required input units are kips and inches, as in STRESS, then the area of the support is 144 inches. A convenient length, say 10 inches, is selected and the modulus of elasticity value would then be:

$$E = k_h \times L = \frac{150 \text{ tons}}{\text{cu ft}} \times \frac{2 \text{ kips}}{\text{ton}} \times \frac{\text{cu ft}}{1728 \text{ cu in.}} \times 10 \text{ in.} = 1.74 \frac{\text{kips}}{\text{sq in.}}$$

Terzaghi** has related the modulus of horizontal subgrade reaction k_h, to the values obtained from field load tests on small plates. In granular soils and normally consolidated cohesive soils, it is

*STRESS and ICES-STRUDL are general structural analysis programs developed at the Massachusetts Institute of Technology, and can be purchased for some in-house computers. Many computer time-sharing services offer their use on a royalty basis.

**Terzaghi, K., "Evaluation of Coefficients of Subgrade Reaction" Geotechnique, Vol. 5, No. 4, Dec., 1955.

assumed that the modulus increases linearly with depth, so that k_h is proportional to n_h (from Table 3) times the depth in feet. In overconsolidated clays, the horizontal subgrade modulus is constant with depth, so that k_h is directly related to the value of k_1 (from Table 4).

Table 3. Typical values of n_h for granular soils and normally consolidated clays from plate load tests (tons/cu ft).

Soil type	Relative Density		
	Loose	Medium	Dense
Sand, dry or moist	7	21	56
Sand, submerged	4	14	34
Clay, soft, normally consolidated	1 to 2		

Table 4. Typical values of k_1 for precompressed clays

	Consistency		
	Stiff	Very stiff	Hard
Unconfined compression strength (tons/sq ft)	1-2	1-4	>4
k_1 range (tons/cu ft)	50-100	100-200	>200
k_1 recommended (tons/cu ft)	75	150	300

The finite element approach models the soil as a continuum and provides a better representation of the system than the discrete element approach although it is more complex to use. Finite Element Analysis of Braced Excavations is described by Goldberg*.

*Goldberg, D. T., Jaworski, W. E. and Gordon, M. D. "Lateral Support Systems and Underpinning", Federal Highway Administration Offices of Research & Development Report No. FHWA-RD-75-129, April, 1976.

Both of the above methods allow consideration of wall-soil interaction effects and support flexibility. An advantage of the discrete element and finite element approach is that they allow computation of diaphragm wall and soil movements, along with bending moments throughout the depth of the wall.

VI. DESIGN OF PREFABRICATED STRUCTURAL MEMBERS

A. SPECIAL CONSIDERATIONS

In designing prefabricated members for cut-and-cover tunnels, the loading conditions at various stages of construction, as well as during service, must be considered. This is particularly significant when the members are to serve the double duty of temporary retaining vessel and permanent structure.

The loading conditions during construction can be controlled to some extent by the proper placement of struts, shores, or tiebacks. Also, stresses and deflections in the members can be changed by controlled jacking of struts, tensioning of tiebacks, or in some cases, placement of post-tensioning tendons in the members which can be tensioned and de-tensioned as construction proceeds to produce the desired effects.

For temporary loading conditions, higher unit stresses (or lower factors of safety) can be allowed than for the permanent structures.

For systems which use wall units placed prior to excavation, the loadings and support conditions change as excavation proceeds, and struts or tiebacks are placed. Prior to each change of support condition, certain stresses become "locked-in". Calculation of stresses, therefore, begins with an initial stress condition that is different from that calculated by taking each construction stage with the loads active upon the structure at that stage. This situation is modified to some extent by creep, which has the effect of redistributing the stresses toward the theoretical stress condition that would occur if there were not "locked-in" stresses. Since the extent of this redistribution is impossible to determine accurately, some reasonable assumptions must be made, with an allowance for error. This is illustrated in Fig. 40 for one given condition of structure configuration and site conditions. Fig. 40 a shows the maximum moments assuming no redistribution, and Fig. 40b shows the maximum moments assuming full redistribution. The actual condition lies somewhere between these two extremes, probably closer to the no-redistribution condition. Fig. 40a assumes that no external force is applied at the struts by jacking. While it is common practice to put a jacking force on temporary struts, when part of the permanent structure is used as a strut, it may not be practical or desirable to do so. With a properly braced rigid wall, such as a cast-in-place or precast concrete diaphragm wall, deflections

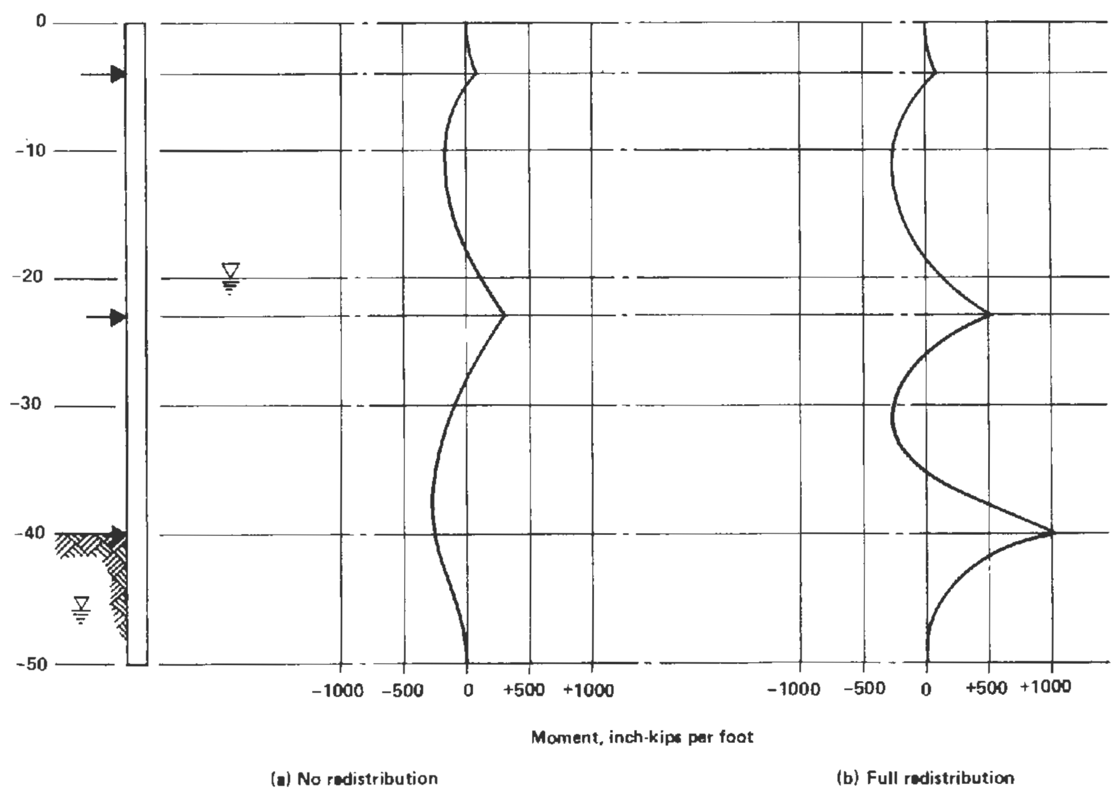


Fig. 40 Moments in rigid diaphragm retaining walls.

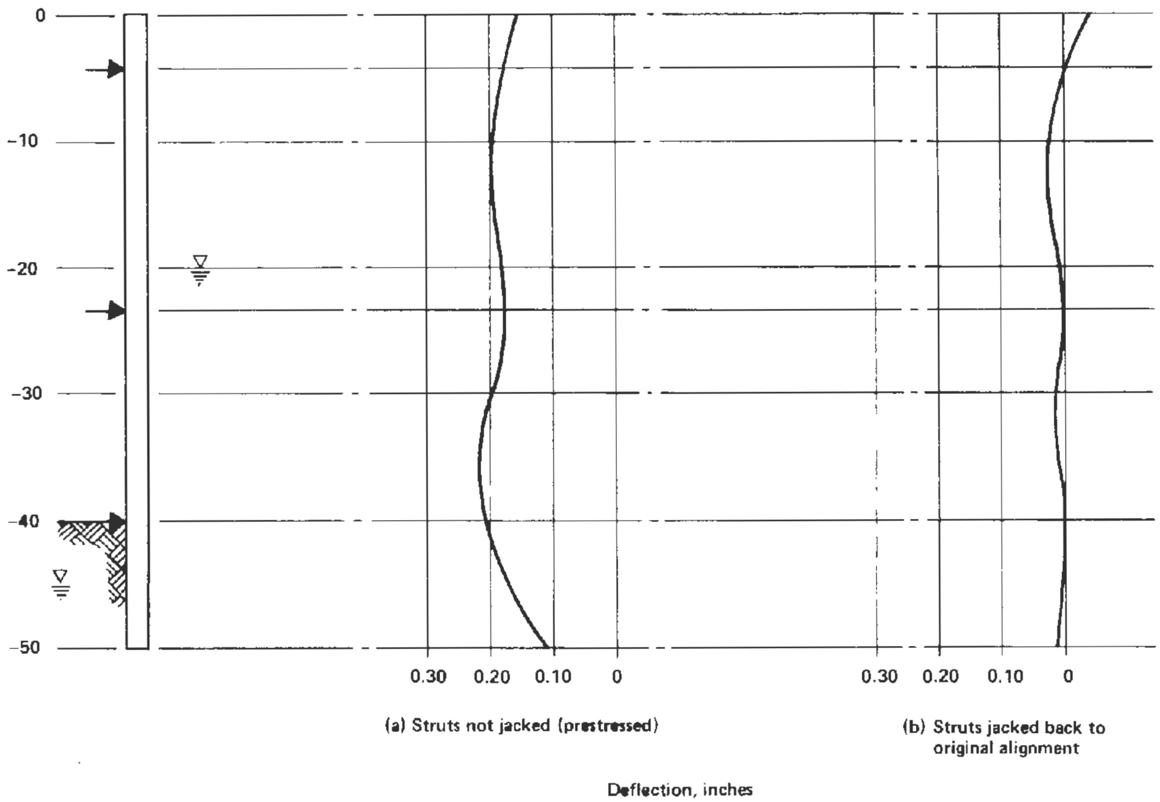


Fig. 41 Deflections in rigid diaphragm retaining walls.

will normally not be so severe that damage to nearby structures is a problem. (See Fig. 41).

If the prefabricated units are to be used for both the ground support during construction and as the permanent tunnel walls, consideration of the construction method is an integral part of the design of the structure. If the construction contract is to be let by competitive bidding, then the designer must also specify and take responsibility for the construction method. The alternative to this is the letting of design-build contracts. While this system is common for all types of construction in other countries, and for much private work in the United States, it has seldom been used for public works projects here.

B. WORKING STRESS VS. LOAD FACTOR DESIGN

Load factor design is a relatively new method. It has been applied to reinforced concrete buildings to some extent since 1956. There is still some confusion in terminology. Prior to the adoption of the 1971 version of Building Code Requirements for Reinforced Concrete (ACI 318-71) by the American Concrete Institute, the method was usually called "Ultimate Strength Design". The Standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials, 1973, refers to the method as "Load Factor Design", and extends its use to structural steel members. However, in Section 6 "Prestressed Concrete", the term "ultimate capacity" is still frequently used. The same is true of most publications of the Prestressed Concrete Institute, including the PCI Design Handbook. In this report, the terms "Load Factor Design and "Ultimate Strength Design" are used interchangeably.

In the design of prestressed concrete members, codes and specifications require that both the flexural stress conditions under service loads and the ultimate capacity of members be checked.

C. LOAD FACTORS

For "load factor" or "ultimate strength" design, the load factors given in the AASHTO Bridge Specifications may not precisely apply to cut-and-cover tunnel design.* The AASHTO Bridge Specifications specify a load

*Some engineers experienced in the design of underground structures do not consider the ultimate strength method applicable to such structures, primarily because the load factors are unclear and may be inadequate. For example, the WMATA Manual of Design Criteria requires that all underground reinforced cast-in-place concrete structures be designed by the "Alternate Design Method" of ACI (working stress method).

factor of 1.30 for dead load. In the cases where nearly all of the effective dead load is caused by earth fill, use of such a low load factor may result in an overall factor of safety which is too small to be comfortable. Also, the AASHTO Bridge Specifications do not specify a load factor for lateral earth or water pressures.

Therefore, with the exception of the conditions where the dead load is due only to the gravity load of the member and surfacing materials, and highway live loads are used, the load factors recommended below are taken from the ACI Building Code (ACI 318-71).

1. Recommended Load Factors:

   Dead loads caused by gravity* = 1.4

   Live loads (truck loading) and impact = 1.3(1.67) = 2.17

   Other live loads = 1.7

   Lateral earth pressure = 1.7

   Lateral fluid pressure = 1.4

   For application of these load factors and other loadings not mentioned here, refer to Section 9.3 of the ACI Building Code (ACI 318-71).

2. Capacity Reduction Factors:

   The 1973 AASHTO Bridge Specifications recommends the capacity reduction factors (ϕ) shown in Table 5 for use in load factor design. ACI 318 makes no distinction between reinforced concrete and prestressed concrete regarding capacity

*Note: When dead loads are the result of member weight only, and highway live loads are used, a 1.3 factor as recommended in the AASHTO Bridge Specifications may be used.

reduction factors. The factors are the same as listed in Table 5 under "Reinforced Concrete".

Table 5. Capacity reduction factors, ϕ, specified in "Standard Specifications for Highway Bridges", American Association of State Highway and Transportation Officials, 1973.

Reinforced Concrete		Prestressed Concrete	
Flexure	0.90	Factory produced precast members	1.0
Shear	0.85	Post-tensioned cast-in-place members	0.95
Spirally reinforced compression members	0.75	Shear	0.90
Tied compression members	0.70		
Bearing	0.70		

3. Design Methods:

Load factor design methods and equations are given in both the AASHTO Bridge Specifications and ACI 318-71. Since most of the members in the schemes suggested in this report are subject to both axial load and flexure, the strain compatibility approach is most applicable.

D. DESIGN STRESSES

1. Structural Steel and Reinforced Concrete:

Use recommendations in AASHTO Bridge Specifications.

2. Prestressed Concrete:

Use recommendations in AASHTO Bridge Specifications, except it is recommended that for stresses caused by dead or sustained loads, such as earth fill and lateral pressures, no tension be allowed. For temporary conditions during construction, it is recommended that tension up to $6\sqrt{f'_c}$ be allowed.

E. DESIGN OF PRESTRESSED CONCRETE BEAM-COLUMNS

In the methods of construction suggested in this report both the vertical and horizontal members are subjected to a combination of axial loads and bending. They must, therefore, be designed as beam-columns, taking into account the interaction between the axial loads and bending moments.

1. For Allowable Stresses.

In most cases with the range of load factors and the allowable stresses recommended in Sections B & C, the allowable stress criteria, rather than the ultimate strength criteria, will control the design.

For vertical load bearing members, subject to lateral loads from soils, ground water, and surcharge, the moments can vary quite widely, and even reverse, during the various stages of handling and construction. Because of this, it is suggested that a concentric prestress force might be most desirable to allow for these variations. This decision must be made by the project engineer.

For horizontal members, the axial load from the horizontal pressures can have the effect of an additional prestressing. However, the magnitude of this horizontal pressure cannot be calculated with sufficient precision to rely on as beneficial. It is, therefore, recommended that the horizontal member be designed to resist the vertical loads with or without this axial load. The end connection of the horizontal and vertical members should be detailed with proper placement of shims, so that the horizontal load is delivered in a position that minimizes the detrimental effect.

Construction of interaction curves, or direct solutions are relatively simple for prestressed concrete members. A typical example for a vertical member is shown in Appendix A.

2. Load Factor Design:

Direct solution based on the ultimate capacity of a prestressed concrete beam-column is a very difficult process. Construction of ultimate interaction curves is much simpler and is suggested as the proper approach. For solid, rectangular sections, load tables and computer programs for calculating interaction points are available. For the voided sections suggested in this report, the problem is more complex because of the more complex shape of the compression stress block. Interaction curves for standard precast, prestressed concrete piles, both solid and hollow, have been published.* The calculation method is illustrated in Appendix A.

3. Slenderness:

For either the working stress or ultimate capacity solution, the effects of slenderness should be considered. This can be done by either using one of the classical methods, or by using the moment magnification concept suggested in the ACI Building Code ACI 318-71). The slenderness effects apply only to moments caused by eccentricity of the axial loads, and not to the moments

*Anderson, Arthur R. and Moustafa, Saad E., "Ultimate Strength of Prestressed Concrete Piles and Columns", ACI JOURNAL, Proceedings V. 67, No. 8, Aug. 1970, pp 620-633.

caused by normal loads. For the unsupported lengths usually associated with highway tunnels, and for vertical members designed to carry lateral loads as well as vertical loads, slenderness effects are usually minimal.

4. <u>Sample Calculations:</u>

Sample calculations for a typical precast, prestressed concrete load bearing wall member subject to lateral loads from earth and water pressure are shown in Appendix A. These calculations are not intended to represent any particular design condition, but only to illustrate the method of design.

VII. COMPARISON OF MATERIAL REQUIREMENTS

As stated earlier, prefabrication of structural concrete elements will reduce costs only if material quantities are reduced. Precasting, especially when combined with pretensioning, does offer several opportunities to substanially reduce the material costs without impairing performance. In fact, performance and quality control can usually be significantly improved.

Savings in materials can be accomplished by incorporating one or more of the following:

A. Cast-in-voids in the interior of the unit. These voids can be round or rectangular, as illustrated in Fig. 44.

B. High strength concrete. Design compressive strengths of 5000 psi or 6000 psi are common in the precasting industry, and strengths up to 9000 or 10,000 are obtainable.

C. Prestressing. The use of pretensioning strands in lieu of mild reinforcing will usually reduce the total reinforcing cost, as well as improving the performance in terms of crack and deflection control. For some uses, discussed in Section XII of this report, the use of post-tensioning may be advantageous. By embedding the post-tensioning tendons and the anchorages in precast elements, more accurate placement is possible than with cast-in-place concrete walls placed in slurry trenches.

Because of differences in performance criteria, precise qualitative comparisons are very difficult. The following analysis does attempt to determine some relative cost advantages resulting from the material savings gained by use of each of the preceding factors. The elastic interaction curves as shown in Fig. 42 are a convenient method of comparison. By matching curves calculated with different material quantities or properties, sections with approximately equal capacities can be compared. These are shown in Figs. 43 through 46.

A. CAST-IN-VOIDS

The potential material savings possible by using cast-in voids in precast structural members is illustrated by comparing the section designed in Appendix A with a solid section of the same dimension. Fig. 43 shows the elastic interaction diagram for both the voided section (solid lines) and the solid section (dashed lines). Note that use of the voided section (area = 567 in.^{2}) would result in a

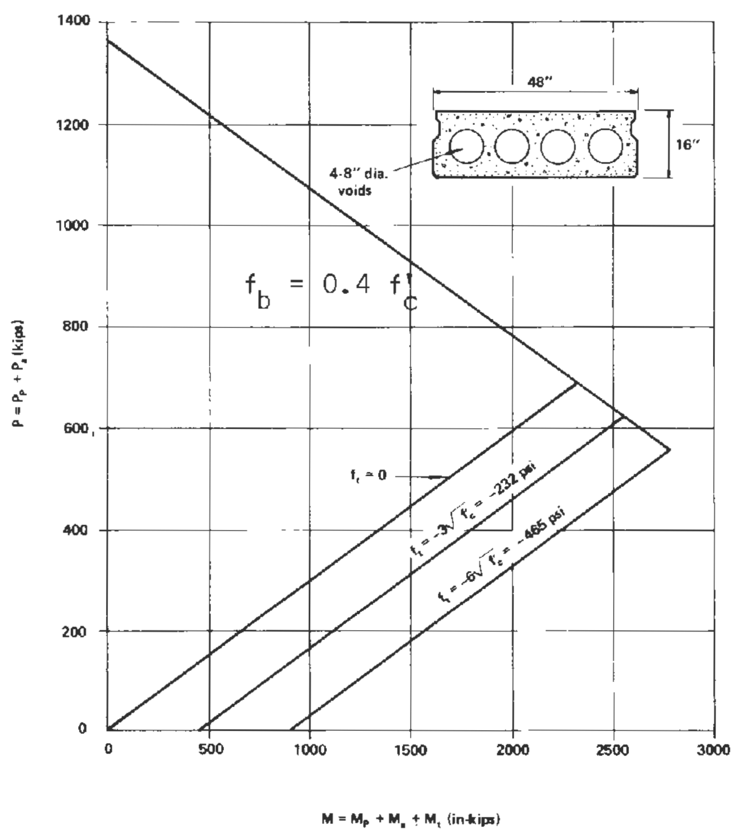


Fig. 42 Elastic interaction curves for a voided prestressed concrete beam-column wall unit. (See Appendix A)

35% savings in concrete over the solid section (area = 768 in.^{2}). Note also that because of the additional area, additional prestressing force is required to meet the same criteria of zero tension under sustained loads.

In this particular case, an additional concrete savings could be realized by using a box section. The comparison of interaction diagrams are shown in Fig. 44.

The savings in material is offset somewhat by the cost of forming the voids. In the case of prefabricated elements, however, additional savings result from reduced hauling and placing costs due to the lighter weight, or, wider (or longer) sections can be used with the same weight, allowing faster placing of the wall.

B. HIGHER STRENGTH CONCRETE

Again using the section described in Fig. 42 as par, a comparison of the material requirements for higher and lower strength concretes can be made using the elastic interaction diagrams. By trial and error it can be shown that for equal capacity, a section using 4000 psi concrete would need to be about 20 inches thick, compared with the 16 inch thick member with 6000 psi concrete. The area of a 20 inch member is 759 sq in., compared with 567 sq in., indicating a 34% savings in concrete. However, the 20 inch member would require somewhat less prestressing steel, in this case 20 strands instead of 24, and the cost of 4000 psi concrete is somewhat less than 6000 psi concrete. (See Fig. 45).

A similar analysis using 8000 psi concrete is shown in Fig. 46. In this case, a 14 in. thick member is adequate to satisfy the stress

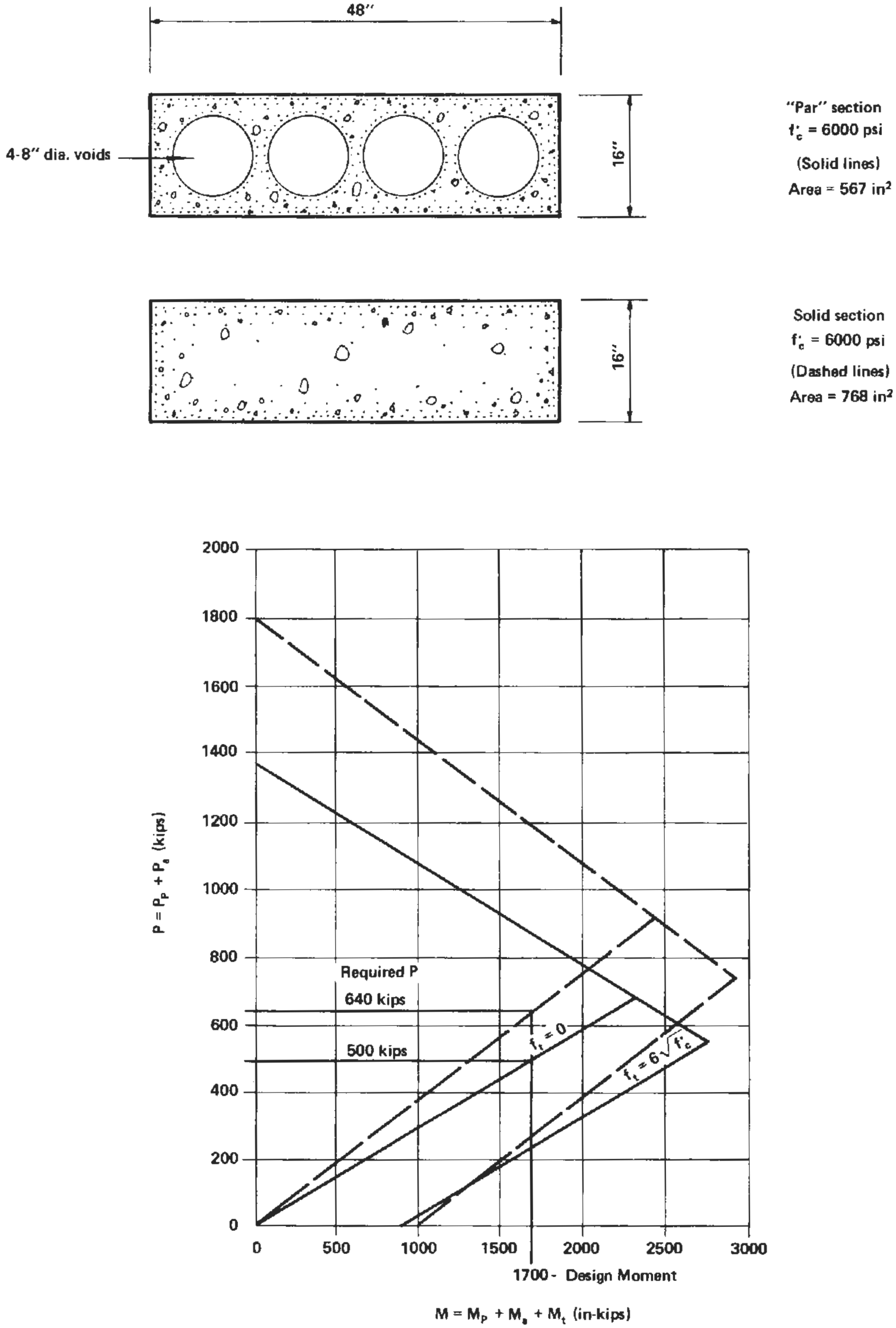
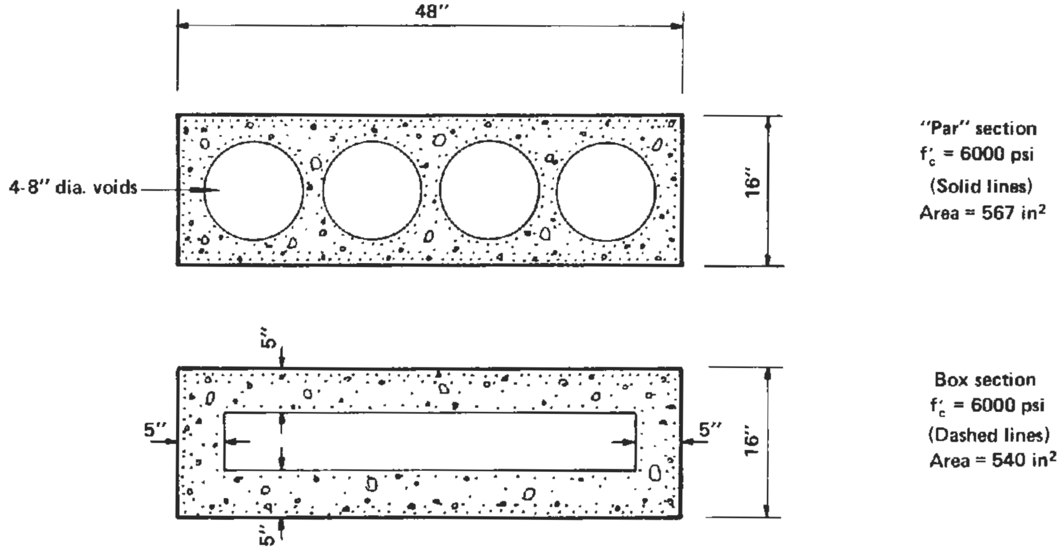


Fig. 43 Comparison of solid section and section with round voids.



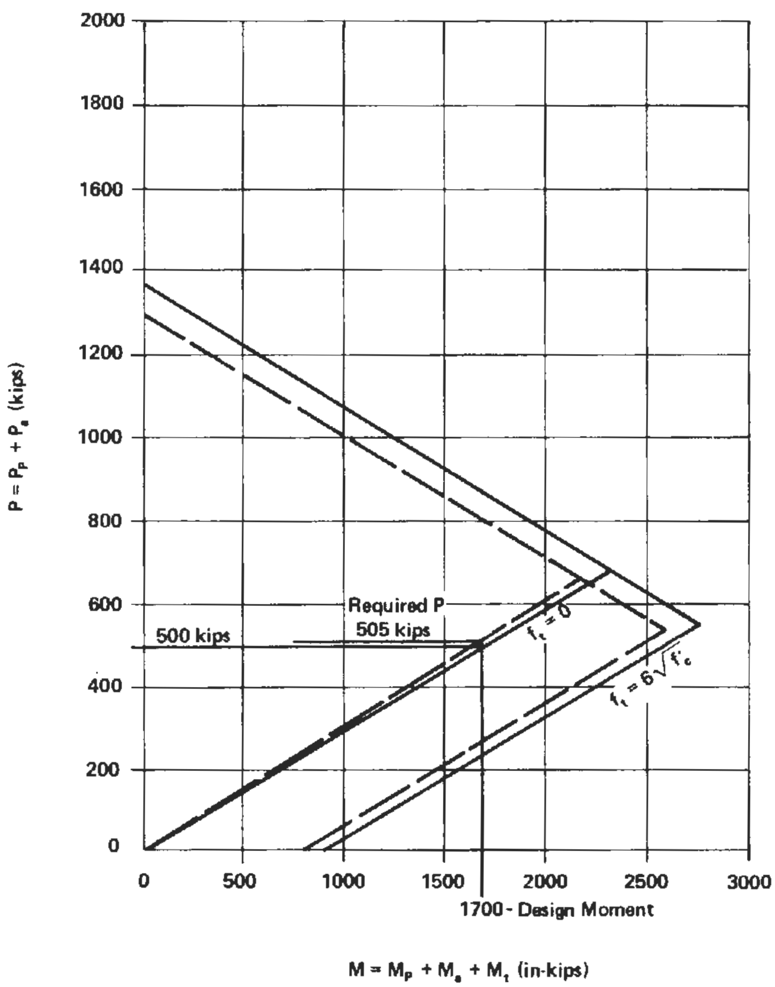


Fig. 44 Comparison of section with round voids and box section.

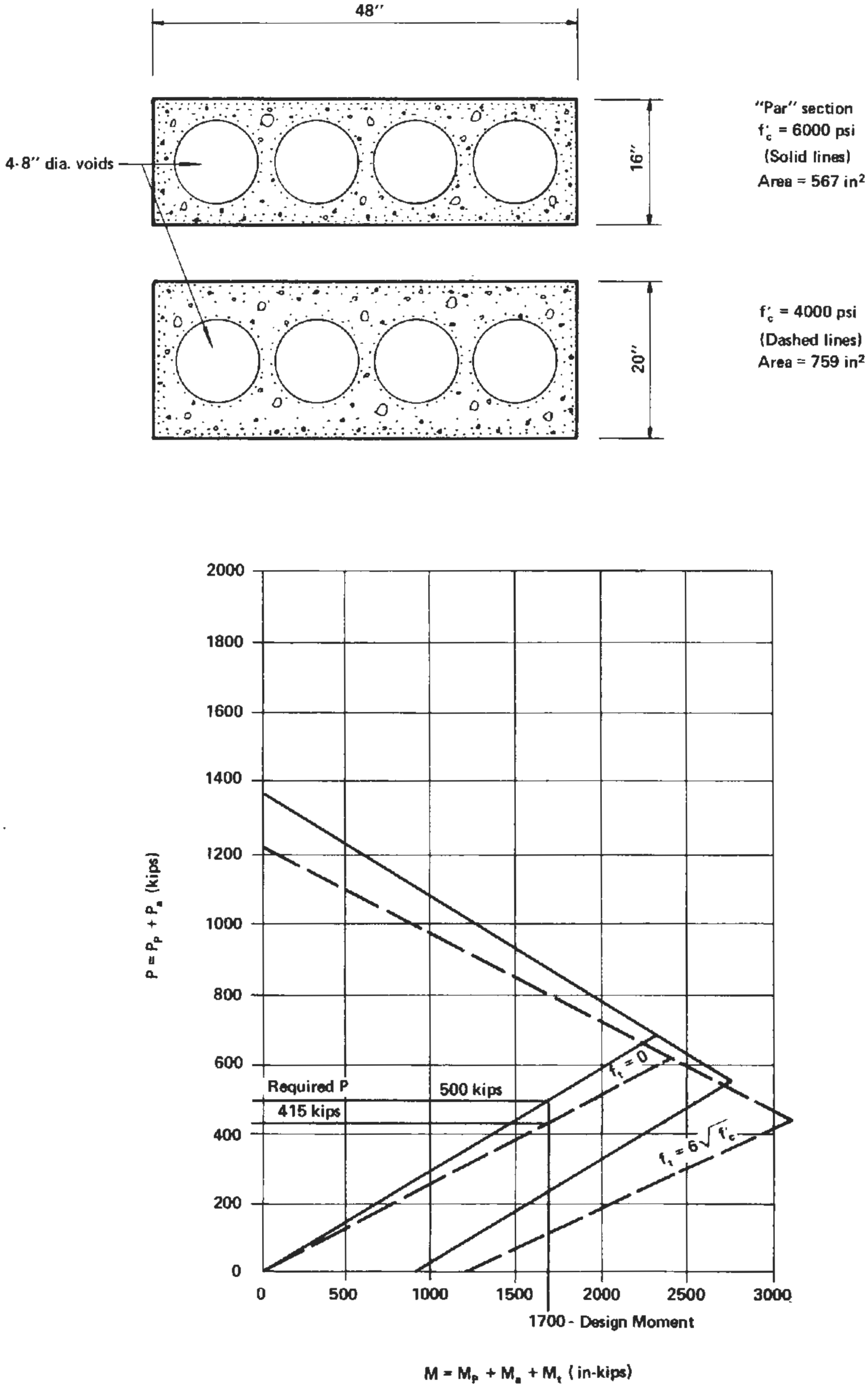
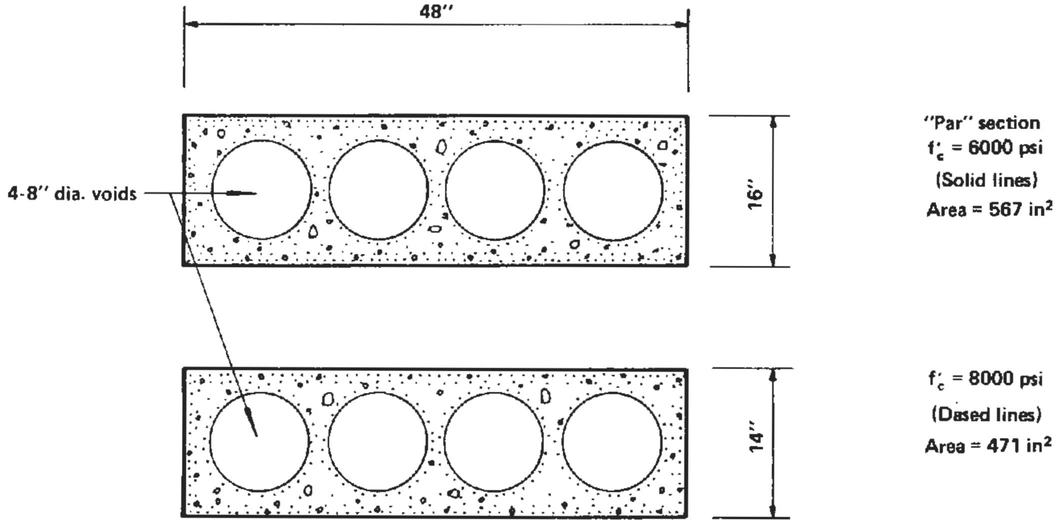


Fig. 45 Comparison of 6000 psi concrete with 4000 psi concrete.



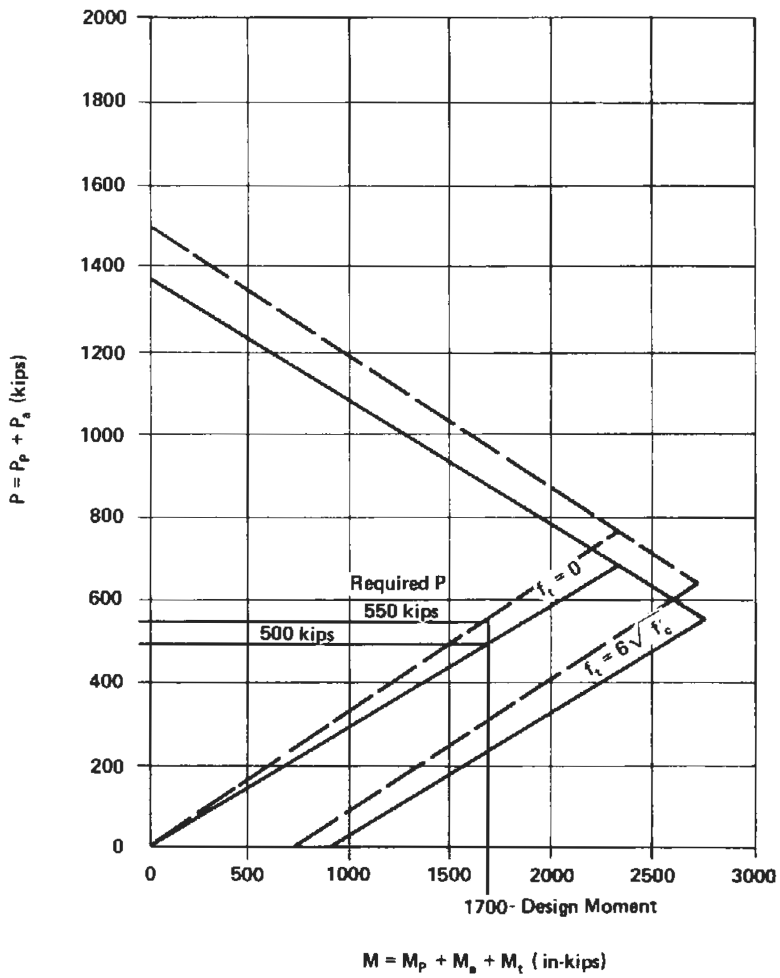


Fig. 46 Comparison of 6000 psi concrete with 8000 psi concrete.

criteria. In this instance, the reduced thickness would probably not offset the additional cost of the higher strength concrete.

It should be remembered that for pretensioned members cast in a plant, the critical strength is often that which is necessary to release the prestress force. For a one-day turnover of the stressing beds, usually desirable for maximum economy, this release strength requirement will normally yield a final (28-day) strength in excess of 5000 psi.

Higher strength concrete also results in an increase in the modulus of elasticity, E, so that elastic deflections will be somewhat less. However, in the range of strengths and the dimensions considered here, the difference in total stiffness is usually not great.

C. PRESTRESSING VS. MILD STEEL REINFORCING

An approximate analysis (by working stress methods) of a mild steel reinforced section the same size and concrete strength as the "par" section indicates a total of twenty-eight #6 longitudinal bars are required. This compares with twenty-four 1/2 in. diameter strands required for the prestressed section. At current (1975) prices, the prestressed member would show a savings of about $2.35 per square foot (35¢ per lb for reinforcing in place, 22¢ per ft for strand in place). This assumes that all bars and strands extend the full length of the member. It may be possible in an actual design to cut off part of the reinforcing bars, but a certain amount of minimum reinforcing would be required.

It should also be noted that the prestressed member is designed to avoid cracking. Some cracking will take place with the reinforced member.

VIII. FOUNDATIONS

A. SUPPORT OF VERTICAL GRAVITY LOADS.

Cut-and-cover tunnel construction as proposed in this report, with the tunnel structure also acting as the ground support system during construction, presents some unique problems in the design of the foundation. In "conventional" construction the excavation is opened up completely by the use of temporary retaining walls and the permanent structure is built within these walls. Footings can be constructed to a required size and vertical loads can be transmitted through the footings to the bearing material with little difficulty. Most often the floor of the tunnel acts as the foundation and is constructed first, while in most of the structures suggested in this report the floor of the tunnel is the last structural element to be placed. Therefore, if the loads are to be transmitted to the bearing soil through the floor slab, special details and construction procedures are required.

If the vertical load bearing elements, whether they are continuous walls or intermittent king piles, are cast-in-place, then the wall itself can bear directly on the foundation soil or rock, assuming that the foundation material has adequate bearing capacity to carry the loads imposed upon it. If additional bearing area is required because of inadequate soil bearing capacity, a standard belling tool as used in the construction of caisson foundations can be used. If the structure is of king pile construction as shown in Section IV, this method is quite straight forward and requires little additional comment. If the selected construction method employs continuous bearing walls then the belling tool must be inserted at regular intervals and a continuous widening of the trench at the bottom can be made. Obviously, this requires an additional construction operation and additional costs.

If the vertical load bearing elements are to be of precast concrete construction, either the king pile type or adjacent load bearing panels, different options are available as follows:

1. A setting bentonite-cement slurry can be used during excavation. This slurry can be designed to have a strength after setting at least equal to the strength of the adjacent soil. Such grout will normally have greater strength than is required to transmit the horizontal earth and water pressure loads to the wall without significant deformation. Setting slurries with strengths up to 30 psi (4320 psf) are quite easily attainable. The Soletanche Company has patented a "coulis" which they claim will provide strength up to 200 psi (28,800 psf). These very-high-strength setting slurries, however, require high percentages of portland cement and admixtures which increases their cost.

2. In many instances it may be more desirable to pour cast-in-place concrete at the base of the precast element to sufficient heights that the precast element is securely anchored into the cast-in-place material. This concept is illustrated in Fig. 47.

3. For some of the methods proposed in this report, particularly those in which there is backfill over the tunnel roof and the supporting elements are spaced apart, it may be more economical to support the loads of the backfill on cast-in-place columns which are cast after the tunnel floor slab is in place. The floor slab of the tunnel is a natural footing because it very often must be quite deep and designed structurally to carry hydrostatic

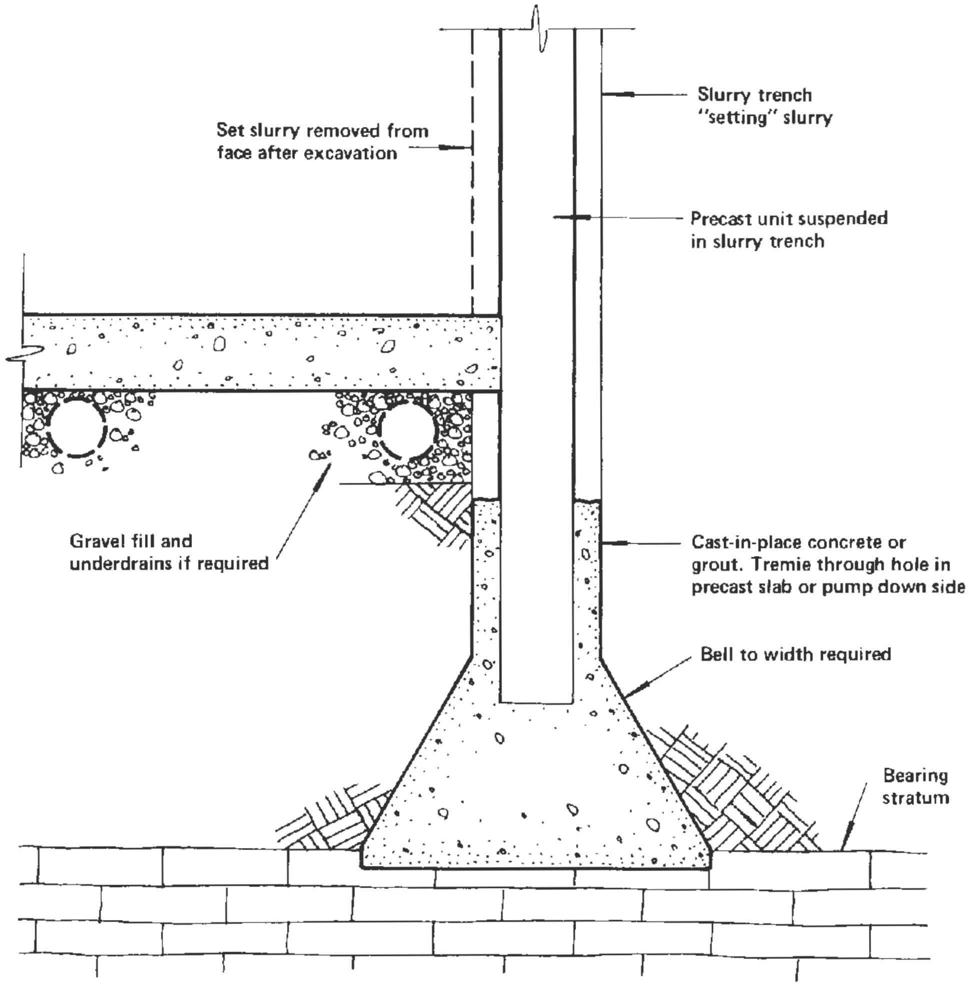


Fig. 47 Foundation for precast concrete load-bearing walls.

uplift loadings. Therefore, it may be desirable to design ways in which a continuous bearing wall can utilize the floor slab as a foundation also. Such a concept is illustrated in Fig. 50.

B. RESISTING HYDROSTATIC UPLIFT.

On many construction sites, the hydrostatic uplift caused by a high water table is a major design consideration. Depending on the soil profile and the location of the water table, it may be possible to reduce the hydrostatic uplift forces on the structure through any of a variety of methods.

1. If an impervious stratum exists within a reasonable distance from the floor of the tunnel, the slurry wall (if that is the method to be used) can be extended to this impervious layer and actually be anchored in it. The slurry itself will act as a water cut off and for all practical purposes eliminate the hydrostatic uplift on the bottom of the tunnel. Normally such an impervious strata will also have a very high allowable bearing and can be used to found the structure, also.

2. Various methods of grout injection below the base of the tunnel can be used to reduce the effects of hydrostatic pressure.

3. The hydrostatic pressure can be relieved through the use of underdrain systems as illustrated in Fig. 47.

4. The cast-in-place floor slab can be designed as a structural member with rock or soil anchors at various points to "hold down" the structure.

5. The most common method of resisting the hydrostatic uplift is through the dead weight of the structure itself. In the designs being proposed in this report, particularly those in which there is no backfill over the tunnel roof, the additional required dead load must be provided in the floor slab and may require an unreasonably thick concrete floor. Again, some detail at the base of the structure must be provided to transmit the load from the uplift to the load of the structure.

IX. CONNECTIONS

In any structure composed of prefabricated structural elements, the design of the connections is usually the most difficult and time consuming task. It is also the source of most of the troubles associated with this type of construction. Special attention must be given to not only the design aspects of the connection, but also to the details of the connection.

In cut-and-cover tunnel construction with prefabricated structural elements, the connection must perform one or more of the following functions:

1. Transfer vertical loads from horizontal members to vertical load bearing members.
2. Transfer vertical loads from vertical members to foundations.
3. Transfer horizontal loads from vertical members to horizontal thrust resisting members.
4. Transfer vertical loads from one horizontal member to another.
5. Transfer horizontal loads from one vertical member to another.
6. Distribute concentrated loads among members.

In addition to resisting the forces caused by the loads from external forces, the connection must also resist forces which are internally generated within the structure. These forces are caused by restrained volume changes in the members due to temperature changes, shrinkage, and creep.

In many respects the connection design and potential problems associated with connections will be less for the type of underground structure considered here than for the typical above ground structure. This is because the soil surrounding the structure tends to provide overall stability. In most cases the deformations caused by the volume changes mentioned above will be small enough that the stability of the soil behind the structure is not impaired. In many cases soil/structure interation can be of overall benefit. The surrounding soil also provides a condition of temperature and humidity uniformity that reduces thermal cycling and shrinkage strain.

The most desirable type of connection for transferring loads from horizontal members to vertical load bearing elements is to bear the horizontal member directly on top of the load bearing member, as shown in Fig. 48. In many cases this is obviously impossible, that is, when the horizontal members come in at an intermediate height on the vertical member. In these cases the ideal situation would be to design a reinforced concrete haunch. (Fig. 48). However, if the vertical elements are placed within a narrow slurry trench or in a hole which is very little bigger than the precast element, there may not be adequate clearance for a reinforced concrete haunch to be added to the member before placing in the excavation. In these cases, provision must be made for adding the haunch after excavation has proceeded to the point where the haunch is needed. For these situations either a precast concrete, a cast-in-place concrete, or a steel haunch can be used, but the connection of the haunch to the precast vertical element is sometimes a rather difficult design and detail condition. Two examples of such a connection are shown in Fig. 49.

A. DESIGN OF CONNECTIONS.

In the case of precast concrete elements, much work has been done in recent years on the design methods for connections. Various design philosophies are presented in the PCI Design Handbook and the PCI Manual

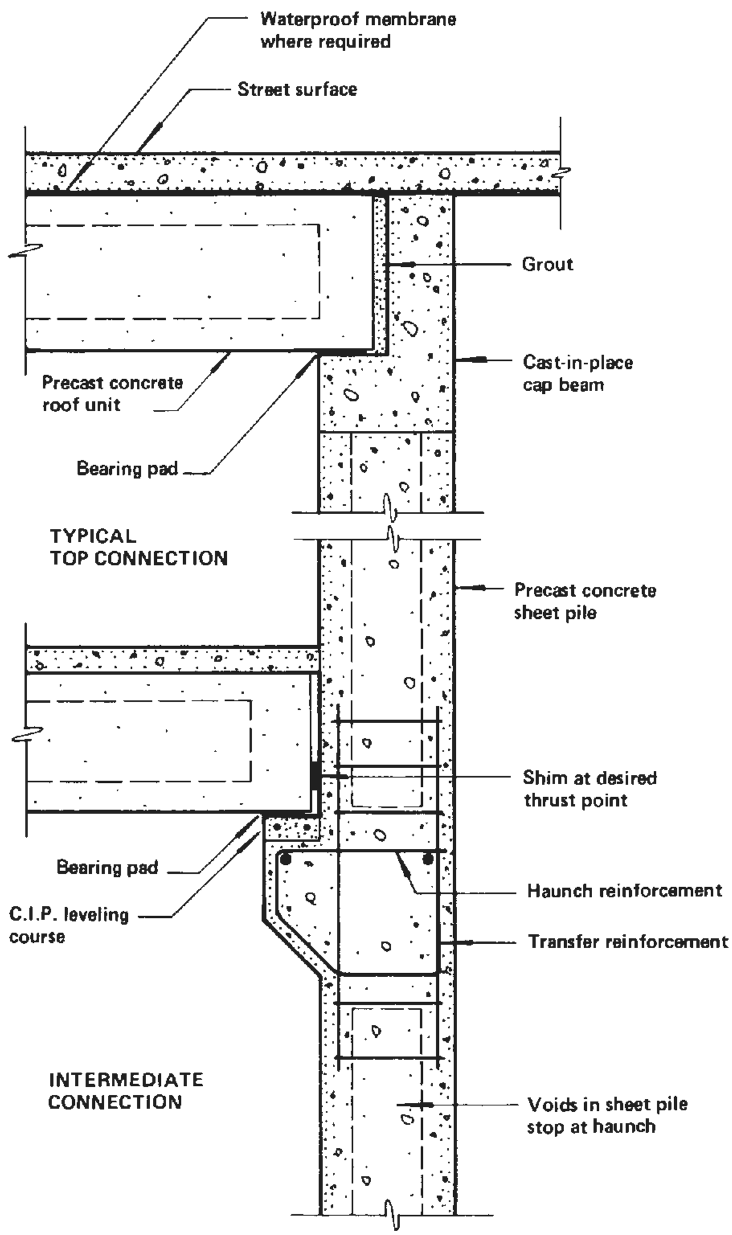
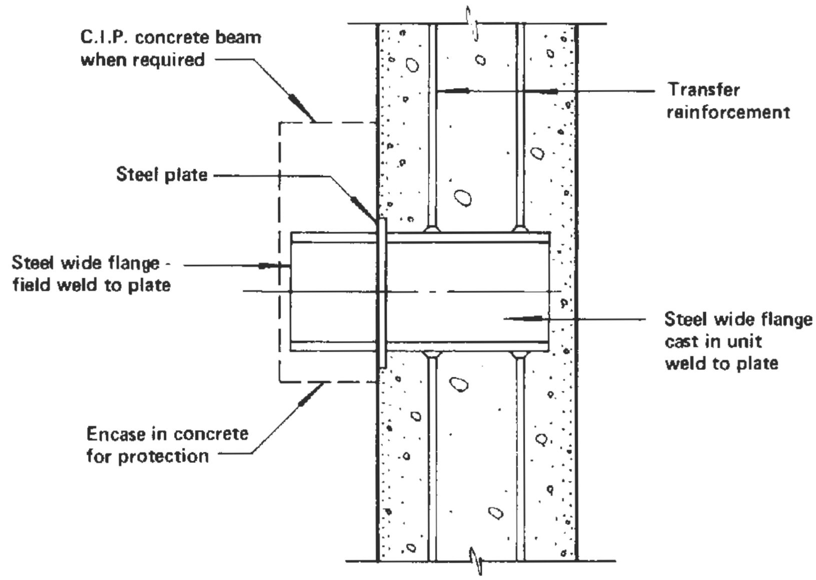


Fig. 48 Examples of connections.



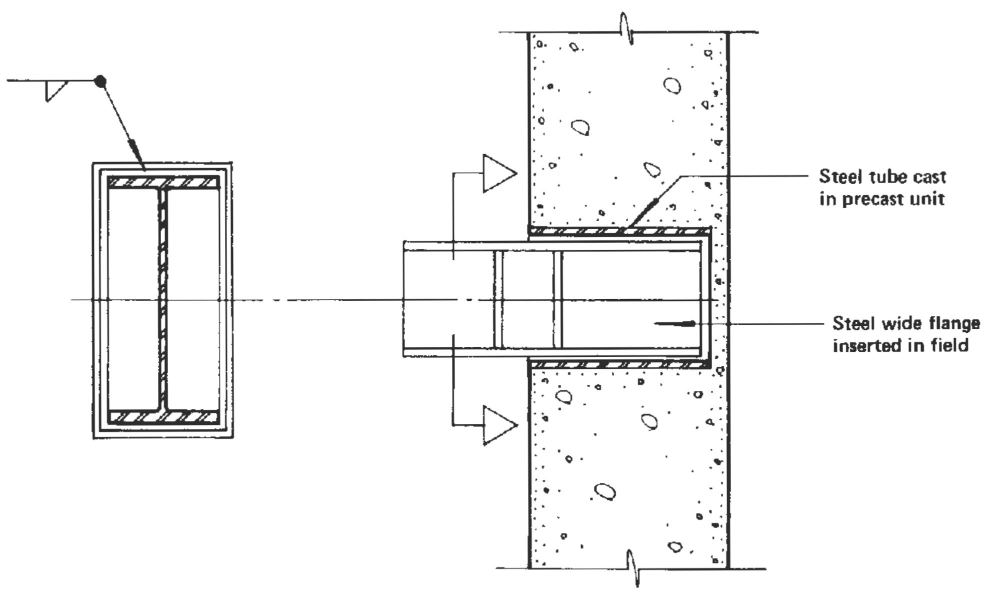


Fig. 49 Examples of connections using haunches which are constructed after the vertical element is in place.

on the Design of Connections. Design of the connections follows these steps:

1. Determine the magnitude and direction of each load which can come to the connection.
2. Visualize the possible modes of failure; i.e., crushing of concrete due to compression, cracking of concrete due to shear or tension, yield of reinforcing bars or other hardware devices due to tension, shear of steel hardware, or weld failures.
3. Provide means for resisting the loads to prevent the failure. If two or more failure modes can occur simultaneously, such as tension and shear, the combined effects should be investigated. The reference cited above suggest the following equations for determining the combined tension and shear capacity:

   a. For concrete failure modes:

   $(P/P')^{4/3} + (V/V')^{4/3} \leq 1$

   b. For steel failure modes:

   $(P/P')^{2} + (V/V')^{2} \leq 1$

   Where P and V are the applied tension and shear, respectively, and P' and V' are the tensile and shear capacities, respectively.

   Normally, concrete should not be relied upon for resistance in direct tension, and reinforcing bars should not be relied upon for resistance in direct shear. However, reinforcing bars can be incorporated into shear connections by the use of the shear-friction concept. This relatively new design tool is explained in the references cited above.

   Placing tolerances of prefabricated members should be given full consideration in the design of connections. For example, in haunch or corbel type connections, a significant difference in calculated moments can result if the load eccentricity is not properly estimated. For this reason it is desirable to use larger factors of safety in the design of connections than may be required for the design of the elements themselves.

B. WATER-TIGHTNESS.

The extent to which special details for water-tightness are necessary at the connections, is dependent on the type of soil, highest position of the water table, the method of construction and the degree of water-tightness necessary. In granular soils with a high water table, pressures on joints can be quite high, and special high-head water stops or joint details may be desired. In these cases, it is often more economical to allow some controlled leakage, and provide a means for draining or pumping out any water which may accumulate.

Diaphragm walls, whether prefabricated or cast-in-place inherently have a high degree of water-tightness provided by the bentonite slurry as discussed in Section III of this report. Experience has shown this water-tightness to be permanent.

In practice, it may be desirable to provide several methods of preventing or controlling leakage in order to take care of any foreseeable contingencies. Such a detail is shown in Fig. 50.

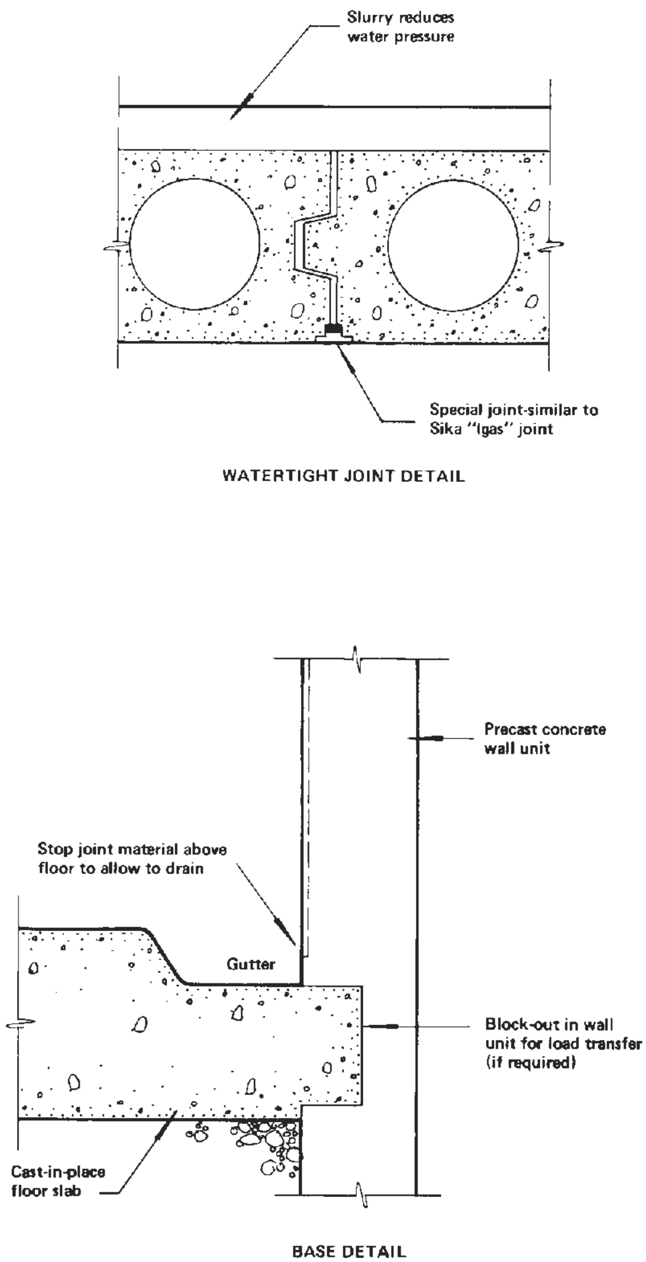


Fig. 50 Example of vertical wall joint designed to prevent or control leakage.

X. QUALITY COMPARISONS

Prefabrication of structural elements offers some opportunities for improved quality control, particularly when comparing precast concrete wall elements placed in a slurry trench with tremie-cast concrete placed in slurry. For purposes of this discussion, quality will include structural integrity, appearance, and water-tightness.

A. STRUCTURAL INTEGRITY.

In a tremie-cast slurry wall several areas which may affect structural integrity are difficult to control effectively. These are discussed below:

1. Concrete quality. In concrete placed by tremie in a fluid-filled trench or hole, the concrete must have a high slump in order to behave properly. Low-slump mixes can block the tremie pipe; since no means of mechanical vibration is possible, the mix must be free flowing. These high slumps--usually seven to eight inches, require more water, and, consequently, more cement to maintain a given strength level, although certain admixtures have proven beneficial in making the concrete flow more freely. It should be emphasized that there is no difficulty in obtaining satisfactory strengths--3000 to 4000 psi--in cast-in-place slurry walls. However, control of the quality of concrete mixes and placing techniques of precast panels cast in a flat position are obviously better.

   A second concern regarding concrete quality is the possibility of contamination of the concrete by the bentonite fluids and suspended sand accumulated in the trench. Generally, the intermixing of the bentonite and concrete is no problem because of the difference in specific gravity of the two materials. Some experienced engineers suggest that the difference in specific gravity should be at least 1.0. Since concrete has a specific gravity of 2.3, the fluid should be less than 1.3. The most economical and best performing bentonite slurries have specific gravities of around 1.1 or less, so this is normally not a serious matter. It is, however, an item that requires field control.

2. Bond of reinforcing steel. This is a consideration that is usually of much concern to structural engineers in designing slurry walls. Experience and a limited number of tests have shown that it is not a problem of overriding concern. Some tests have apparently shown no loss of bond strength caused by bentonite coating of bars, while others have indicated a loss of up to 50%. Usually, critical design sections are located where bond is not a problem. When splices in load carrying bars are necessary, most experienced engineers will use a development length of 1.5 to 2.0 times that normally required. In some cases, hooks or other mechanical anchorage devices can be employed.

3. Placement of reinforcing steel. In cast-in-place slurry walls, reinforcing cages are normally fabricated on the ground, and then suspended in the bentonite slurry while the concrete is tremie-placed around them. Maintaining the proper location of the bars requires good bar-tying techniques and control procedures. In order to assure that the bars are within reasonable tolerance of their design position, the cages must be well braced to avoid distortion during handling and concrete placing. Spacing of the bars

away from the trench sides so that there is adequate cover is accomplished by a variety of means, and is usually effective. It is quite apparent that this item is considerably more difficult to control in cast-in-place walls than in precast ones.

4. Dimensional control. In cast-in-place slurry walls, the thickness of the wall is usually controlled by the width of the excavation tool used. In some soils, when the fluid-filled trench is left open for a few days, some tests have shown that there may be a "squeezing" or distortion of the trench width. The amount is so small, however, that it probably never results in a structural problem of the wall itself. It is of more concern regarding the possible settlement of adjacent structures, in which case the effect is no different for a precast wall or a cast-in-place wall.

It is quite apparent when considering the above items that control of the structural integrity of a cast-in-place slurry wall is considerably more difficult than a comparable prefabricated wall. It should also be remembered that the final product in a cast-in-place wall is not available for inspection until after the excavation adjacent to it. Even then, such items as the improper placing of reinforcing and excessive coating of bars may be difficult to detect, and even more difficult to correct if it is detected. Since one purpose of safety factors is to allow for uncontrollable deviations from design, it is suggested that a significantly lower factor of safety is applicable in the design of prefabricated wall elements.

B. APPEARANCE.

A formed concrete surface will always be an exact reflection of the surface it is cast against. The appearance of the surface of a concrete wall cast against earth will be poor. In addition, concrete walls cast in a slurry trench may have relatively large "bulges" or uneven surfaces caused by soft pockets adjacent to the intended excavation.

Precast elements, on the other hand, will ordinarily be cast flat in steel molds. If the details of the member are such that this steel formed surface can be the surface that is exposed, the appearance can be quite good. The top surface (in the casting position) will normally be finished by hand or machine, and the appearance is a function of the skill and care of the finisher, or the equipment being used. It is also possible to apply special architectural finishes to precast units such as embedded tile or exposed aggregate. These finishes are quite costly, however, and unless the finish is one that is particularly desirable architecturally, the advantages of this vs. field applied finishes is open to question.

The acceptance criteria for appearance is, of course, dependent on the use of the wall element. For subway tunnels in mass-transit systems that are not normally seen by the public, the earth-formed surface may be satisfactory. In highway tunnels which are exposed to motorist view, such surfaces would obviously not be satisfactory, whereas a steel-formed surface, perhaps painted to improve reflectivity, may be. The top surface of a precast element, if finished with no more care than ordinarily received in a structural precasting plant, would probably require additional surface treatment in a highway tunnel. The most severe appearance criteria is for areas regularly exposed to pedestrian viewing, such as in subway stations. It is doubtful that precast walls would be satisfactory here without special treatment.

The joints between prefabricated wall elements can also be an appearance problem. Properly designed, the joints need not be distracting, and

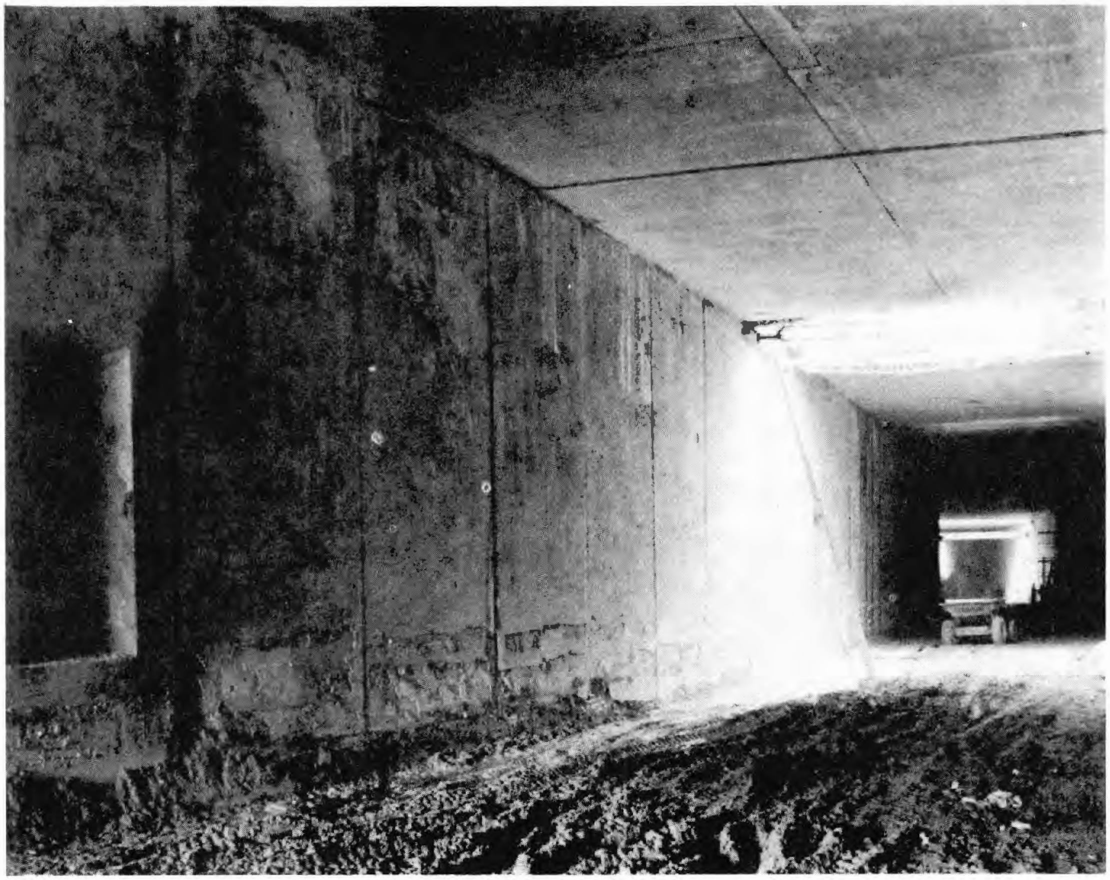


Fig. 51 Precast concrete wall panels before any finish is applied. (Courtesy Soletanche and Rodio, Inc.)

can even be architecturally pleasing. However, placing of adjacent units must be very precise, unless the joint design is such that some placing tolerance can be accommodated. This can usually only be accomplished if adjacent units are intentionally not in alignment at the exposed surface, such as in some of the king pile schemes suggested in Section IV of this report.

C. WATER-TIGHTNESS.

The problem of designing watertight joints has often been cited as a major deterrent to the use of prefabricated panels for walls of tunnels. Suggested joint designs for minimizing or controlling water passage through these joints are shown in Section IX.

Cast-in-place slurry walls also have joints between adjacent cast panels, and numerous joint designs have been used with variable effectiveness. While actual experience with prefabricated panels is limited, it would seem that control of water passage through joints between prefabricated elements would be no more difficult than in cast-in-place walls. Because of size and weight limitations, joints would be more frequent in prefabricated construction.

Leakage at points other than joints is much more likely in cast-in-place walls than in precast walls. Precast units will usually have a higher quality, less permeable concrete, and if they are prestressed, random or flexural cracking is virtually eliminated although some longitudinal cracking in prestressed, voided slabs may occur. It is also possible that location of leakage problems could be more difficult to pinpoint in a cast-in-place wall. Because of the concrete quality, inadequate compaction around reinforcing bars, or voids left by a bentonite coating around the bar, horizontal movement of water may occur. Thus the water may appear at a different place than where it enters, making repair more difficult. Whether the wall is cast-in-place or prefabricated, some maintenance to maintain watertightness should be anticipated.

XI. MINIMIZING SURFACE DISRUPTION

Disruption of the surface environment is a major disadvantage of cut-and-cover tunneling in urban areas. One of the most effective ways of minimizing this surface disruption is by carrying on most of the construction operations under a roof. This method is used in most subway construction today, with the "roof" consisting of temporary wood decking.

Highway tunnels are especially adaptable to "under-the-roof" construction because: 1) they are usually relatively short and relatively shallow, 2) they normally are constructed in a straight line from the entrance to the exit and 3) by the nature of their final usage, the entrances are accessible from the surface.

The use of prefabricated structural elements for horizontal members at street level, along with slurry walls for the permanent, load bearing side walls of the tunnel offers unique opportunities for further minimizing this surface disruption. Schemes A and B described in this section show construction methods which enable a permanent restoration of the street level in the shortest possible time.

A. CONSTRUCTION PROCEDURES - SCHEME A.

1. Assumed Site Conditions. In order to illustrate methods of construction, it is necessary to make a number of assumptions regarding site conditions. For the first illustration the following assumptions were used:

    a. The construction is to be under an existing street in an urban business district. Existing buildings are office buildings or commercial establishments which impart medium to heavy surcharge loads to the tunnel walls.

b. Utility lines are relatively heavy in a direction parallel to the tunnel. Cross utilities are fairly light except at the intersections, as shown in Fig. 52. This condition is considered typical in a street as described above. In a street fronted by a large number of small businesses or row houses, cross utilities might be more numerous, and would perhaps need additional consideration in the selection of the construction method. Heavy cross utilities are very difficult to accommodate in slurry wall construction and this factor is extremely important in the selection of the construction method.

It is also assumed that in the final construction, utilities will be permanently located in a utilidor above the tunnel roof as shown in Fig. 63.

c. Soils are granular to a depth below the invert of the tunnel, but an impervious stratum is located a short distance below the invert of the tunnel. This stratum also provides firm bearing for the load bearing walls. The water table is relatively high--within six to ten feet below the surface. It is recognized that these soil conditions may be somewhat idealized, but methods of handling other types of soil conditions are discussed elsewhere in this report, and in other literature on slurry wall construction. These soil conditions were selected to avoid distracting from the main intent of this section, that is, the construction methods which will minimize surface disruption.

d. It has been previously determined that a single block, including one intersection, can be closed to all traffic for a single short period of time. It is assumed feasible to handle emergencies, deliveries, etc., from the side streets. This assumption pre-supposes agreement among fire and police departments, businesses fronting along the street, etc. This type of agreement obviously requires a great deal of prior coordination and public relations work, but it is believed that such agreements are possible. Alternative methods would usually require a much longer partial disruption.

2. Structural Elements.

The basic, final cross section of the tunnel is shown in Fig. 63. Structural design of the elements will not be shown in this section, but the design methods would follow the procedures illustrated elsewhere in this report.

3. Construction Procedures.

A step-by-step procedure of tunnel construction for a typical block is shown in Figs. 52 through 63. This procedure presumes that entrance construction is complete and access is available from the end. Some of the steps can be carried out prior to completion of the ends.

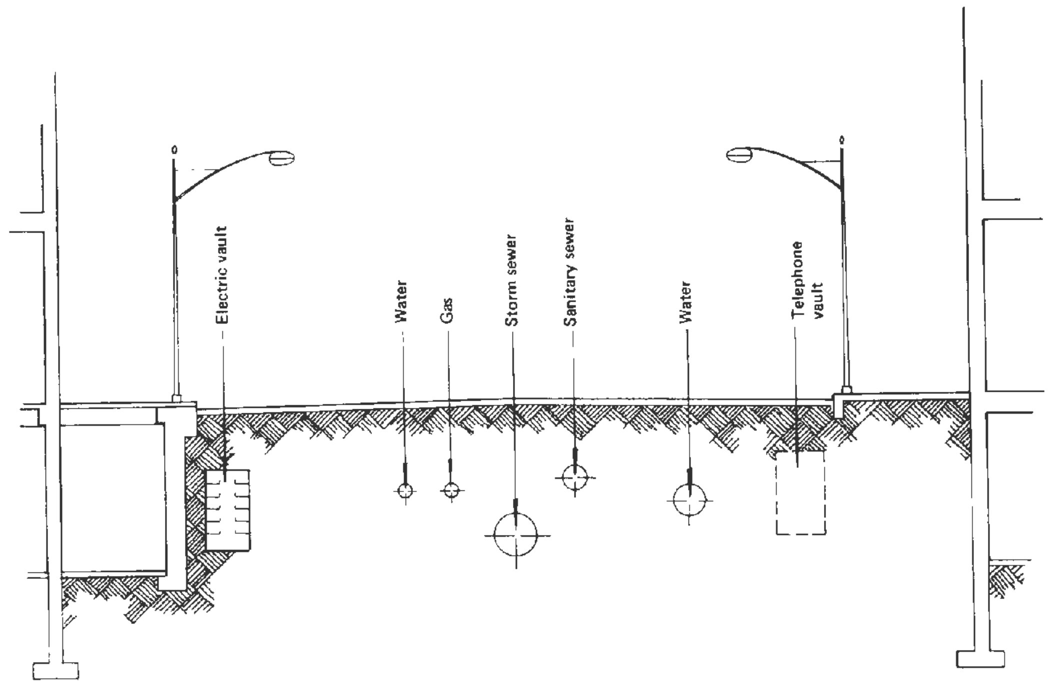


Fig. 52. Step 1: Identify all longitudinal and lateral utility lines. This step presumes that most utility locations are known from existing drawings, manhole locations, basement surveys, sonic probes, etc. It may still be necessary to positively verify some locations by probes or hand digging.

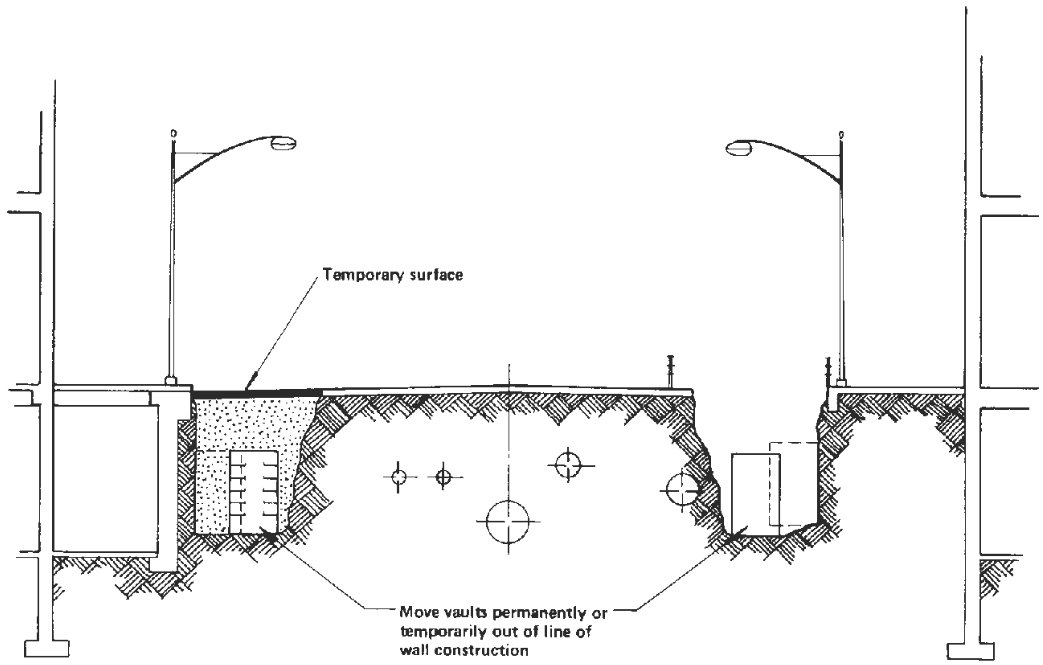


Fig. 53. Step 2: The relocation of any utilities which must be permanently moved outside the excavation, should be completed prior to the next step. In some cases, manholes and vaults may need to be moved into the excavation to clear the new wall construction.

Step 3: Complete construction around cross utilities (if required) by one of the methods suggested in Section B. (Note: In some cases this may be a part of Step 6 below, depending on the method selected.)

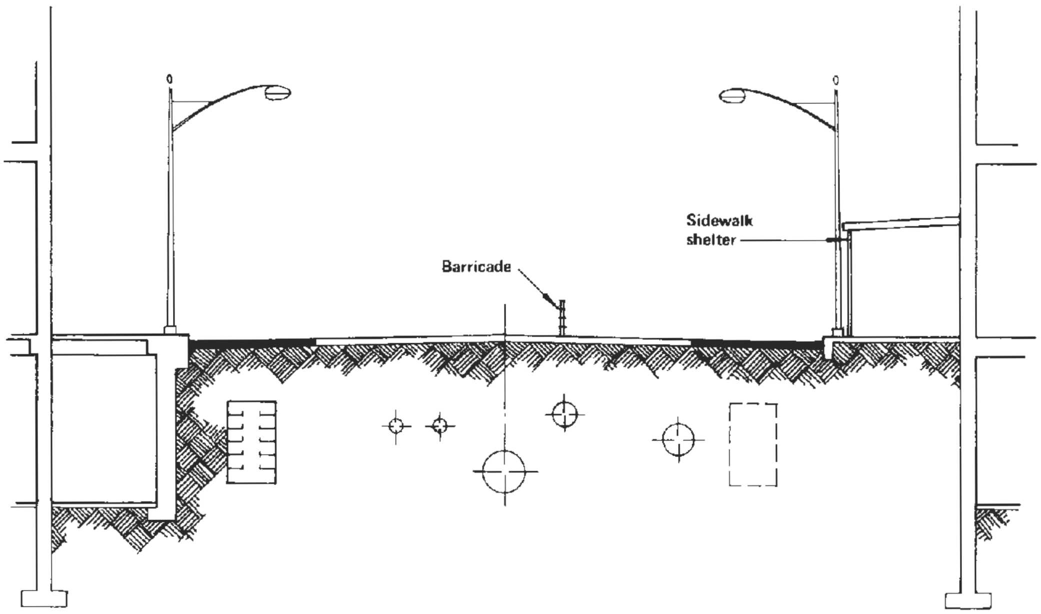


Fig. 54. Step 4: Erect sidewalk shelter and barricades on one side of street.

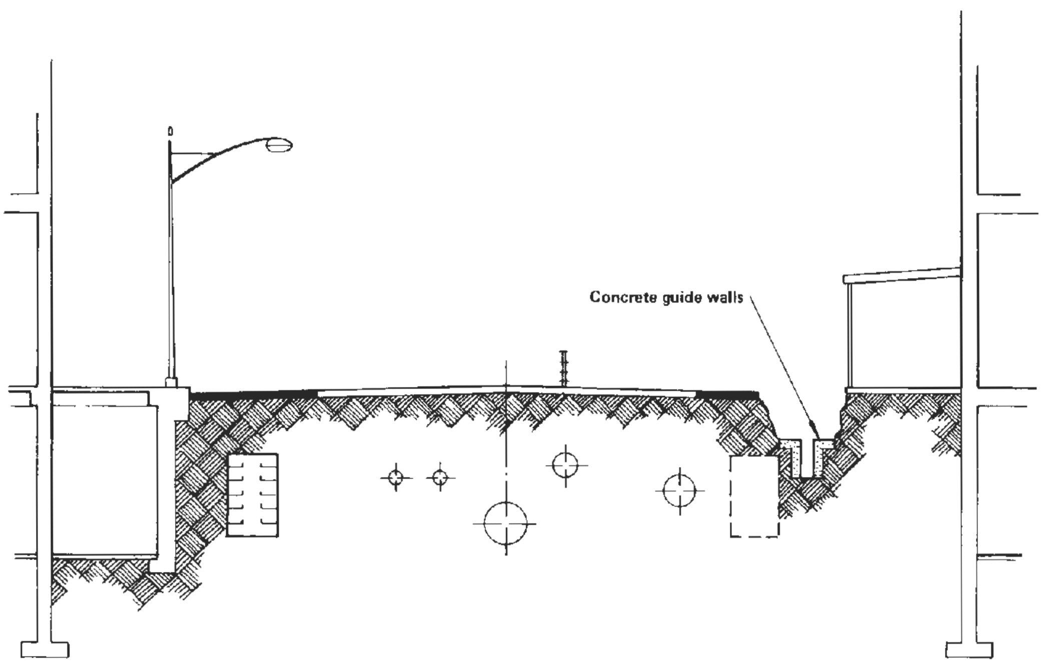


Fig. 55. Step 5: Cut off two lanes of traffic and construct guide walls for slurry trench.

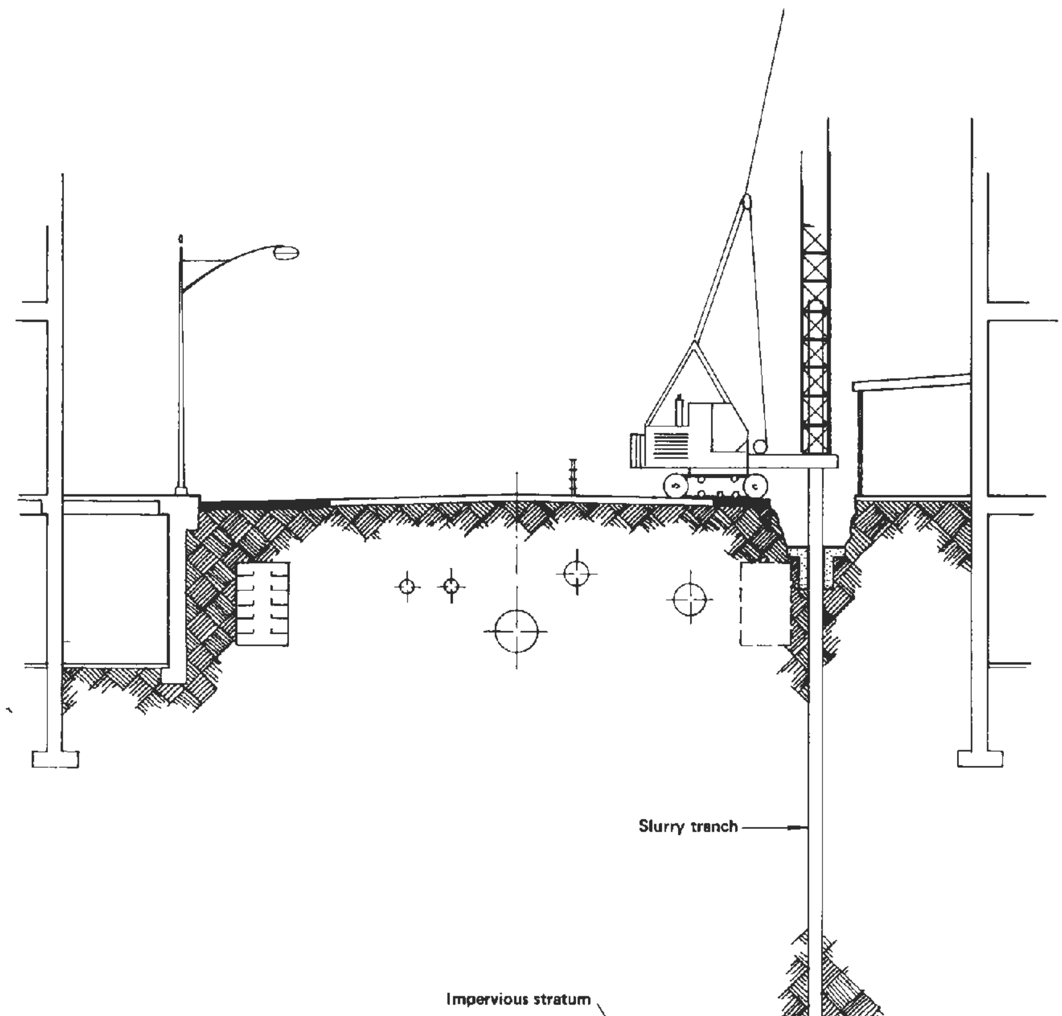


Fig. 56. Step 6: Dig slurry wall and install prefabricated vertical elements.

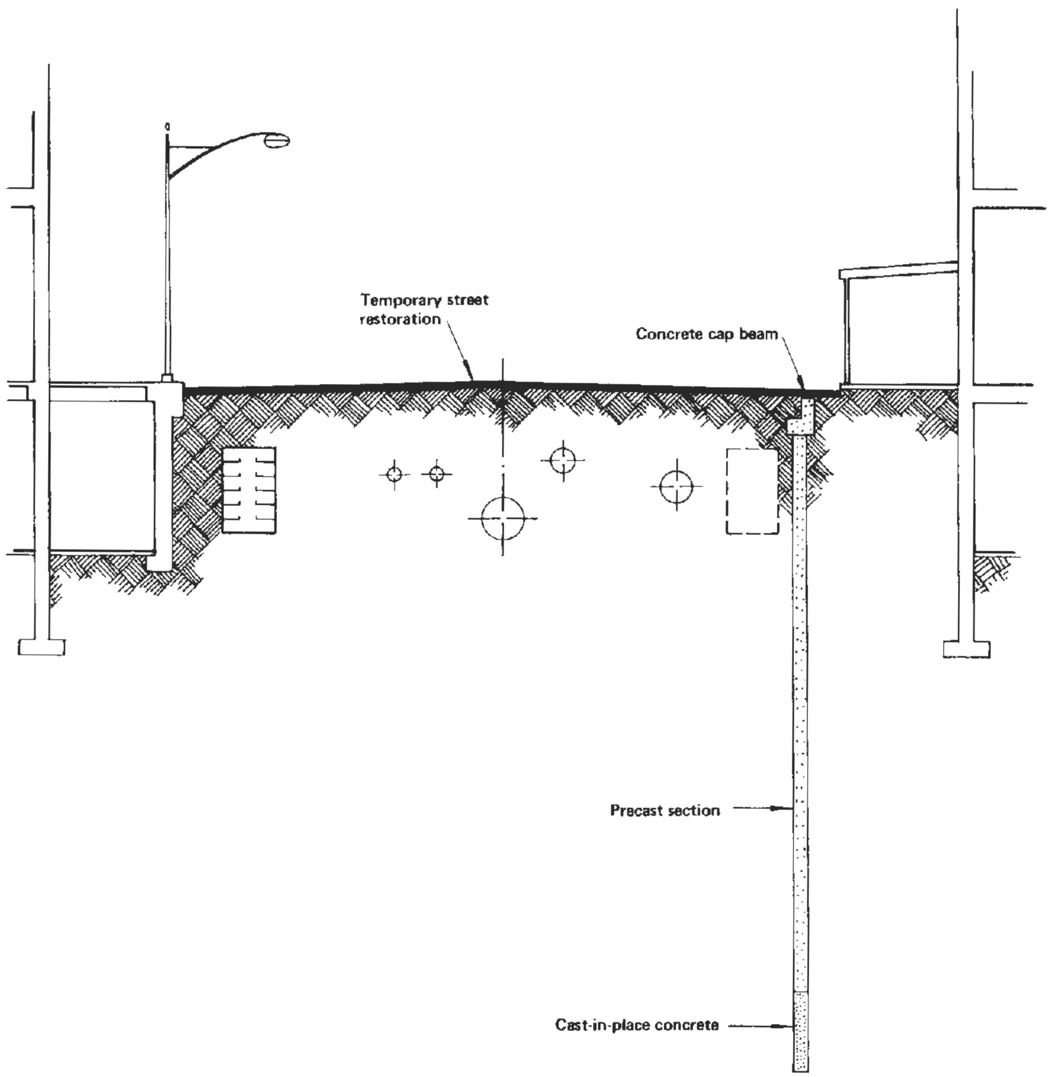


Fig. 57. *__Step 7:__ Cast cap beam on vertical element and restore traffic.*

__Step 8:__ Repeat Steps 4, 5, 6, and 7 for other side of street.

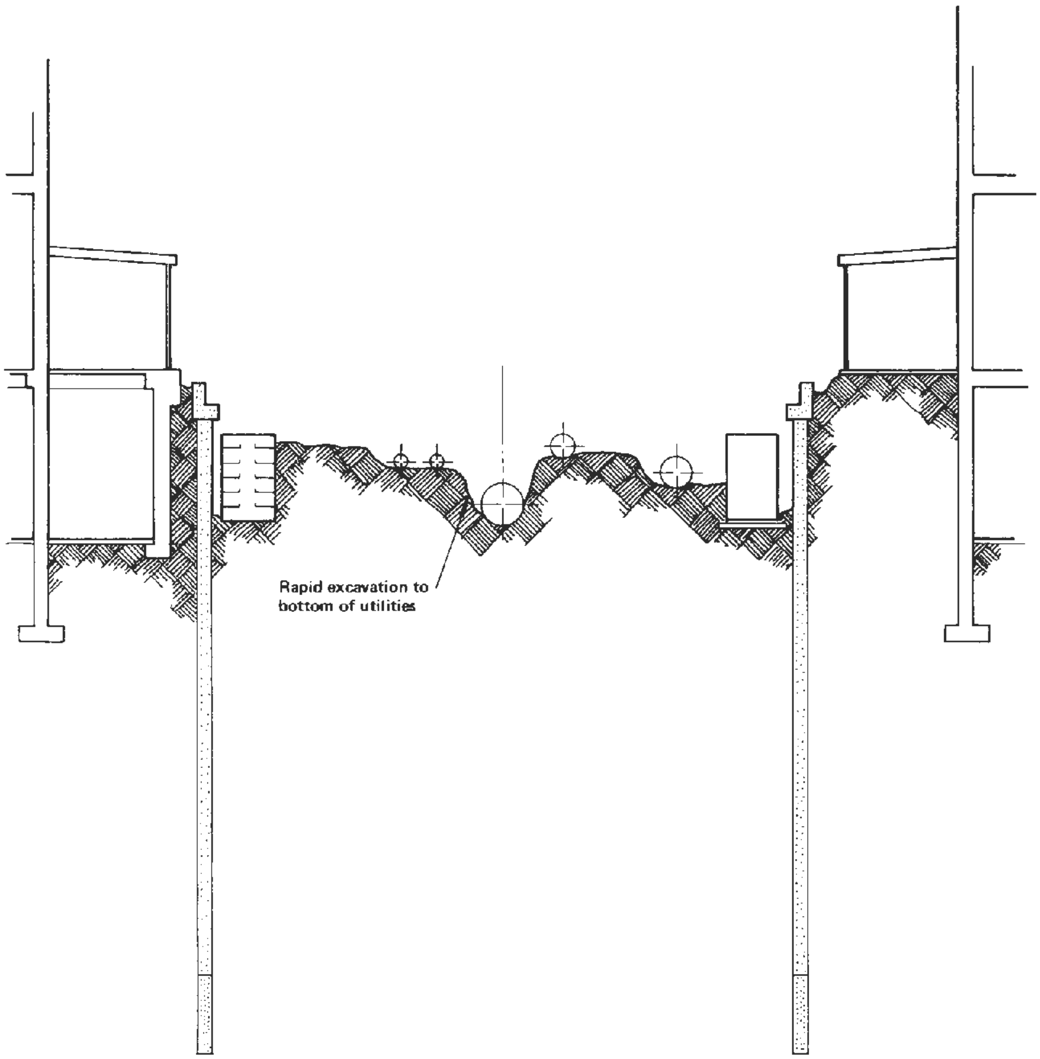


Fig. 58. <u>Step 9:</u> Close down street and rapidly excavate to underside of utilities.

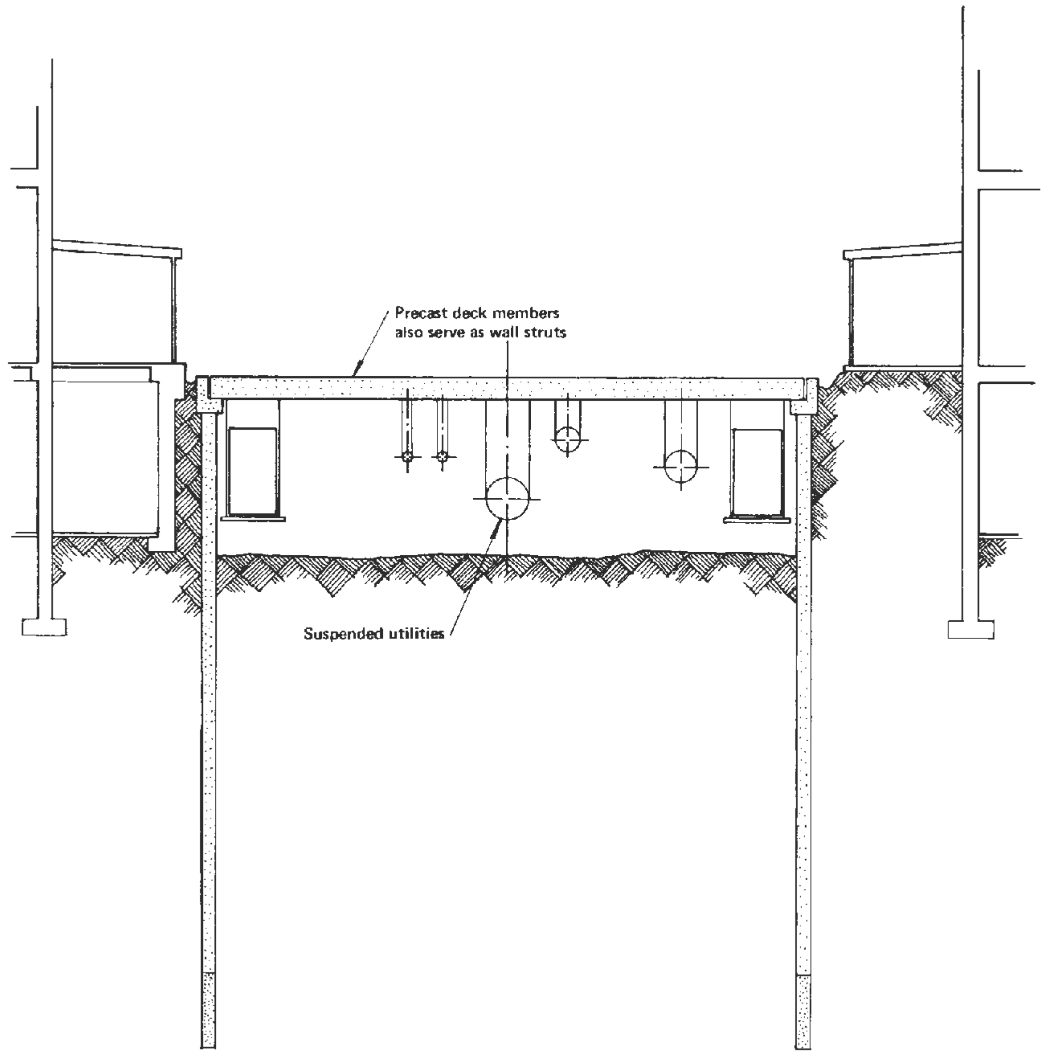


Fig. 59. Step 10: Set street level prefabricated units and suspend utilities.

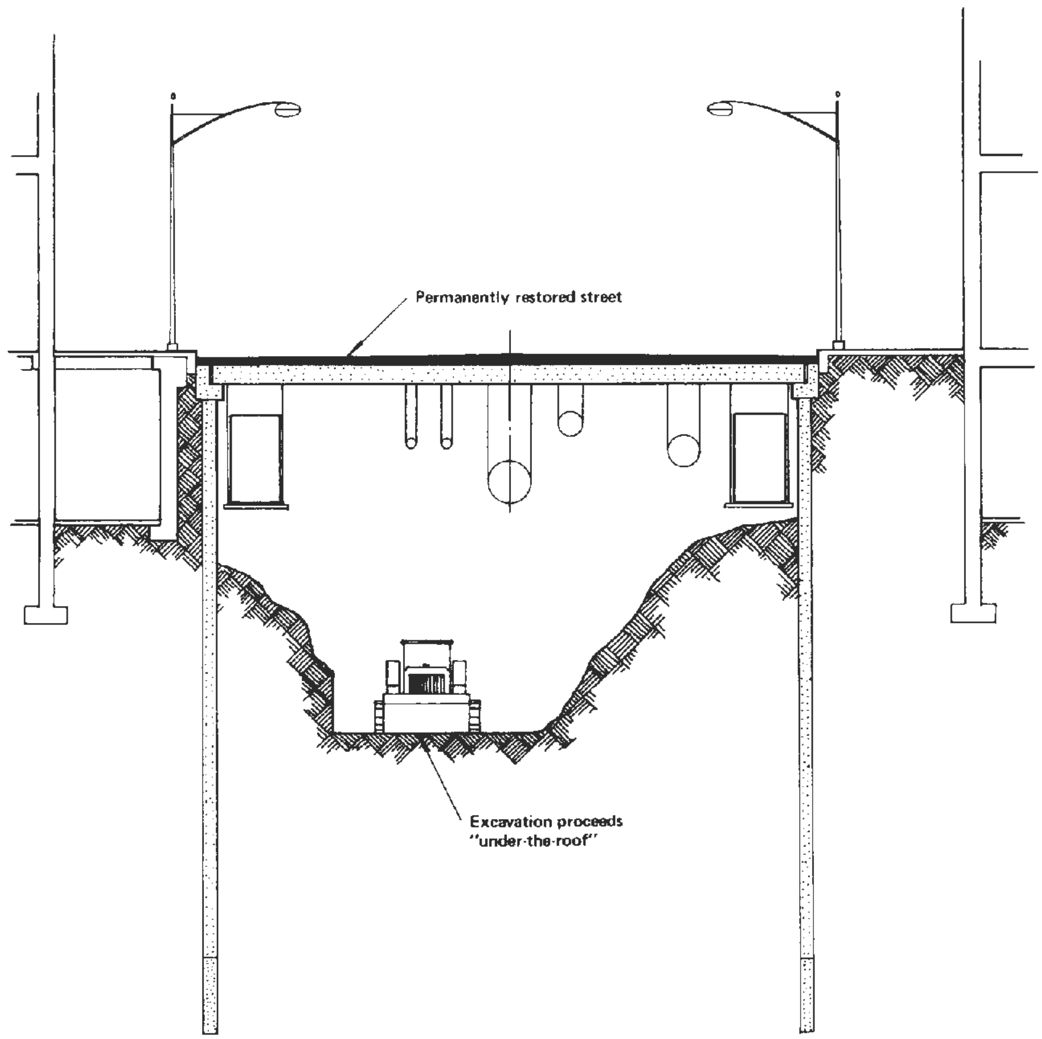


Fig. 60. *<u>Step 11</u>: Restore street and open to traffic. Note: 9, 10, and 11 can be carried out under accelerated scheduling to reduce the time that street is closed. <u>Step 12</u>: Excavate from below with front end loaders, dozers, and trucks.*

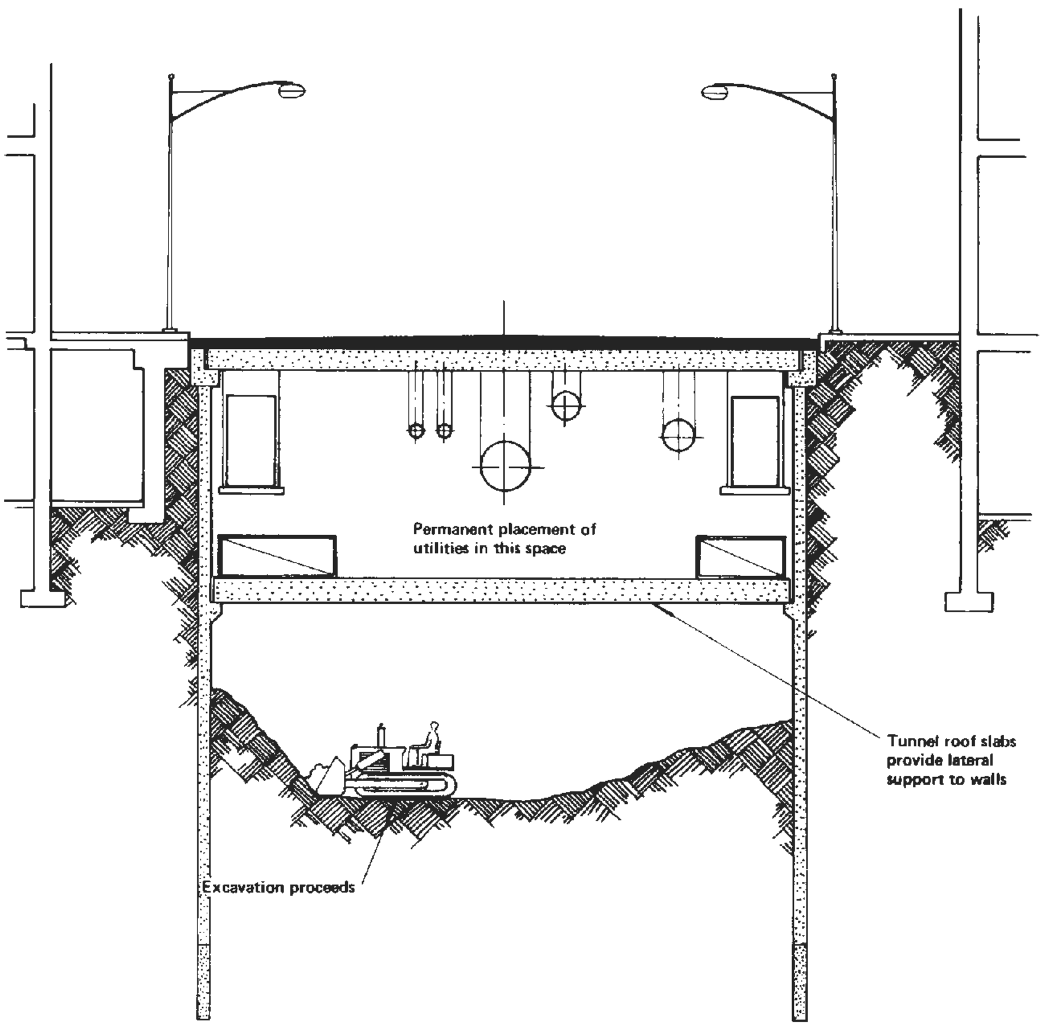


Fig. 61. *<u>Step 13:</u> Set tunnel roof slabs. Upper level is now available to utility companies for permanent placement of utilities in this space.*

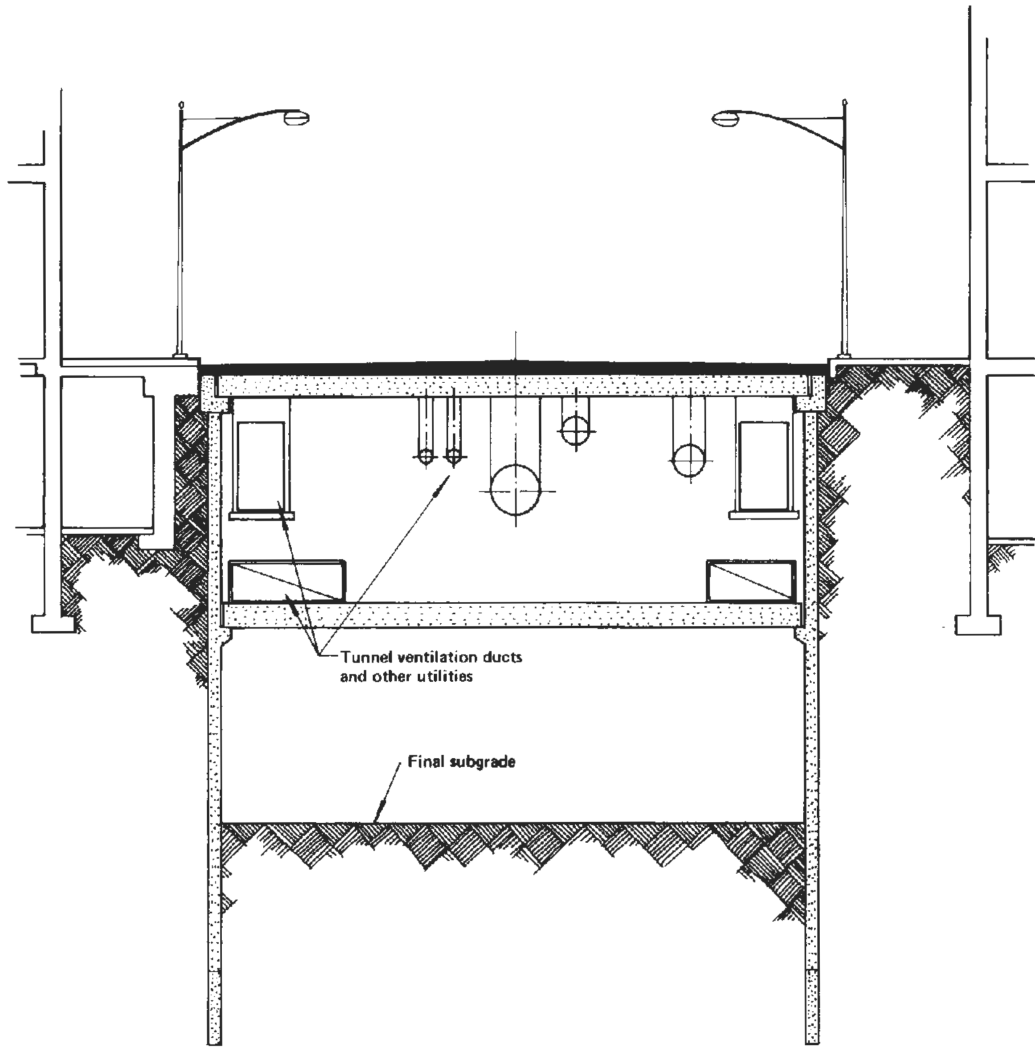


Fig. 62. *<u>Step 14:</u> Complete tunnel construction below second level.*

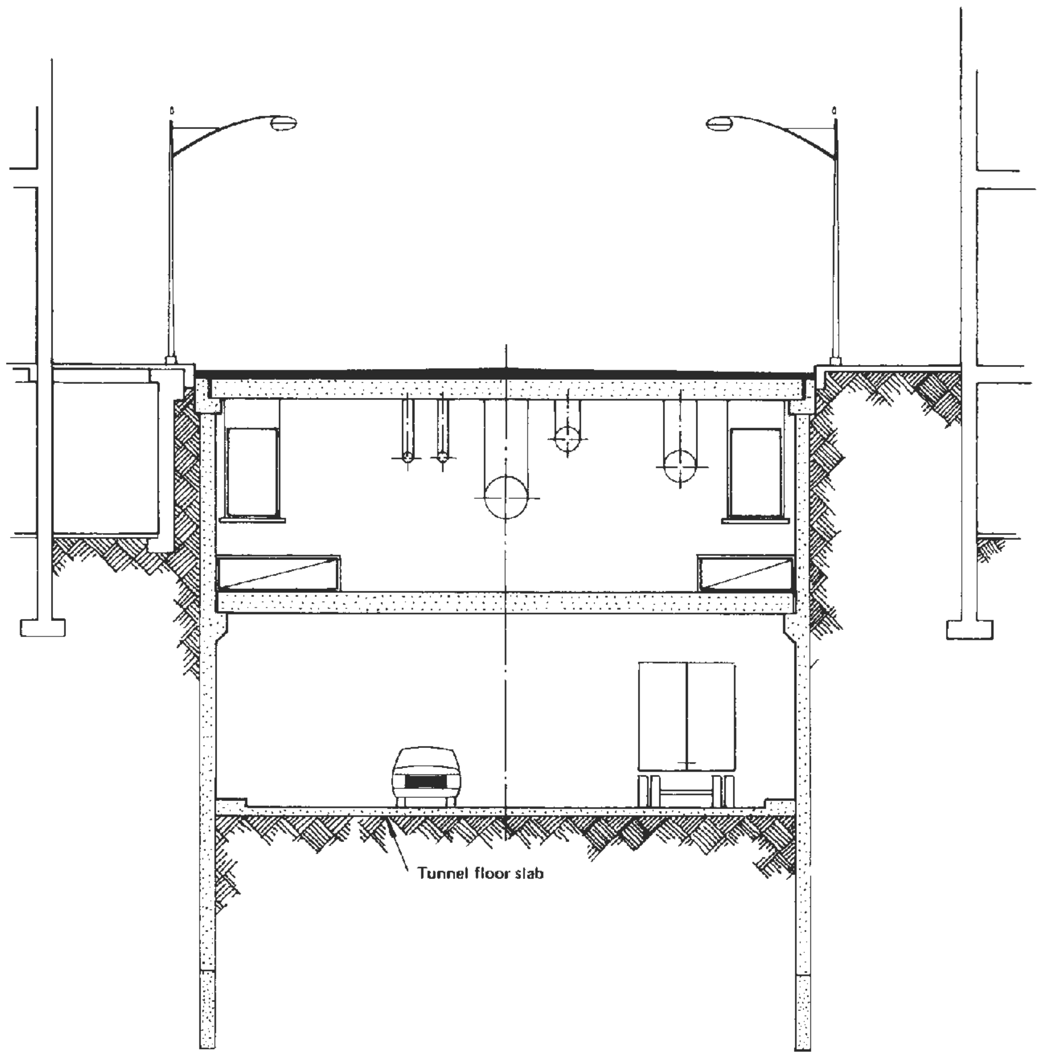


Fig. 63 Completed Structure.

It should be noted that major utility relocation is not undertaken before construction of the tunnel is started. In most conventional construction methods, this utility relocation is one of the most time consuming and disruptive activities of the construction process. Reducing this as much as possible is one of the aims of this construction procedure.

B. WORKING AROUND CROSS UTILITIES.

Cross utilities are one of the primary disadvantages of slurry wall construction, especially when prefabricated wall panels are used. The method of handling them will vary from site to site, but a few possibilities are mentioned here.

1. Single Utility Lines.

Depending on the size of the line, it may be possible to excavate a slurry trench under the line with the same equipment used for the rest of the trench (Fig. 64), or it may require additional equipment or hand excavation. In some cases it may be more feasible to use cast-in-place concrete. Since reinforcing cages are almost as difficult to move laterally as the precast panels, the engineer may wish to consider one of the special new concretes that exhibit high flexural strength. Some of these are described in Section XIII in this report and include fiber-reinforced and polymer modified concrete.

2. Multiple, Closely Spaced Utilities.

This situation will often occur at intersections. In some cases they may be handled by a series of small drilled piers so placed as to avoid the utility lines. These can be designed to retain the earth pressure. Ground water control would be accomplished by grouting.

In extreme cases, it may be necessary to use a completely different type of construction at the intersections. For example, a conventional soldier pile and lagging wall could be installed several feet away from the final wall line. The tunnel wall

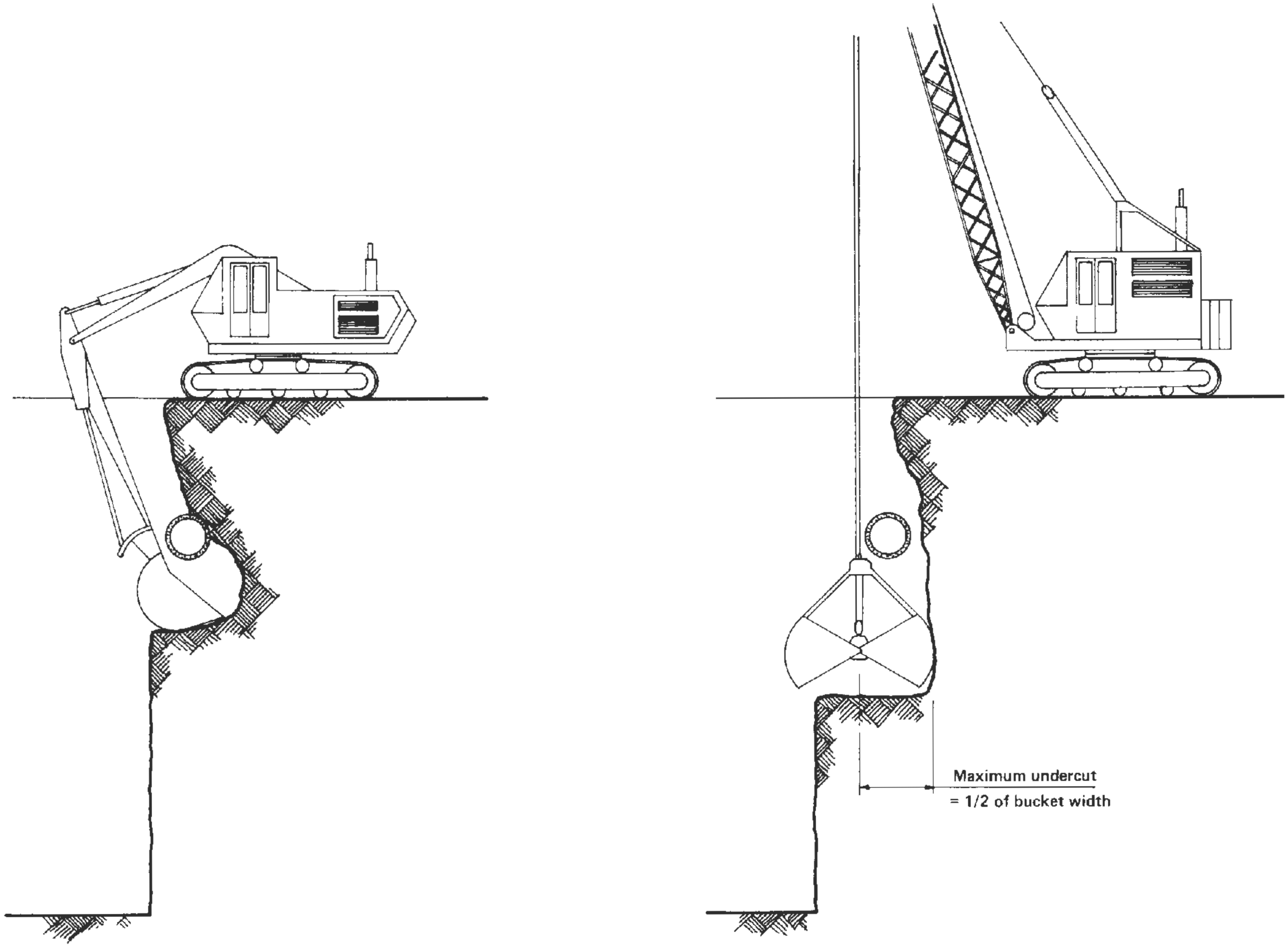


Fig. 64 Excavating under a cross utility.

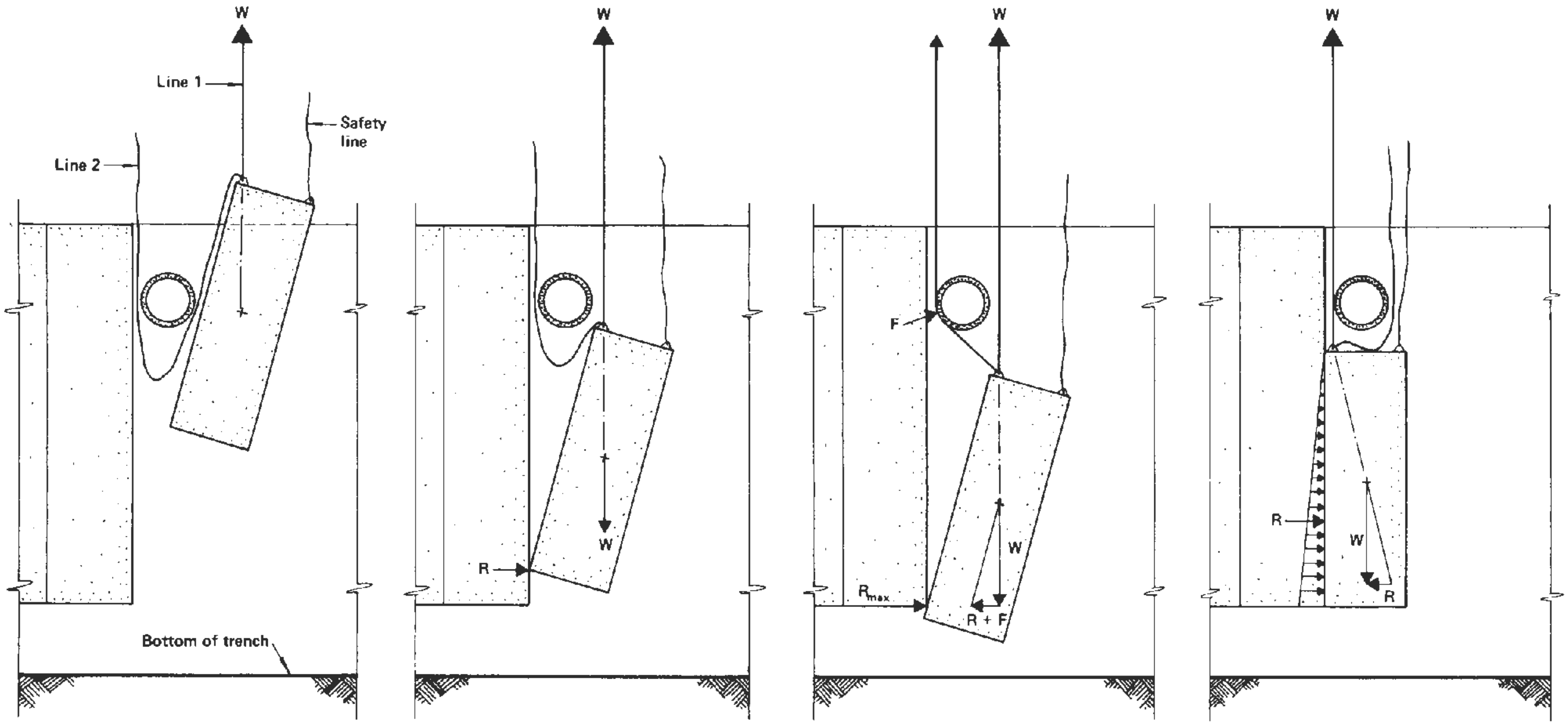


Fig. 65 *Setting prefabricated panel under a cross utility*

a *Panel is lowered into trench by line 1. Line 2. is attached, but slack.*

b *When panel touches adjacent panel, a reaction, R, starts to build up.*

c *As panel is lowered, weight is transferred to line 2, reaction R builds to maximum.*

d *Panel is raised to proper position by line 2. Reaction is distributed as shown.*

would then be conventionally formed and poured. This may involve lowering the water table and some underpinning of nearby structures may be required.

C. CONSTRUCTION PROCEDURES - SCHEME B

1. Assumed Site Conditions.

The site conditions for this scheme are assumed to be the same as Scheme A, except that in this case it has been determined that at least a portion of the street must be kept open to traffic continuously.

2. Structural Elements.

The structural elements are as shown in Fig. 25.

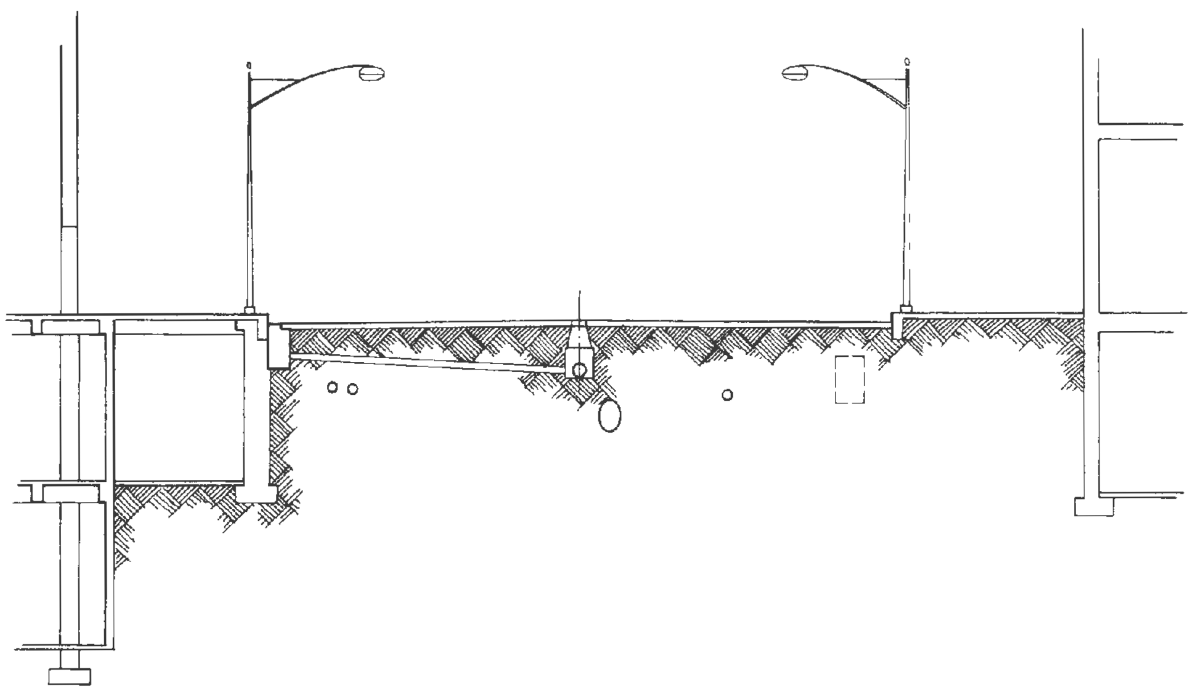


Fig. 66 Construction Procedures, Scheme B. Assumed site conditions.

3. <u>Construction Procedures.</u>

Construction procedures are described in Figs. 66-71. Steps 1 through 4 are the same as in Scheme A. (Figs. 52-54).

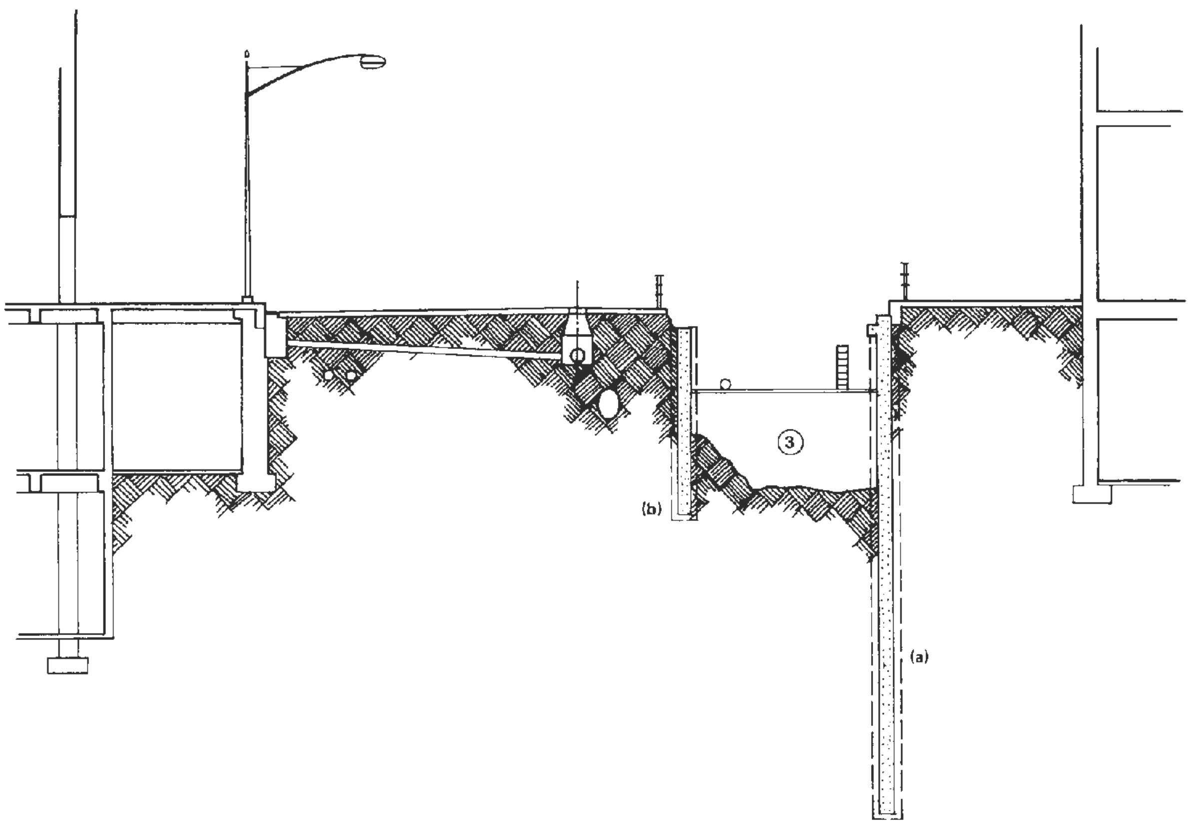


Fig. 67. *<u>Step 5:</u>* *Construct guide walls for slurry trench (a).*

<u>Step 6:</u> *Construct slurry trench (a), insert precast panels.*

<u>Step 7:</u> *Construct slurry trench (b), insert precast panels.*

<u>Step 8:</u> *Excavate to expose utilities in cell 3.*

<u>Step 9:</u> *Place cover on cell 3 - open to traffic.*

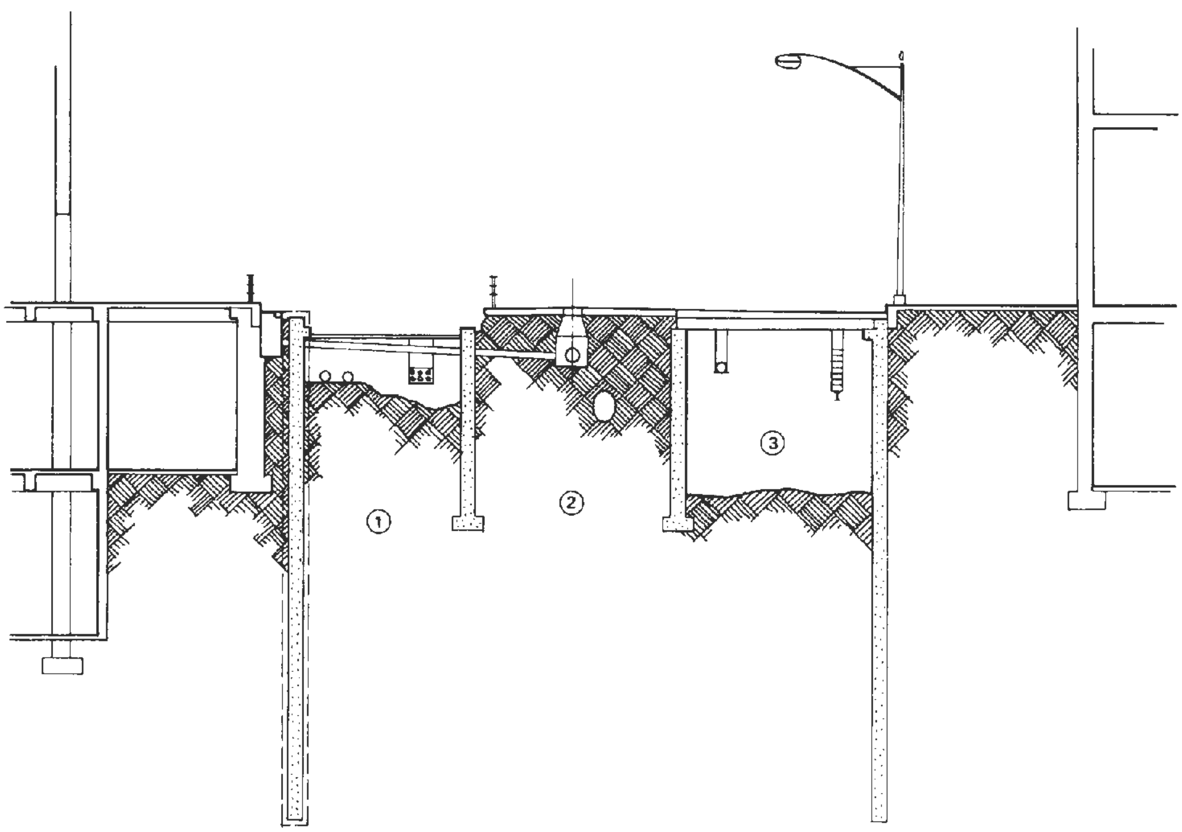


Fig. 68. *Step 10:* *Close traffic over cell 1 .*

Step 11: *Repeat Steps 6 thru 9 for cell 3 .*

Step 12: *Suspend or relocate utilities in cell 3 .*

Step 13: *Additional excavation in cell 3 - leave enough to support interior wall. Dewater as required.*

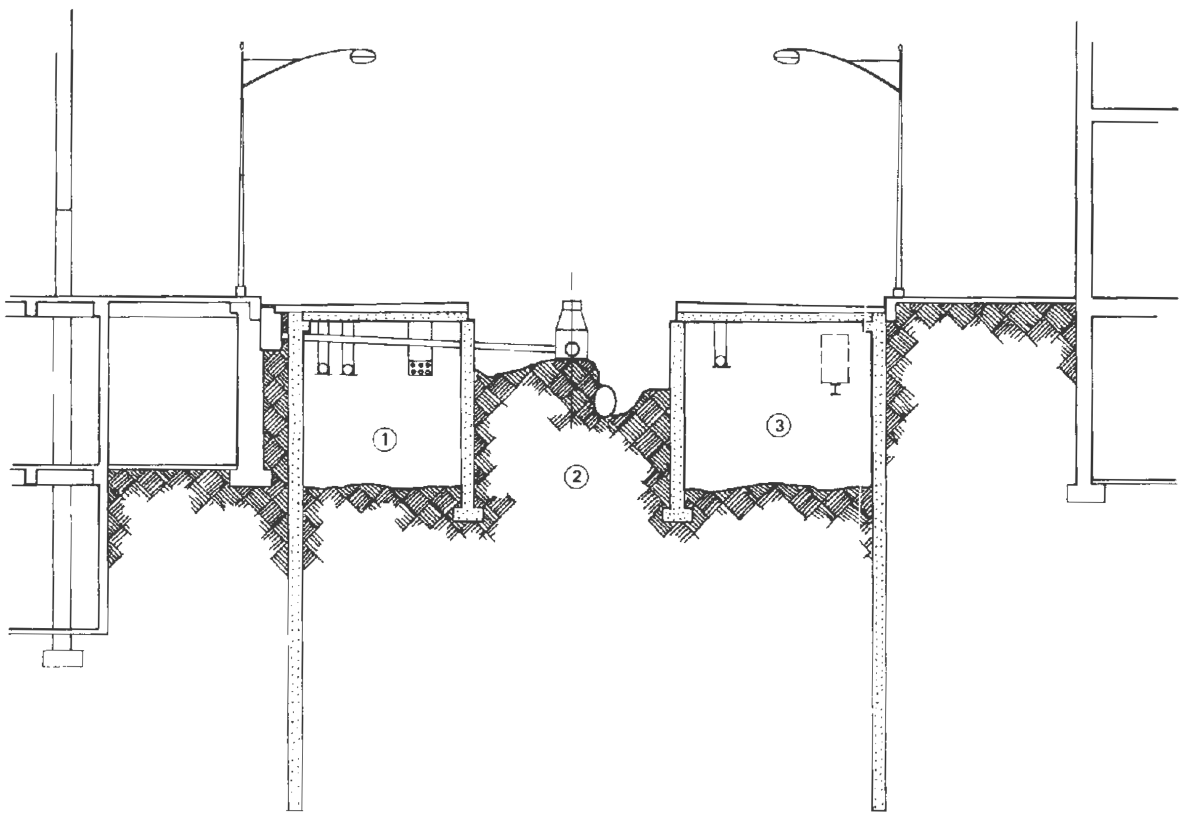


Fig. 69. *<u>Step 14:</u>* *Close traffic over cell 2 .*

<u>Step 15:</u> *Repeat steps 6 thru 9 for cell 2 .*

<u>Step 16:</u> *Repeat steps 12 thru 13 for cell 1 .*

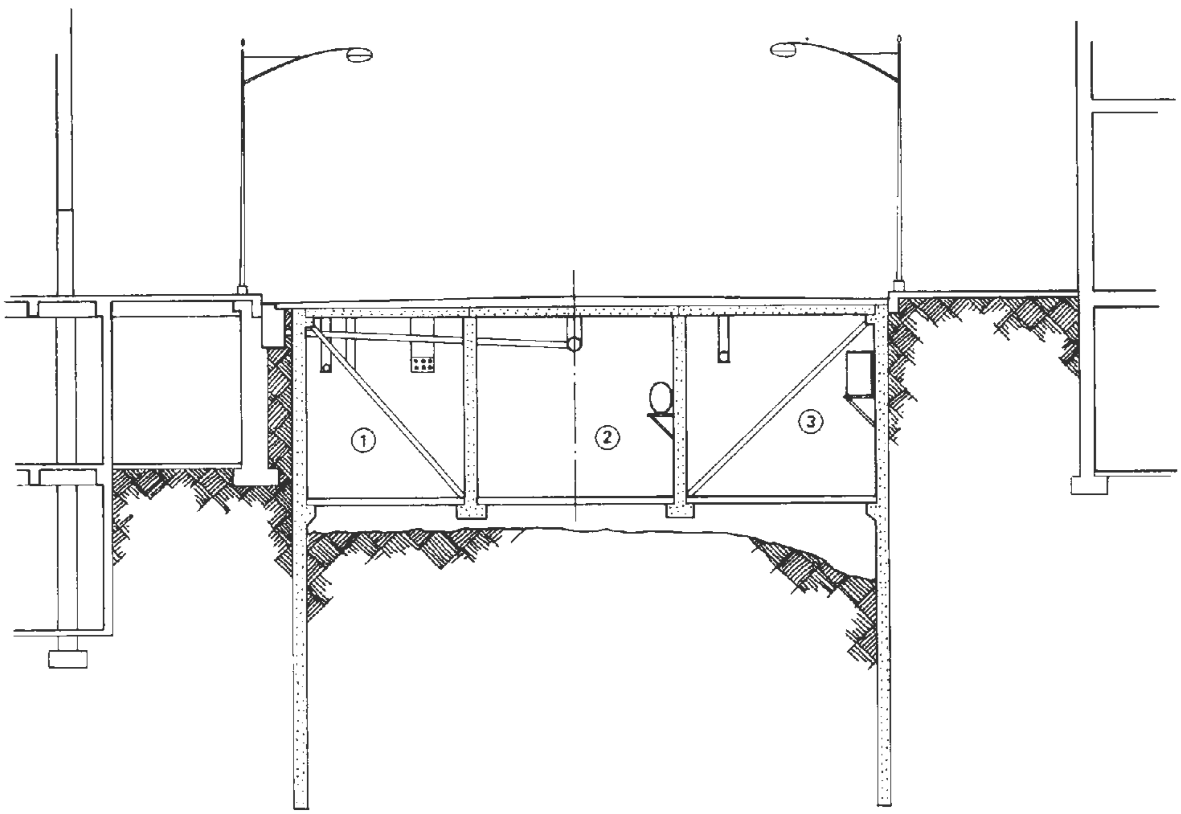


Fig. 70. Step 17: Suspend or relocate utilities in cell 2 .

Step 18: Excavate in all three cells to tunnel roof line. Maintain bearing under interior walls.

Step 19: Place and connect tunnel roof units and diagonals.

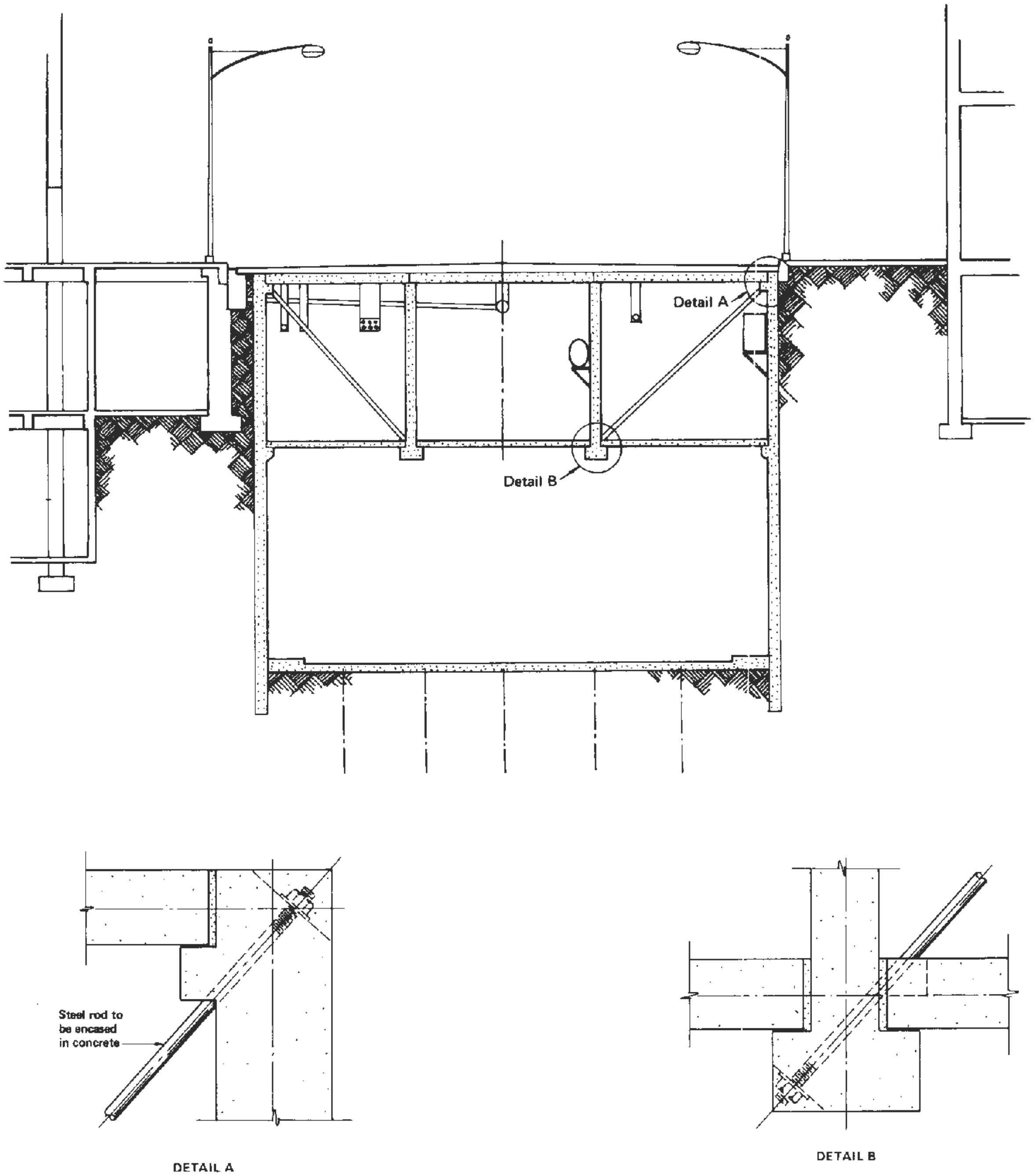


Fig. 71. Step 20: Complete tunnel under cover

Step 21: Complete utility work and restoration of street.

If desired, all utilities could be placed in center wall on special brackets for easy maintenance access (utilidor).

D. COMPARISON OF CONSTRUCTION TIME

In order to quantify the surface disruption associated with cut-and-cover construction, a comparison of anticipated construction schedules for a hypothetical highway tunnel under a congested urban street was made by Critical Path Methods. The precedence diagrams, computer outputs and bar chart schedules are shown on the following pages, and at the end of the report for (1) the construction done by present conventional means, i.e., steel soldier piles and timber lagging; temporary timber decking; permanent structure of reinforced concrete; backfilled over roof (Fig. 72* and Table 6) and (2) by methods as illustrated in Scheme A shown in Figs. 52-63. (Fig. 73* and Table 7).

The hypothetical project studied includes one city block, 300 ft long plus an intersection. It is assumed to be an interior section of a longer project, and no entrance or exit work was included. The starting date for the project was input as January 5, 1976. A comparison of the critical dates and construction times of the two methods is shown in Table 8.

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* Figures 72 and 73 appear on pages 171 and 172

Table 6

PRØJECT SCHEDULE

CØNVENTIØNAL TUNNEL CØNST

	DEPT: NUMBER	ØPERATIØN	DUR	EARLIEST START	EARLIEST FINISH	LATEST START	LATEST FINISH	TØTAL FLØAT
*	1	START 1	0	5JAN76	5JAN76	5JAN76	5JAN76	0
	2	START 2	0	6JAN76	6JAN76	5APR76	5APR76	64
	10	CUT ØFF TRAF SIDE 1	0	9JAN76	9JAN76	5FEB76	5FEB76	19
*	12	ØPEN TRAF SIDE 1	0	19FEB76	19FEB76	19FEB76	19FEB76	0
*	14	CUT ØFF TRAF SIDE 2	0	20FEB76	20FEB76	20FEB76	20FEB76	0
	16	CLØSE STREET	0	6JAN76	6JAN76	1MAR76	1MAR76	39
	18	ØPEN STREET TEMP	0	30MAR76	30MAR76	10MAR77	10MAR77	241
	20	CLØSE INTERSECTIØN	0	7JAN76	7JAN76	15APR76	15APR76	71
	22	ØPEN INTERSECT TEMP	0	6APR76	6APR76	5APR77	5APR77	254
	24	CLØSE STREET	0	31MAR76	31MAR76	11MAR77	11MAR77	241
	26	ØPEN STREET PERM	0	16MAR77	16MAR77	4MAY77	4MAY77	35
*	28	CLØSE INTERSECTIØN	0	6APR77	6APR77	6APR77	6APR77	0
*	30	ØPEN INTERSECT PERM	0	4MAY77	4MAY77	4MAY77	4MAY77	0
*	40	ERECT SDWK BAR SD 1	3	6JAN76	8JAN76	6JAN76	8JAN76	0
	42	ERECT SDWK BAR SD 2	3	6JAN76	8JAN76	21JAN76	23JAN76	11
*	45	RELØCATE UTIL AS REQ	40	6JAN76	1MAR76	6JAN76	1MAR76	0
*	100	UNDRPN SD 1 STO-1	20	9JAN76	5FEB76	9JAN76	5FEB76	0
	102	UNDRPN SD 2 STO-1	20	9JAN76	5FEB76	26JAN76	20FEB76	11
*	104	SØL PIL SD 1 STO-1	3	6FEB76	10FEB76	6FEB76	10FEB76	0
*	106	SØL PIL SD 2 STO-1	3	23FEB76	25FEB76	23FEB76	25FEB76	0
	108	L W T EL 1 STO-1	5	6FEB76	12FEB76	24FEB76	1MAR76	12
	110	L W T EL 2 STO-1	8	13FEB76	24FEB76	18MAR76	29MAR76	24
	112	L W T EL 3 STO-1	12	25FEB76	11MAR76	9APR76	26APR76	32
*	114	EXCAVAT EL 1 STO-1	5	2MAR76	8MAR76	2MAR76	8MAR76	0
*	116	EXCAVAT EL 2 STO-1	15	30MAR76	19APR76	30MAR76	19APR76	0
*	118	EXCAVAT EL 3 STO-1	15	27APR76	17MAY76	27APR76	17MAY76	0
*	120	SET DECK STO-1	5	9MAR76	15MAR76	9MAR76	15MAR76	0
*	122	SET STRT EL 2 STO-1	5	20APR76	26APR76	20APR76	26APR76	0
	124	SET STRT EL 3 STO-1	5	18MAY76	24MAY76	9JUN76	15JUN76	15
*	126	SUSP UTIL STO-1	10	16MAR76	29MAR76	16MAR76	29MAR76	0
*	128	F A P BS SL STO-1	5	16JUN76	22JUN76	16JUN76	22JUN76	0
*	130	F A P WALLS STO-1	20	23JUN76	21JUL76	23JUN76	21JUL76	0
	132	WATPRF WALLS STO-1	5	19AUG76	25AUG76	29ØCT76	4NØV76	50
*	134	F A P RØØF STO-1	20	22JUL76	18AUG76	22JUL76	18AUG76	0
	136	WATPRF RØØF STO-1	5	17SEP76	23SEP76	5NØV76	11NØV76	35
	138	BKFILL LEV 3 STO-1	5	24SEP76	30SEP76	5NØV76	11NØV76	30
	140	BKFILL LEV 2 STO-1	5	22ØCT76	28ØCT76	12NØV76	18NØV76	15
*	142	RES UTIL STO-1	20	29NØV76	27DEC76	29NØV76	27DEC76	0
	144	REM DECK STO-1	1	28DEC76	28DEC76	14MAR77	14MAR77	53
	146	BKFILL STO-1 EL 1	2	29DEC76	30DEC76	15MAR77	16MAR77	53
	148	REM SØL PILS STO-1	5	3JAN77	7JAN77	17MAR77	23MAR77	53
	150	PAVE STO-1	2	3JAN77	4JAN77	12APR77	13APR77	71
	152	RES SUR UTIL STO-1	10	10JAN77	21JAN77	24MAR77	6APR77	53
	200	UNDRPN SD 1 ST1-2	20	9JAN76	5FEB76	14JAN76	10FEB76	3
	202	UNDRPN SD 2 ST1-2	20	9JAN76	5FEB76	27JAN76	23FEB76	12

Table 6

P R Ø J E C T   S C H E D U L E

CØNVENTIØNAL TUNNEL CØNST

DEPT: NUMBER	ØPERATIØN	DUR	EARLIEST START	EARLIEST FINISH	LATEST START	LATEST FINISH	TØTAL FLØAT
* 204	SØL PIL SD 1 ST1-2	3	11FEB76	13FEB76	11FEB76	13FEB76	0
* 206	SØL PIL SD 2 ST1-2	3	26FEB76	1MAR76	26FEB76	1MAR76	0
208	L W T EL 1 ST1-2	5	6FEB76	12FEB76	16MAR76	22MAR76	27
210	L W T EL 2 ST1-2	8	13FEB76	24FEB76	8APR76	19APR76	39
212	L W T EL 3 ST1-2	12	25FEB76	11MAR76	30APR76	17MAY76	47
214	EXCAVAT EL 1 ST1-2	5	9MAR76	15MAR76	23MAR76	29MAR76	10
* 216	EXCAVAT EL 2 ST1-2	15	20APR76	10MAY76	20APR76	10MAY76	0
* 218	EXCAVAT EL 3 ST1-2	15	18MAY76	8JUN76	18MAY76	8JUN76	0
220	SET DECK ST1-2	5	16MAR76	22MAR76	30MAR76	5APR76	10
* 222	SET STRT EL 2 ST1-2	5	11MAY76	17MAY76	11MAY76	17MAY76	0
* 224	SET STRT EL 3 ST1-2	5	9JUN76	15JUN76	9JUN76	15JUN76	0
226	SUSP UTIL ST1-2	10	23MAR76	5APR76	6APR76	19APR76	10
228	F A P BS SL ST1-2	5	8JUL76	14JUL76	15JUL76	21JUL76	5
* 230	F A P WALLS ST1-2	20	22JUL76	18AUG76	22JUL76	18AUG76	0
232	WATPRF WALLS ST1-2	5	17SEP76	23SEP76	29ØCT76	4NØV76	30
* 234	F A P RØØF ST1-2	20	19AUG76	16SEP76	19AUG76	16SEP76	0
236	WATPRF RØØF ST1-2	5	15ØCT76	21ØCT76	5NØV76	11NØV76	15
238	BKFILL LEV 3 ST1-2	5	22ØCT76	28ØCT76	12NØV76	18NØV76	15
* 240	BKFILL LEV 2 ST1-2	5	19NØV76	26NØV76	19NØV76	26NØV76	0
* 242	RES UTIL ST1-2	20	28DEC76	25JAN77	28DEC76	25JAN77	0
244	REM DECK ST1-2	1	26JAN77	26JAN77	28MAR77	28MAR77	43
246	BKFILL ST1-2 EL 1	2	27JAN77	28JAN77	29MAR77	30MAR77	43
248	REM SØL PIL ST1-2	5	31JAN77	4FEB77	31MAR77	6APR77	43
250	PAVE ST1-2	2	31JAN77	1FEB77	14APR77	15APR77	53
252	RES SUR UTIL ST1-2	10	7FEB77	18FEB77	7APR77	20APR77	43
300	UNDRPN SD 1 ST2-3	20	9JAN76	5FEB76	19JAN76	13FEB76	6
302	UNDRPN SD 2 ST2-3	20	9JAN76	5FEB76	17FEB76	15MAR76	27
* 304	SØL PIL SD 1 ST2-3	3	16FEB76	18FEB76	16FEB76	18FEB76	0
306	SØL PIL SD 2 ST2-3	3	2MAR76	4MAR76	18MAR76	22MAR76	12
308	L W T EL 1 ST2-3	5	6FEB76	12FEB76	13APR76	19APR76	47
310	L W T EL 2 ST2-3	8	13FEB76	24FEB76	6MAY76	17MAY76	59
312	L W T EL 3 ST2-3	12	25FEB76	11MAR76	28MAY76	15JUN76	67
314	EXCAVAT EL 1 ST2-3	5	16MAR76	22MAR76	20APR76	26APR76	25
316	EXCAVAT EL 2 ST2-3	15	11MAY76	1JUN76	18MAY76	8JUN76	5
318	EXCAVAT EL 3 ST2-3	15	9JUN76	29JUN76	16JUN76	7JUL76	5
320	SET DECK ST2-3	5	23MAR76	29MAR76	27APR76	3MAY76	25
322	SET STRT EL 2 ST2-3	5	2JUN76	8JUN76	9JUN76	15JUN76	5
324	SET STRT EL 3 ST2-3	5	30JUN76	7JUL76	8JUL76	14JUL76	5
326	SUSP UTIL ST2-3	10	30MAR76	12APR76	4MAY76	17MAY76	25
328	F A P BS SL ST2-3	5	15JUL76	21JUL76	12AUG76	18AUG76	20
* 330	F A P WALLS ST2-3	20	19AUG76	16SEP76	19AUG76	16SEP76	0
332	WATPRF WALLS ST2-3	5	15ØCT76	21ØCT76	5NØV76	11NØV76	15
* 334	F A P RØØF ST2-3	20	17SEP76	14ØCT76	17SEP76	14ØCT76	0
* 336	WATPRF RØØF ST 2-3	5	12NØV76	18NØV76	12NØV76	18NØV76	0
338	BKFILL LEV 3 ST2-3	5	22ØCT76	28ØCT76	13DEC76	17DEC76	35

Table 6

P R Ø J E C T   S C H E D U L E

CØNVENTIØNAL TUNNEL CØNST

	DEPT: NUMBER	ØPERATIØN	DUR	EARLIEST START	EARLIEST FINISH	LATEST START	LATEST FINISH	TØTAL FLØAT
	340	BKFILL LEV 2 ST2-3	5	29NØV76	3DEC76	20DEC76	27DEC76	15
*	342	RES UTIL ST2-3	20	26JAN77	22FEB77	26JAN77	22FEB77	0
	344	REM DECK ST2-3	1	23FEB77	23FEB77	11APR77	11APR77	33
	346	BKFILL EL 1 ST2-3	2	24FEB77	25FEB77	12APR77	13APR77	33
	348	REM SØL PIL ST2-3	5	28FEB77	4MAR77	14APR77	20APR77	33
	350	PAVE ST2-3	2	28FEB77	1MAR77	18APR77	19APR77	35
	352	RES SUR UTIL ST2-3	10	7MAR77	18MAR77	21APR77	4MAY77	33
	360	CURE PAVING	10	2MAR77	15MAR77	20APR77	3MAY77	35
	400	UNDRPN SD 1 NEXT BLK	20	7JAN76	3FEB76	6APR76	3MAY76	64
	402	UNDRPN SD 2 NEXT BLK	20	7JAN76	3FEB76	6APR76	3MAY76	64
	404	SØL PIL SD 1 INTER	2	8JAN76	9JAN76	16APR76	19APR76	71
	406	SØL PIL SD 2 INTER	2	8JAN76	9JAN76	16APR76	19APR76	71
	408	L W T EL 1 INTER	5	6FEB76	12FEB76	4MAY76	10MAY76	62
	410	L W T EL 2 INTER	8	13FEB76	24FEB76	4JUN76	15JUN76	79
	412	L W T EL 3 INTER	12	25FEB76	11MAR76	28JUN76	14JUL76	87
	414	EXCAVAT EL 1 INTER	5	23MAR76	29MAR76	11MAY76	17MAY76	35
	416	EXCAVAT EL 2 INTER	15	27APR76	17MAY76	16JUN76	7JUL76	35
	418	EXCAVAT EL 3 INTER	15	25MAY76	15JUN76	15JUL76	4AUG76	35
	420	SET DECK INTER	5	30MAR76	5APR76	18MAY76	24MAY76	35
	422	SET STRT EL 2 INTER	5	18MAY76	24MAY76	8JUL76	14JUL76	35
	424	SET STRT EL 3 INTER	5	16JUN76	22JUN76	5AUG76	11AUG76	35
	426	SUSP UTIL INTER	15	6APR76	26APR76	25MAY76	15JUN76	35
	428	F A P BS SL INTER	5	22JUL76	28JUL76	10SEP76	16SEP76	35
*	430	F A P WALLS INTER	20	17SEP76	14ØCT76	17SEP76	14ØCT76	0
	432	WATPRF WALLS INTER	5	15ØCT76	21ØCT76	6DEC76	10DEC76	35
*	434	F A P RØØF INTER	20	15ØCT76	11NØV76	15ØCT76	11NØV76	0
	436	WATPRF RØØF INTER	5	12NØV76	18NØV76	13DEC76	17DEC76	20
	438	BKFILL LEV 3 INTER	5	22ØCT76	28ØCT76	12JAN77	18JAN77	55
	440	BKFILL LEV 2 INTER	5	6DEC76	10DEC76	19JAN77	25JAN77	30
*	442	RES UTIL INTER	30	23FEB77	5APR77	23FEB77	5APR77	0
*	444	REM DECK INTER	1	7APR77	7APR77	7APR77	7APR77	0
*	445	BKFILL EL 1 INTER	2	8APR77	11APR77	8APR77	11APR77	0
*	446	REM SØL PIL INTER	5	12APR77	18APR77	12APR77	18APR77	0
*	447	PAVE INTER	1	19APR77	19APR77	19APR77	19APR77	0
*	448	CURE PAVING	10	20APR77	3MAY77	20APR77	3MAY77	0
*	450	END	0	5MAY77	5MAY77	5MAY77	5MAY77	0

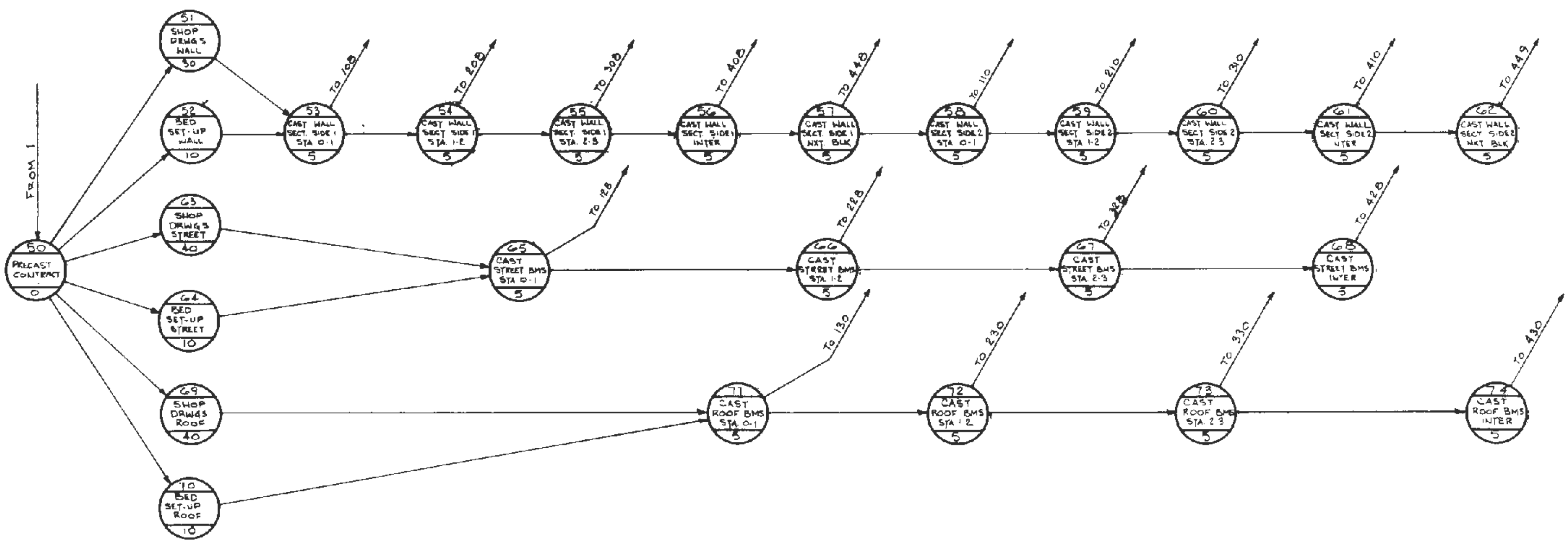


Fig. 73a. Precedence diagram for prefabrication of members
(Use with Fig. 73. at end of report)

Table 7

P R Ø J E C T   S C H E D U L E

PRECAST TUNNEL CØNST

	DEPT: NUMBER	ØPERATIØN	DUR	EARLIEST START	EARLIEST FINISH	LATEST START	LATEST FINISH	TØTAL FLØAT
*	1	START 1	0	5JAN76	5JAN76	5JAN76	5JAN76	0
	2	START 2	0	6JAN76	6JAN76	18MAR76	18MAR76	52
	10	CUT ØFF TRAF SIDE 1	0	9JAN76	9JAN76	13FEB76	13FEB76	25
*	12	RES & ØPEN SIDE 1	2	22MAR76	23MAR76	22MAR76	23MAR76	0
*	14	CUT ØFF TRAF SIDE 2	0	24MAR76	24MAR76	24MAR76	24MAR76	0
*	16	CLØSE STREET	0	29APR76	29APR76	29APR76	29APR76	0
	20	CUT ØFF SIDE ST SIDE	0	7JAN76	7JAN76	7APR76	7APR76	65
	22	CUT ØFF SIDE ST SIDE	0	7JAN76	7JAN76	22APR76	22APR76	76
*	26	ØPEN STREET	0	21JUN76	21JUN76	21JUN76	21JUN76	0
*	30	ØPEN INTERSECTIØN	0	25JUN76	25JUN76	25JUN76	25JUN76	0
	40	ERECT SDWK BAR SD 1	3	6JAN76	8JAN76	10FEB76	12FEB76	25
	42	ERECT SDWK BAR SD 2	3	7JAN76	9JAN76	19MAR76	23MAR76	52
	45	RELØCATE UTIL AS REQ	40	6JAN76	1MAR76	5MAR76	29APR76	43
*	50	PRECAST CØNTRACT	0	6JAN76	6JAN76	6JAN76	6JAN76	0
*	51	SHØP DWGS WALL	30	7JAN76	17FEB76	7JAN76	17FEB76	0
	52	BED SET UP WALL	10	7JAN76	20JAN76	4FEB76	17FEB76	20
*	53	CAST WALL S1 ST0-1	5	18FEB76	24FEB76	18FEB76	24FEB76	0
	54	CAST WALL S1 ST1-2	5	25FEB76	2MAR76	26FEB76	3MAR76	1
	55	CAST WALL S1 ST2-3	5	3MAR76	9MAR76	4MAR76	10MAR76	1
	56	CAST WALL S1 INTER	5	10MAR76	16MAR76	15MAR76	19MAR76	3
	57	CAST WALL S1 NXT BLK	5	17MAR76	23MAR76	22MAR76	26MAR76	3
	58	CAST WALL S2 ST0-1	5	24MAR76	30MAR76	29MAR76	2APR76	3
	59	CAST WALL S2 ST1-2	5	31MAR76	6APR76	6APR76	12APR76	4
	60	CAST WALL S2 ST2-3	5	7APR76	13APR76	13APR76	19APR76	4
	61	CAST WALL S2 INTER	5	14APR76	20APR76	23APR76	29APR76	7
	62	CAST WALL S2 NXT BLK	5	21APR76	27APR76	30APR76	6MAY76	7
	63	SHØP DWGS STREET BMS	40	7JAN76	2MAR76	5MAR76	29APR76	42
	64	BED SET UP STREET BM	10	7JAN76	20JAN76	16APR76	29APR76	72
	65	CAST STR BMS ST0-1	5	3MAR76	9MAR76	30APR76	6MAY76	42
	66	CAST STR BMS ST1-2	5	10MAR76	16MAR76	7MAY76	13MAY76	42
	67	CAST STR BMS ST2-3	5	17MAR76	23MAR76	14MAY76	20MAY76	42
	68	CAST STR BMS INTER	5	24MAR76	30MAR76	21MAY76	27MAY76	42
	69	SHØP DWGS RØØF BMS	40	7JAN76	2MAR76	22APR76	17JUN76	76
	70	BED SET UP RF BMS	10	7JAN76	20JAN76	4JUN76	17JUN76	106
	71	CAST RF BMS ST0-1	5	3MAR76	9MAR76	18JUN76	24JUN76	76
	72	CAST RF BMS ST1-2	5	10MAR76	16MAR76	25JUN76	1JUL76	76
	73	CAST RF BMS ST2-3	5	17MAR76	23MAR76	2JUL76	9JUL76	76
	74	CAST RF BMS INTER	5	24MAR76	30MAR76	12JUL76	16JUL76	76
	100	CØN GDE TR S1 ST0-1	4	12JAN76	15JAN76	16FEB76	19FEB76	25
*	102	CØN GDE TR S2 ST0-1	4	25MAR76	30MAR76	25MAR76	30MAR76	0
	104	C A CRS UT S1 ST0-1	3	16JAN76	20JAN76	20FEB76	24FEB76	25
*	106	C A CRS UT S2 ST0-1	3	31MAR76	2APR76	31MAR76	2APR76	0
*	108	CØN SLRY WL S1 ST0-1	6	25FEB76	3MAR76	25FEB76	3MAR76	0
*	110	CØN SLRY WL S2 ST0-1	6	5APR76	12APR76	5APR76	12APR76	0
	112	CAP BM S1 ST0-1	2	4MAR76	5MAR76	18MAR76	19MAR76	10

Table 7

P R Ø J E C T   S C H E D U L E

PRECAST TUNNEL CØNST

	DEPT: NUMBER	ØPERATIØN	DUR	EARLIEST START	EARLIEST FINISH	LATEST START	LATEST FINISH	TØTAL FLØAT
	114	CAP BM S2 STO-1	2	13APR76	14APR76	5MAY76	6MAY76	16
	116	L W T EL 1 STO-1	2	20APR76	21APR76	28APR76	29APR76	6
	118	L W T EL 2 STO-1	3	22APR76	26APR76	6JUL76	8JUL76	51
	120	L W T EL 3 STO-1	4	27APR76	30APR76	2AUG76	5AUG76	67
*	122	EXCAVAT EL 1 STO-1	5	30APR76	6MAY76	30APR76	6MAY76	0
	124	EXCAVAT EL 2 STO-1	15	28MAY76	18JUN76	9JUL76	29JUL76	28
	126	EXCAVAT EL 3 STO-1	15	28JUN76	19JUL76	6AUG76	26AUG76	28
*	128	SET STREET BMS STO-1	5	7MAY76	13MAY76	7MAY76	13MAY76	0
	130	SET RØØF BMS STO-1	5	21JUN76	25JUN76	30JUL76	5AUG76	28
	132	CAST BASE SL STO-1	5	20JUL76	26JUL76	27AUG76	2SEP76	28
	134	SUSP UTIL STO-1	10	14MAY76	27MAY76	24JUN76	8JUL76	28
	136	CAST TØPPING STO-1	2	28JUN76	29JUN76	9AUG76	10AUG76	29
	138	SEAL WL JNTS STO-1	3	27JUL76	29JUL76	3SEP76	8SEP76	28
	140	WATPRF STREET STO-1	3	14MAY76	18MAY76	24MAY76	26MAY76	6
	142	RES UTIL STO-1	20	30JUN76	28JUL76	11AUG76	8SEP76	29
	144	PAVE STO-1	2	19MAY76	20MAY76	27MAY76	28MAY76	6
	200	CØN GDE TR S1 ST1-2	3	12JAN76	14JAN76	25FEB76	27FEB76	32
	202	CØN GDE TR S2 ST1-2	3	25MAR76	29MAR76	5APR76	7APR76	7
	204	C A CRS UT S1 ST1-2	3	15JAN76	19JAN76	1MAR76	3MAR76	32
	206	C A CRS UT S2 ST1-2	3	30MAR76	1APR76	8APR76	12APR76	7
*	208	CØN SLRY WL S1 ST1-2	5	4MAR76	10MAR76	4MAR76	10MAR76	0
*	210	CØN SLRY WL S2 ST1-2	5	13APR76	19APR76	13APR76	19APR76	0
	212	CAP BM S1 ST1-2	2	5JAN76	6JAN76	18MAR76	19MAR76	53
	214	CAP BM S2 ST1-2	2	20APR76	21APR76	12MAY76	13MAY76	16
	216	L W T EL 1 ST1-2	2	27APR76	28APR76	5MAY76	6MAY76	6
	218	L W T EL 2 ST1-2	3	29APR76	3MAY76	6JUL76	8JUL76	46
	220	L W T EL 3 ST1-2	4	4MAY76	7MAY76	2AUG76	5AUG76	62
	222	EXCAVAT EL 1 ST1-2	5	29APR76	5MAY76	7MAY76	13MAY76	6
	224	EXCAVAT EL 2 ST1-2	15	7JUN76	25JUN76	9JUL76	29JUL76	23
	226	EXCAVAT EL 3 ST1-2	15	6JUL76	26JUL76	6AUG76	26AUG76	23
*	228	SET STREET BMS ST1-2	5	14MAY76	20MAY76	14MAY76	20MAY76	0
	230	SET RØØF BMS ST1-2	5	28JUN76	2JUL76	30JUL76	5AUG76	23
	232	CAST BASE SL ST1-2	5	27JUL76	2AUG76	27AUG76	2SEP76	23
	234	SUSP UTIL ST1-2	10	21MAY76	4JUN76	24JUN76	8JUL76	23
	236	CAST TØPPING ST1-2	2	6JUL76	7JUL76	9AUG76	10AUG76	24
	238	SEAL WL JNTS ST1-2	3	3AUG76	5AUG76	3SEP76	8SEP76	23
	240	WATPRF STREET ST1-2	3	21MAY76	25MAY76	26MAY76	28MAY76	3
	242	RES UTIL ST1-2	20	8JUL76	4AUG76	11AUG76	8SEP76	24
	244	PAVE ST1-2	2	26MAY76	27MAY76	1JUN76	2JUN76	3
	300	CØN GDE TR S1 ST2-3	3	12JAN76	14JAN76	3MAR76	5MAR76	37
	302	CØN GDE TR S2 ST2-3	3	25MAR76	29MAR76	12APR76	14APR76	12
	304	C A CRS UT S1 ST2-3	3	15JAN76	19JAN76	8MAR76	10MAR76	37
	306	C A CRS UT S2 ST2-3	3	30MAR76	1APR76	15APR76	19APR76	12
*	308	CØN SLRY WL S1 ST2-3	5	11MAR76	17MAR76	11MAR76	17MAR76	0
*	310	CØN SLRY WL S2 ST2-3	5	20APR76	26APR76	20APR76	26APR76	0

Table 7

P R Ø J E C T   S C H E D U L E

PRECAST TUNNEL CØNST

	DEPT:			EARLIEST		LATEST		TØTAL
	NUMBER	ØPERATIØN	DUR	START	FINISH	START	FINISH	FLØAT
*	312	CAP BM S1 ST2-3	2	18MAR76	19MAR76	18MAR76	19MAR76	0
*	314	CAP BM S2 ST2-3	2	27APR76	28APR76	27APR76	28APR76	0
	316	L W T EL 1 ST2-3	2	29APR76	30APR76	12MAY76	13MAY76	9
	318	L W T EL 2 ST2-3	3	3MAY76	5MAY76	6JUL76	8JUL76	44
	320	L W T EL 3 ST2-3	4	6MAY76	11MAY76	2AUG76	5AUG76	60
	322	EXCAVAT EL 1 ST2-3	5	3MAY76	7MAY76	14MAY76	20MAY76	9
	324	EXCAVAT EL 2 ST2-3	15	14JUN76	2JUL76	9JUL76	29JUL76	18
	326	EXCAVAT EL 3 ST2-3	15	13JUL76	2AUG76	6AUG76	26AUG76	18
*	328	SET STREET BMS ST2-3	5	21MAY76	27MAY76	21MAY76	27MAY76	0
	330	SET RØØF BMS ST2-3	5	6JUL76	12JUL76	30JUL76	5AUG76	18
	332	CAST BASE SL ST2-3	5	3AUG76	9AUG76	27AUG76	2SEP76	18
	334	SUSP UT ST2-3	10	28MAY76	11JUN76	24JUN76	8JUL76	18
	336	CAST TØPPING ST2-3	2	13JUL76	14JUL76	9AUG76	10AUG76	19
	338	SEAL WL JNTS ST2-3	3	10AUG76	12AUG76	3SEP76	8SEP76	18
*	340	WATPRF STREET ST2-3	3	28MAY76	2JUN76	28MAY76	2JUN76	0
	342	RES UTIL ST2-3	20	15JUL76	11AUG76	11AUG76	8SEP76	19
*	344	PAVE ST2-3	2	3JUN76	4JUN76	3JUN76	4JUN76	0
*	360	CURE PAVING	10	7JUN76	18JUN76	7JUN76	18JUN76	0
	400	CØN GDE TR S1 INTER	2	8JAN76	9JAN76	8APR76	9APR76	65
	402	CØN GDE TR S2 INTER	2	8JAN76	9JAN76	23APR76	26APR76	76
	404	C A CRS UT S1 INTER	6	12JAN76	19JAN76	12APR76	19APR76	65
	406	C A CRS UT S2 INTER	6	12JAN76	19JAN76	27APR76	4MAY76	76
	408	CØN SLRY WL S1 INTER	2	18MAR76	19MAR76	20APR76	21APR76	23
	410	CØN SLRY WL S2 INTER	2	27APR76	28APR76	5MAY76	6MAY76	6
	412	CAP BM S1 INTER	2	22MAR76	23MAR76	26MAY76	27MAY76	47
	414	CAP BM S2 INTER	2	29APR76	30APR76	26MAY76	27MAY76	19
	416	L W T EL 1 INTER	2	6MAY76	7MAY76	14MAY76	17MAY76	6
	418	L W T EL 2 INTER	3	10MAY76	12MAY76	22JUN76	24JUN76	30
	420	L W T EL 3 INTER	4	13MAY76	18MAY76	2AUG76	5AUG76	55
	422	EXCAVAT EL 1 IINTERR	8	10MAY76	19MAY76	18MAY76	27MAY76	6
*	424	EXCAVAT EL 2 INTER	15	25JUN76	16JUL76	25JUN76	16JUL76	0
	426	EXCAVAT EL 3 INTER	15	26JUL76	13AUG76	6AUG76	26AUG76	9
*	428	SET STREET BMS INTER	4	28MAY76	3JUN76	28MAY76	3JUN76	0
*	430	SET RØØF BMS INTER	5	19JUL76	23JUL76	19JUL76	23JUL76	0
	432	CAST BASE SL INTER	5	16AUG76	20AUG76	27AUG76	2SEP76	9
*	434	SUSP UTIL INTER	15	4JUN76	24JUN76	4JUN76	24JUN76	0
*	436	CAST TØPPING INTER	2	26JUL76	27JUL76	26JUL76	27JUL76	0
	438	SEAL WL JNTS INTER	3	23AUG76	25AUG76	3SEP76	8SEP76	9
*	440	WATPRF STREET INTER	3	4JUN76	8JUN76	4JUN76	8JUN76	0
*	441	RES UTIL INTER	30	28JUL76	8SEP76	28JUL76	8SEP76	0
*	442	PAVE INTER	2	9JUN76	10JUN76	9JUN76	10JUN76	0
*	443	CURE PAVING	10	11JUN76	24JUN76	11JUN76	24JUN76	0
	444	GDE TR S1 NEXT BLK	3	7JAN76	9JAN76	14APR76	16APR76	70
	445	GDE TR S2 NEXT BLK	3	31MAR76	2APR76	29APR76	3MAY76	21
	446	CRS UTIL S1 NEXT BLK	3	12JAN76	14JAN76	19APR76	21APR76	70

Table 7

P R Ø J E C T   S C H E D U L E

PRECAST TUNNEL CØNST

DEPT: NUMBER	ØPERATIØN	DUR	EARLIEST START	EARLIEST FINISH	LATEST START	LATEST FINISH	TØTAL FLØAT
447	CRS UTIL S2 NEXT BLK	3	5APR76	7APR76	4MAY76	6MAY76	21
448	SLRY WL S1 NEXT BLK	5	24MAR76	30MAR76	22APR76	28APR76	21
449	SLRY WL S2 NEXT BLK	5	29APR76	5MAY76	7MAY76	13MAY76	6
* 450	END	0	9SEP76	9SEP76	9SEP76	9SEP76	0

Table 8. Comparison of Critical Dates and Construction Times

		Conventional	Slurry Trench & Prefabricated Components
a.	Starting date (input)	Jan. 5, 1976	Jan. 5, 1976
b.	First surface disruption (construct sidewalk barricades)	Jan. 6, 1976	Feb. 10, 1976
c.	Street closed to traffic	Mar. 1, 1976 - Mar. 30, 1976	Apr. 29, 1976 - June 21, 1976
d.	Street open to traffic on temporary decking	Mar. 30, 1976- Mar. 11, 1977	
e.	Street partially closed to traffic (one side)	Feb. 5, 1976 - Mar. 1, 1976	Feb. 13, 1976 - Apr. 29, 1976
f.	Street closed to traffic for restoration	Mar. 11, 1977 - May 4, 1977	
g.	Intersection closed to traffic	Mar. 13, 1976 - Apr. 6, 1976	May 18, 1976 - June 25, 1976
h.	Intersection open to traffic on temporary decking	Apr. 6, 1976 - Apr. 6, 1977	
i.	Intersection partially closed to traffic (one side)		Apr. 7, 1976 - May 18, 1976
j.	Intersection closed to traffic for restoration	Apr. 6, 1977 - May 4, 1977	
k.	Total length of time street or intersection closed	14 weeks	8 weeks
l.	Total length of time street or intersection partially closed	4 weeks	13 weeks
m.	Total length of time of full or partial disruption	16 months	$4\frac{1}{2}$ months
n.	Total length of time for project	16 months	8 months

XII. APPLICATIONS OF POST-TENSIONED CONCRETE

The possible uses of post-tensioning in conjunction with precast concrete elements has been briefly mentioned in previous sections of this report. In this section we will further explore these possible applications.

A. USE IN HORIZONTAL ELEMENTS.

One of the most recent developments in the prestressed concrete industry has been the joining of precast elements by post-tensioning, known as "segmental" construction. This method has been used extensively on long span bridges in Europe, and has recently been specified on several bridge projects in the United States. In the more publicized method of segmental construction, relatively wide, deep and short members are prefabricated in a precasting plant, with or without pretensioning. The precasting procedure has recently involved a "match-casting" technique, and the segments are field-jointed using epoxy cements as well as post-tensioning.

While this type of segmental construction has no apparent application in cut-and-cover tunnel construction, another method has also been called "segmental construction". This is the joining of two standard girders with post-tensioning. In this method, a space is left between the two segments, and filled with cast-in-place concrete. As has been normally applied to bridge construction, in order to achieve longer spans, the cast-in-place section is allowed to cure before the tensioning is done, and the tensioning force is applied at the ends in the conventional manner.

A different method was used on the Yerba-Buena Island tunnel as a part of the San Francisco-Oakland Bay Bridge reconstruction. On this structure, the two sections were jacked apart from the center to apply the tension to the tendons as illustrated in Fig. 6 and previously described in Section III. This method is ideally adapted to those cut-and-cover tunnel construction situations that require partial traffic maintenance. In this case, a similar procedure to that described in Section XI-B of this report could be used. The sections on half the street could be set, using a temporary support at the center. Traffic could then be diverted over these members while the other half of the members were placed.

B. USE IN VERTICAL ELEMENTS.

Post-tensioning has been used by ICOS in cast-in-place slurry walls. It also may be applicable in precast concrete elements. In the precast case, the post-tensioning tendons would be placed in the form prior to casting. Locating these tendons in precast elements is easier than in cast-in-place walls. Also, anchorage at the bottom (dead) end can be more positive. The prefabricated elements could be pretensioned or conventionally reinforced for handling stresses.

The use of post-tensioning in wall units has several structural advantages over pretensioning. First, since the prestressing force is applied against the ground, passive pressure develops which in turn provides a balancing moment on the section; hence, theoretically, the only limitation on the prestressing force, if the tendons can be placed in the theoretical optimum position, is the direct compressive force, P/A. Excessive tensile stresses induced by post-tensioning are reduced by the passive pressure bending moments. Secondly, since the unit can cure longer before the prestressing force is applied, the unit can be designec for high compressive strength without the limitation imposed on economical pretensioning by the need for rapid bed turnover. A third advantage is that tendons can be economically placed closer to their theoretical optimum position, utilizing parabolic drape. Fourth, the prestressing force can be changed as excavation progresses and external moments change.

The main advantages of pretensioning compared with post-tensioning in precast members are: 1) Pretensioning can also take care of handling stresses, so supplemental reinforcing is not required. 2) Pretensioning materials and procedures are less costly than post-tensioning. 3) Control of prestressing operations is easier under factory conditions than in the field.

Of the structural schemes suggested in this report, the ones which appear most applicable to post-tensioning are the ones which employ the deeper king piles. These deeper members allow more flexibility in arranging tendon drape, and can more readily accommodate handling stresses.

Sample preliminary calculations indicate that the post-tensioning concept would be more economically feasible in construction methods where it was desirable to have relatively deep clear spans--twenty feet or more.

XIII. APPLICATION OF OTHER MODERN MATERIALS

In addition to pretensioned concrete, post-tensioned concrete and high-strength concretes discussed in previous sections, other materials of construction have been developed, or are being developed which may be useful in fabricated structural elements for cut-and-cover tunnels. Some of these materials have proven themselves in other types of construction; others are still experimental, but may have future application.

A. STRUCTURAL STEEL.

Prior to 1960, most structural steel used for construction was defined by ASTM Specification A-7. This was a carbon steel with a minimum yield point of 33,000 psi. ASTM A-36 was first adopted in 1960 and in 1967 entirely replaced the A-7 steel in the ASTM Book of Standards. A-36 has a minimum yield strength of 36,000 psi.

During the 1960's ASTM adopted specifications covering a broad range of structural steels. Prior to that time the steels had been available under trade names of various steel companies, so that by the time the ASTM standards were adopted many of them had ample opportunity to prove themselves in field practice. These new high-strength structural steels fall into three broad categories:

1) Carbon steel, which includes ASTM A-36 and ASTM A-283, a less expensive steel per pound than A-36, but with lower yield strength.
2) High-strength low alloy steels, which contain moderate portions of alloying elements other than carbon. These include ASTM A-440 with yield strengths of 42,000 to 50,000 psi depending on the thickness. This material is generally considered unsuitable for welding

because of high carbon content. ASTM A-441 was adopted in 1960 and has similar properties to A-440 except that the required strength level is achieved by adding vanadium rather than increasing the carbon. The result is better weldability. Specification A-572 was issued in 1966. This defines six grades of high-strength, low alloy steels having specified minimum yield points ranging from 42,000 to 65,000 psi.

Included in the high-strength low alloy steel group are the "weathering" or "self-painting" steels, which provide outstanding resistance to atmospheric corrosion by developing a protective oxide coating when exposed to the atmosphere. Uncoated weathering steels are not recommended for exposure to concentrated industrial fumes, or exposure in marine locations where salt can be deposited on the steel by either spray or fog, or where the steel is either buried in soil or submerged in water. These steels have yield points of 42,000 to 50,000 psi and are covered by ASTM specifications A-242 and A-588.

3) Quenched and tempered steel. These steels differ from high-strength low alloy steels in that they rely upon heat treatment to develop higher strength levels and other improved mechanical properties. There are two basic classes of quenched and tempered steel--alloy and carbon--with the alloy providing a yield strength beyond that of carbon. The quenched and tempered alloy steels have specified minimum yield strengths of 90,000 to 100,000 psi depending upon the thickness. They are covered by ASTM A-514 and ASTM A-517.

The primary structural steel for construction is ASTM A-36. For most applications this will prove to be the most economical. Since the modulus of elasticity of structural steel is quite constant regardless of the yield points, deflection and buckling criteria will very often govern the design of the steel member rather than the tensile strength. The cost per pound is significantly higher for most of the higher strength steels.

For use in tunnel construction, high-strength structural steels would probably have very little application except as tension members for trusses or other applications where tensile strength is the prime requirement. The "weathering" steels may be of an advantage for those schemes, illustrated in this report, where steel members would be exposed in a utility tunnel above the main roof of the tunnel. The resistance to corrosion could be an advantage in minimizing maintenance. However, it is doubtful that the additional cost would be worth it.

B. HIGH-STRENGTH PRESTRESSING STEEL.

Three basic materials are commonly used for prestressing steel in prestressed concrete members. These same steels are used in underground construction for prestressed concrete rock and soil anchors. In at least one of the schemes suggested in this report for using precast concrete elements in cut-and-cover tunnel applications, high tensile strength tension ties are required for structural purposes. These tension ties can also be of the same material commonly used in prestressing applications.

Steels used are: 1) ASTM designation A-421 "Uncoated Stress Relieved Wire for Prestressed Concrete"; 2) ASTM A-416 "Uncoated Seven Wire Stress Relieved Strand for Prestressed Concrete"; and 3) high alloy steel bars, either smooth or deformed. All of these materials depend upon cold working to attain their high ultimate tensile strength.

C. CONCRETE MATERIALS.

1. Expansive Cements.

Expansive cement grouts of various proprietary formulations have been used for many years for specialized applications such as grouting under base plates, in joints or any other applications where shrinkage normally associated with portland cement grouts would be undesirable. In recent years there has been a development of a "non-shrink" concrete made with a specially formulated portland cement which exhibits mild expansive characteristics. This material is now widely available from most cement manufacturers either under the proprietary trade name "Chem-Comp" or under license of patents held by the Portland Cement Association.

The primary application to date of these expansive cement concretes has been to reduce cracking in concrete slabs on grade or in structural topping over precast concrete elements. When used in slabs on grade, there have been many examples of slabs with joint spacing up to 100 ft on centers that have been completely crack free. In order to obtain these results, however, it has been necessary to practice very sound formulation, mixing, placing, and curing of the concrete. It is also necessary to have a design of the floor slab with adequate reinforcing steel so that the "prestressing" effect of the expansion can control the cracking.

When used as a structural topping over precast concrete elements there is an additional advantage. Very often the differential shrinkage characteristics of the freshly cast concrete topping on the well cured precast member can set up forces which induce negative moments in the precast element and may have an undesirable effect on deflection. For this reason these expansive cement concretes have been in favor for use on prestressed concrete parking structures. It is possible that this use would have some advantage for the pavement slabs on top of precast concrete decks at the street level on some of the methods of construction suggested in this report.

In nearly all of the methods of construction suggested, the base slab of the tunnel is to be of cast-in-place concrete. The slab also acts as a reaction against the passive pressures of the soil on the outside and expansive cement concretes could be advantageous in assuring that this reaction is adequately developed.

2. <u>Polymer Modified Concretes.</u>

These materials are currently under development and at this time must be considered experimental. The American Concrete Institute has set up a committee (ACI 548) on polymer modified concretes. This committee has established two general categories of polymer modified concrete: 1) polymer impregnated concrete, PIC, and 2) polymer cement concrete, PCC.

Both PIC and PCC have exhibited some extraordinary characteristics. They are exceptionally durable under extended freeze-thaw cycles, are essentially inert to most chemicals, have near zero water absorption, and are essentially impermeable. Compressive strengths of as high as 22,000 psi have been obtained in the laboratory and a modulus of rupture of 2,000 psi is not unusual.

The modulus of elasticity of most formulations is above 6,000,000 psi and long term creep is near zero. Bond strength is as much as 250% of that of regular concrete; surface hardness and wear resistance are greatly increased.

Polymer impregnated concrete is made by first casting regular concrete and letting it cure. It is then dried at elevated temperatures and vacuumed for near total removal of free water and air. A monomer is then forced into the voids of the concrete under pressure and the monomer is polymerized by use of a catalyst and heat. This process results in a polymerzation of the concrete near the surface, but the remainder of the section is essentially the same as the concrete as originally cast. The Federal Highway Administration and the Bureau of Reclamation with technical assistance from the Prestressed Concrete Institute have jointly sponsored research on the use of PIC for precast concrete bridge deck units. This research was conducted by Dr. Allen G. Thurman of Denver and Prestressed Concrete of Colorado in Denver. It is quite apparent that the multiple step operation in producing the PIC is quite costly and the equipment involved in pressurizing, vacuuming the unit, and curing at elevated temperatures requires a high capital investment. The tests have shown that the deck units themselves do successfully perform the functions intended of them, with the virtual elimination of scaling and deterioration of the deck surface.

Polymer cement concretes have been made with two different processes. One involves the mixing of the monomer in the wet concrete mix and then achieving the polymerzation within the mix through a catalyst. Second, and probably a more promising development has been the mixing of the material which has been previously polymerized with the other concrete materials. Manufactured in this manner it is entirely feasible for a standard precasting plant to use the material in its precast and prestressed sections. While the materials themselves at the present time are extremely costly compared with normal concrete mixes, it is anticipated that if markets can be developed the costs can be significantly reduced in the future.

Polymer cement concrétes also may have some application in cast-in-place concrete. An extensive field testing program by Dow Chemical Company has shown that it can be used as a surfacing material over bridges and exhibit the resistarce to freeze-thaw and chemical scaling which has been such a problem with bridge decks in the past. The high modulus of rupture also suggests that this could be used in other flexural applications. Frequently in cut-and-cover tunnels using slurry walls it is difficult to place reinforcing steel in the proper position becaʲse of interference by utilities or other underground structures which must be maintained in place. In some applications it may be adequate to put a small amount of this type of material (PCC) to span short distances unreinforced. Since relatively small amounts might be involved in this application it is possible that the high cost would be worth it.

3. Fiber Reinforced Concrete.

This material consists of random fibers of reinforcing material which is mixed with the wet concrete prior to placing. Much research has been done on this concept in the last ten years, although the original research was done as early as 1920. Experimentation has been with three basic categories of fibers: 1) mineral (glass and asbestos); 2) organic (nylon, polypropylene and polyethylene) and 3) metallic (primarily steel).

Asbestos fibers have been successfully combined with portland cement paste to form a product called asbestos cement. The product has been used for building siding material and for pipe. Manufacturing processes of asbestos cement are much different from conventional concrete, closely resembling the process used in manufacturing paper. Only a few countries are important producers of asbestos, and in these countries the better grades are being depleted. Because of this and because of a lung cancer called "Asbestosis" the markets for the product are not expanding.

The use of glass fibers for reinforcing has been moderately successful in laboratory investigations. The material has exhibited much higher modulus of rupture than ordinary concrete with some researchers reporting values as high as 2500 to 3000 psi. Some types of glass fibers are susceptible to alkaline attack from the cement. This has been overcome by use of a protective organic coating or by using a glass fiber of high alkali resistance. It appears that glass fibers may some day replace asbestos fibers for certain applications.

Some of the available organic fibers are also susceptible to deterioration by alkalis such as cotton, rayon, and polyester. Those which are not subjected to chemical attack in cement paste include nylon, polypropylene and polyethylene. Because of their considerably lower modulus of elasticity these fibers do not appreciably increase strength of the portland cement matrix. Often they reduce the strength. However, considerable increase in impact resistance (10 to 25 times) has been observed. Although organic fibers are lighter and cheaper on volume basis, they are more difficult to distribute in fresh concrete and they have poorer bond.

Steel fibers have a modulus of elasticity which is six times that of high strength concrete. They have reasonably good bond, have a high elongation at fracture, and are easier to mix. The modulus of rupture of steel fiber reinforced concrete can be greatly increased with some values reported as high as 4,000 psi. Some experimentation has centered around size and shape of the steel fibers. The sizes of the steel fibers vary from 10 to 20 mils and have lengths from 1/2 to 2 inches. Thus the commercially available aspect ratio ranges from about 30 to 150. An aspect ratio of about 100, length of about 1 in. and volume of about 2% are about the most common proportions tried in field applications.

One of the biggest problems with the use of fibers in concrete is getting uniform distribution throughout the concrete mix. The fibers have a tendency to segregate or ball up during the mixing. Mixing is extremely difficult with fibers having aspect ratios greater than 100 or volumes larger than 4%. The problems of mixing are increased and many of the improved properties associated with fiber reinforced concrete are decreased with increasing size and volume of aggregates. Thus, sand alone is often used as aggregate. A small quantity of small sized aggregates, a large quantity of cement (6 to 10 sacks per cu yd) and low slump are characteristics of fiber reinforced concrete mixes.

Since the fibers reinforce the concrete in a random manner it is obvious that there is often reinforcing in areas where it is not needed. For this reason the utilization of the reinforcement is relatively poor and this obviously increases the cost. At the present time the cost of fiber reinforced concrete is prohibitive for most structural applications. For very specialized uses where there are few other solutions, it may have some application. In cut-and-cover tunnel applications it seems possible that some use could be made for placement of concrete in areas where access for placing reinforcing was difficult or impossible.

D. EPOXY MATERIALS.

The use of epoxy and related materials for concrete repair has been of proven effectiveness for several years. Many commercial materials are available on the market. More recently the epoxy materials have also been used in connections of precast concrete. Usually they are used in conjunction with some mechanical means of connection such as post-tensioning, bolting, or welding.

The injection under pressure of liquid epoxy resin adhesives to repair cracking in precast and cast-in-place concrete has been successfully used for repair of cracks as small as 0.004 in. wide. Specialists in this procedure are available throughout the United States. Cracks narrower than 0.004 in. are considered by the ACI Committee 224, Cracking, to be of no significance for any type of application, and for most applications cracks as wide as 0.01 in. can be tolerated.

Availability of epoxy repair materials and procedures can be important in overcoming some of the reticence of engineers to specify methods of construction where inspection of the finished product is not available until replacement of the product would be impractical such as slurry wall construction. Epoxy joining materials could also have good applications when it is determined that lateral transfer of load is required, as for example, in prefabricated structural members placed in slurry trenches.

Epoxy resins have also been used as matrix material in certain types of concretes. These would appear applicable only in very specialized situations and their use in tunnel work would seem to be negligible.

SUMMARY AND CONCLUSIONS

1. Underground transportation tunnels are generally considered the only feasible solution to the growing problem of moving people and goods in urban areas. However, the extremely high cost and extended periods of surface disruption during construction of the facilities threaten the extension of such systems. The most common method of constructing these tunnel facilities involves constructing a temporary ground support system during excavation and building the permanent structure within this support system.

2. The best, and perhaps the only way to substantially reduce costs and surface disruption in cut-and-cover tunnel construction in urban areas is to design the permanent structure to act also as the temporary ground support system.

3. The only ways found in this study to accomplish this are (a) with drilled-in tangent or overlapping cast-in-place piles or (b) with the use of slurry wall construction. Of the two, slurry wall construction seems to hold the most promise.

4. "Under the roof" methods of construction will significantly reduce surface disruption time. In this method, the permanent deck is installed early in the construction period and all subsequent work is performed under that deck. This method of construction, however, makes backfilling over the tunnel roof very difficult, if not impossible. Since most urban cut-and-cover tunnels are placed under existing streets, this means that present utilities must be either relocated or installed in a utility tunnel within the space between the tunnel roof and the street.

5. This method of construction implies that the designers of the permanent structure must also specify the construction procedures and sequences.

6. A variety of prefabricated structural elements, practical with present technology, are suitable for use in cut-and-cover tunnel construction. These include both steel and concrete members and both vertical and horizontal members. There are several ways in which prefabricated members can be combined with cast-in-place members. This study reaches the conclusion that no specific type or shape of element is "best".

7. The key to significant reduction in surface disruption time is the construction of the walls by slurry trench methods. Very little significant advantage (in time) can be documented for any particular type of wall, i.e., totally prefabricated, totally cast-in-place, or combinations. The reason is that the slowest operation in the slurry wall construction process is the excavation of the trench rather than the placing of the structural elements. While this excavation process could be expedited by using additional equipment, when the total construction is considered, there appears to be only incremental savings of total time.

8. Of the prefabricated elements investigated, precast concrete would be the most readily accepted. Precasting of the horizontal elements would result in lower costs and shorter construction time than cast-in-place construction. Pretensioning of the precast elements shows significant material savings. Designing of the precast section for maximum efficiency is another way to reduce costs.

9. The constraints imposed by the site conditions will determine the most feasible methods. The best ideas seem to be as follows:

a. Vertical walls:

   1) Continuous interlocking sheet piles 4' to 8' wide.

   2) King piles at 8 to 12 ft spacing with precast or cast-in-place between.

b. Horizontal elements:

   1) With no backfill, precast, prestressed sections of any of a wide variety of shapes. If the street can be closed for 4 to 6 weeks, members span full width. If street must remain open continuously, modified Vierendiel or segmental precast/post-tensioned schemes.

   2) Steel trusses

   3) Concrete trusses

   4) If backfill required, precast concrete arches.

Volume II of this study, "Designs for Three Sites", reports on case studies in which the concepts explored in this report are tested by applying them to designs of tunnels at three specific sites in various parts of the United States. The cases selected represent sites where tunnels have actually been proposed.

REFERENCES

1. Consoer, Townsend and Associates and Chicago Urban Transportation District, Utility Tunnel Economic Feasibility Analysis in Conjunction With the Construction of Subways by the Cut-and-Cover Method, Contract No. DOT OS 40166, Department of Transportation, Office of the Secretary of Transportation, and the National Science Foundation (1975).

2. Xanthakos, Petros P., Underground Construction in Fluid Trenches, University of Illinois at Chicago Circle, Chicago, Illinois (1974).

3. Sverdrup and Parcel and Associates, Inc., Cut-and-Cover Tunneling Techniques, Volume 1, A Study of Art, Contract No. DOT-FH-22-7803, Federal Highway Administration, National Technical Information Service, PB-222-997, (1973).

4. Sverdrup and Parcel and Associates, Inc., Cut-and-Cover Tunneling Techniques, Volume 2, Appendix, Contract No. DOT FH 11-7803, Federal Highway Administration, National Technical Information Service, PB-222-998, (1973).

5. D'Appolonia, D. J., et al, Proceedings of Workshop on Cut-and-Cover Tunneling: Precast and Cast-in-Place Diaphragm Walls Constructed Using Slurry Trench Techniques, Federal Highway Administration, Report No. FHWA-RD-74-57 (1974).

6. Gerwick, Ben C., Jr. "Slurry-Trench Techniques in Deep Foundation Construction", ASCE Structural Engineering Conference, Seattle, Wash., May 8-12, 1967.

7. Precast, Prestressed Concrete Short Span Bridges -- Spans to 100 Feet, First edition, first printing. Prestressed Concrete Institute, Chicago, Ill. (1975).

8. PCI Design Handbook, Precast and Prestressed Concrete, First edition, second printing. Prestressed Concrete Institute, Chicago, Ill. (1972).

9. Building Code Requirements for Reinforced Concrete (ACI 318-71), American Concrete Institute, Detroit, Mich. (1971).

10. Halperin, David, "The Sinking of the Amsterdam Metro", Civil Engineering, Vol. 45, No. 9, Sept., 1975 pp. 92-95.

11. Standard Specifications for Highway Bridges Eleventh edition, 1973, with 1974 and 1975 Interim specifications. American Association of State Highway and Transportation Officials, Washington, D. C.

12. Field Design Standards, New York City Transit Authority (1974).

13. Navfac DM-7, Design Manual, Soil Mechanics, Foundations and Earth Structures, Department of the Navy, Navy Facilities Engineering Command, 1971.

14. Peck, Hanson, Thornburn, Foundation Engineering, 2nd Ed., 1974.

15. Terzaghi, Karl, "Evaluation of Coefficients of Subgrade Reaction", Geotechnique, Vol. 5, No. 4, Dec., 1955 pp. 297-326.

16. Clough, G. Wayne, "Analytical Modeling of Diaphragm Walls for Excavation Support" (An address at the seminar on Underground Construction Problems, Techniques and Solutions, Chicago, Ill., Oct., 1975.

17. "STRESS, A Users Manual", Massachusetts Institute of Technology, Cambridge, Mass.

18. Manual of Design Criteria, Washington Metropolitan Area Transit Authority.

19. Cohen, Edward, Solomon, Jacob and Bacci, Joseph "Washington, D. C. Subway" Journal of the American Concrete Institute, Proceedings Vol. 72, No. 7, July, 1975 pp. 305-312.

20. Wickham, G. E. and Tiedemann, H. R., Optimizing Cut-and-Cover Tunneling Operations, Interim Report, Contract No. DOT-FH-11-8513, Federal Highway Administration, January, 1975.

21. Anderson, Arthur R. and Moustafa, Saad E., "Ultimate Strength of Prestressed Concrete Piles and Columns", Journal of the American Concrete Institute, Proceedings Vol. 67, No. 8, Aug., 1970, pp. 620-633.

22. PCI Manual on Design of Connections for Precast, Prestressed Concrete, First edition, Prestressed Concrete Institute, Chicago, Ill. (1973).

23. PCI Post-tensioning Manual, First edition, first printing, Prestressed Concrete Institute, Chicago, Ill. (1972).

24. Manual of Steel Construction, Seventh Edition, American Institute of Steel Construction, New York, N. Y.

25. Modern Steels for Construction, Design Data from Bethlehem Steel, Booklet 2489, Bethlehem Steel Corporation, Bethlehem, Pa. 18016

26. Thurman, Allen G., Polymer Impregnated Precast Concrete Bridge Deck, Design, Fabrication and Erection Considerations (Unpublished Interim Report), Contract No. 14-06-D-7369, Bureau of Reclamation, United States Department of Interior, Denver, Colo. (1973).

27. Monfore, G. E., "A Review of Fiber Reinforcement of Portland Cement Paste, Mortar, and Concrete," Journal, PCA Research and Development Laboratories, Vol. 10, No. 3, Sept., 1968, pp. 43-49.

28. Shah, S. P., "New Reinforcing Materials in Concrete", Journal of the American Concrete Institute, Proceedings Vol. 71, No. 5, May, 1974, pp. 257-262.

29. ACI Committee 224, "Control of Cracking in Concrete Structures", Journal of the American Concrete Institute, Proceedings, Vol. 69, No. 12, Dec., 1972, pp. 717-753.

30. Goldberg, D. T., Jaworski, W. E. and Gordon, M. D., "Lateral Support Systems and Underpinning" Federal Highway Administration Offices of Research and Development, April, 1976. Available from the National Technical Information Service, Springfield, VA 22161 in three volumes: FHWA-RD-75-128, Vol. I, "Design and Construction"; -129, Vol. II, "Design Fundamentals" and Vol. III, "Construction Methods".

APPENDIX A

SAMPLE CALCULATIONS FOR THE DESIGN OF A PRECAST, PRESTRESSED LOAD BEARING WALL PANEL FOR A CUT-AND-COVER TUNNEL.

1. Notation:

A, A_g = Cross-sectional area of the gross section

A_{comp} = Area of the section in compression

A_{ps} = Area of prestressing steel

a = Depth of the equivalent stress block

b = Width of the compression face of the member

C = Coefficient relating to concrete strength

c = Distance from extreme fiber to neutral axis

d = Distance from extreme compression fiber to centroid of prestressing strand in tensile area

d' = Distance from extreme compression fiber to centroid of prestressing strand in compressive area

E_s = Modulus of elasticity of steel

e = Eccentricity of design load or prestress force parallel to axis measured from the centroid of the section

f_b = Maximum allowable compression

f'_c = Specified compressive strength of concrete

f_{ps} = Calculated stress in prestressing steel at design (ultimate) loads (tensile zone)

f_{pu} = Ultimate strength of prestressing steel

f_s = Stress in steel at design (ultimate) loads (compressive zone)

f_{se} = Effective stress in prestressing steel after losses

f_t = Maximum allowable tension

I, I_g = Moment of inertia of the gross section

M = $M_p + M_a + M_t$

M_a = Moment caused by eccentricity of applied axial loads

M_p = Moment caused by eccentricity of prestress force

M_t = Moment caused by applied transverse loads

M_u = Applied or allowable design (ultimate) moment

M_{uo} = Allowable design (ultimate) moment with no axial force present

P = $P_p + P_a$

P_a = Axial load caused by applied loads

P_p = Axial load caused by prestress force

P_u = Applied or allowable design (ultimate) axial load

P_{uo} = Design (ultimate) concentric axial load

y' = Distance from extreme compression fiber to centroid

y_t = Distance from extreme compression fiber to C.G. of the section

Z = Section modulus of the section

β_1 (beta) = 0.85 for concrete strengths, f'_c, up to 4000 psi and reduced at a rate of 0.05 for each 1000 psi of strength in excess of 4000 psi

ε_{ps} (epsilon) = Strain of prestressing strand in tensile area

ε_s = Strain of steel in compressive area

ϕ (phi) = Capacity reduction factor

$\bar{\omega}_p$ (omega) = $A_{ps}\ f_{pu}/bdf'_c$

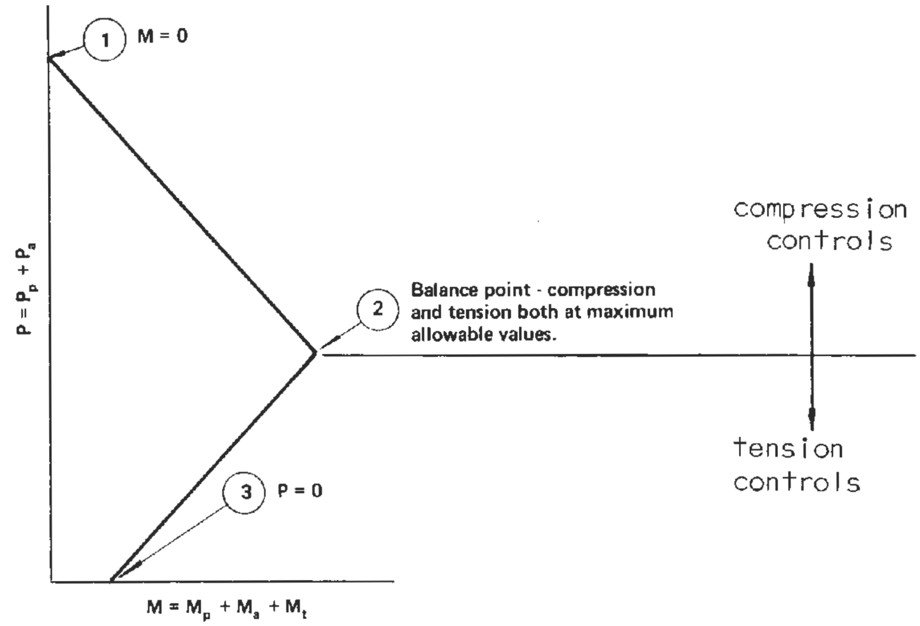


Fig. 74 Typical elastic interaction diagram

2. Construction of elastic interaction diagram for symmetrical prestressed concrete beam columns. (see Fig. 74)

Sign convention: + = compression

- = tension

Point 1 , $M = 0$

$P/A = f_b$; $P = f_b A$

Point 2 , Balance point

$$P/A + M/Z = f_b$$

$$\underline{P/A - M/Z = f_t}$$

$$2\ P/A = f_b + f_t$$

$$P = (f_b + f_t)\ A/2$$

$$M = (P/A - f_t)\ Z = Z\ (f_b - f_t)/2$$

Point 3 , $P = 0$

$M = -f_t Z$

3. Example 1: Construction of elastic interaction diagram. Given the section shown in Fig. 75.

$f'_c = 6000$ psi

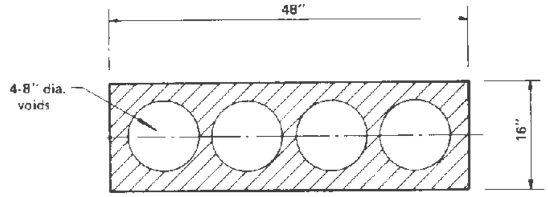


Fig. 75 Section for Example 1

Construct interaction curves for:

$f_b = 0.40\ f'_c = 0.40\ (6000) = 2400$ psi

and

$f_t = 0,\ -3\sqrt{f'_c} = -232$ psi, $-6\sqrt{f'_c} = -465$ psi

Solution:

Section properties:

$A = 48\ (16) - 4\ (\pi 8^2/4) = 567\ \text{in.}^2$

$I = bd^3/12 - 4\ (\pi d^4/64) = 48\ (16)^3/12 - \pi(8)^4/16$

$= 15{,}580\ \text{in.}^4$

$Z = I/c = 15{,}580/8 = 1947\ \text{in.}^3$

Point 1

$P = f_b A = 2400\ (567) = 1{,}360{,}800$ lb $= 1361$ kips

Point 2

for $f_t = 0$

$P = (f_b + f_t)\ A/2 = (2400 + 0)(567)/2 = 680{,}400$ lb

$= 680.4$ kips

$M = (P/A - f_t)\ Z = (680.4/567 - 0)(1947)$

$= 2336$ in.-kips $= 195$ ft-kips

for $f_t = -3\sqrt{f'_c} = -232$ psi

$P = (2400 - 232)(567)/2 = 614{,}600$ lb $= 614.6$ kips

$M = (614.6/567 + 0.232)(1947) = 2562$ in.-kips

for $f_t = -6\sqrt{f'_c} = -465$ psi

$P = (2400 - 465)(567)/2 = 548{,}600$ lb $= 548.6$ kips

$M = (548.6/567 + 0.465)(1947) = 2789$ in.-kips

Point 3

for $f_t = 0$: $M = 0$

for $f_t = -232$ psi : $M = 0.232\ (1947) = 451.7$ in.-kips

for $f_t = -465$ psi : $M = 0.465\ (1947) = 905.4$ in.-kips

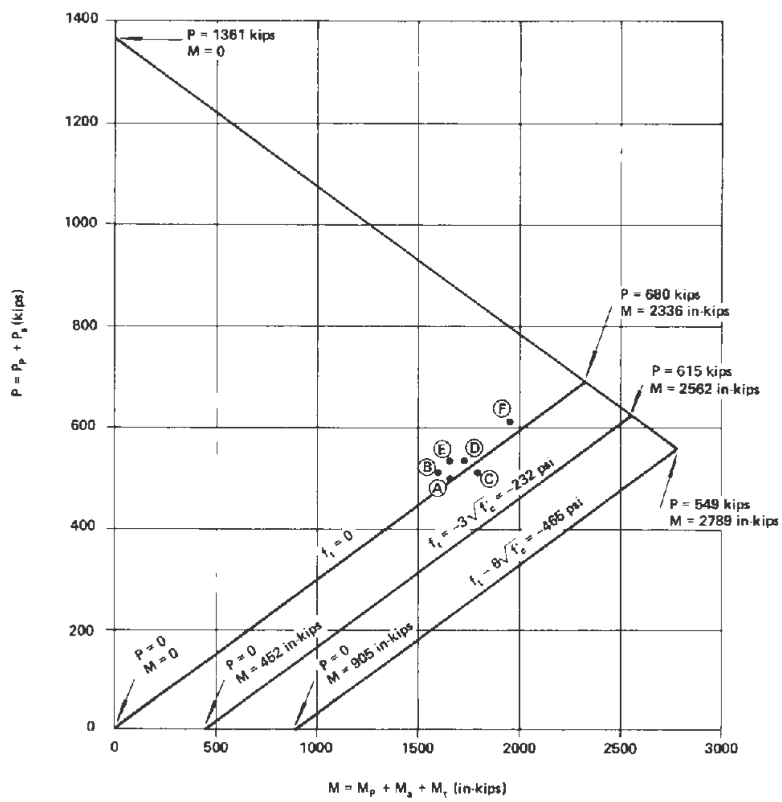


Fig. 76 Elastic interaction diagram for Example 1

4. Example 2: Use of elastic interaction diagram. Given the following data, determine the number of prestressing strands re-required in the section shown in Fig. 75:

Use 1/2" diameter prestressing strands

A_{ps} = 0.153 sq. in. per strand

f_{pu} = 270,000 psi

Assume 35,000 psi loss of prestress

Stress strands to 0.7 f_{pu}

Loading data per 4 ft wide section (see Fig. 77a)

(Final permanent loading condition)

P_{a1} = 20 kips D.L., 50 kips L.L.

P_{a2} = 20 kips D.L., 20 kips L.L.

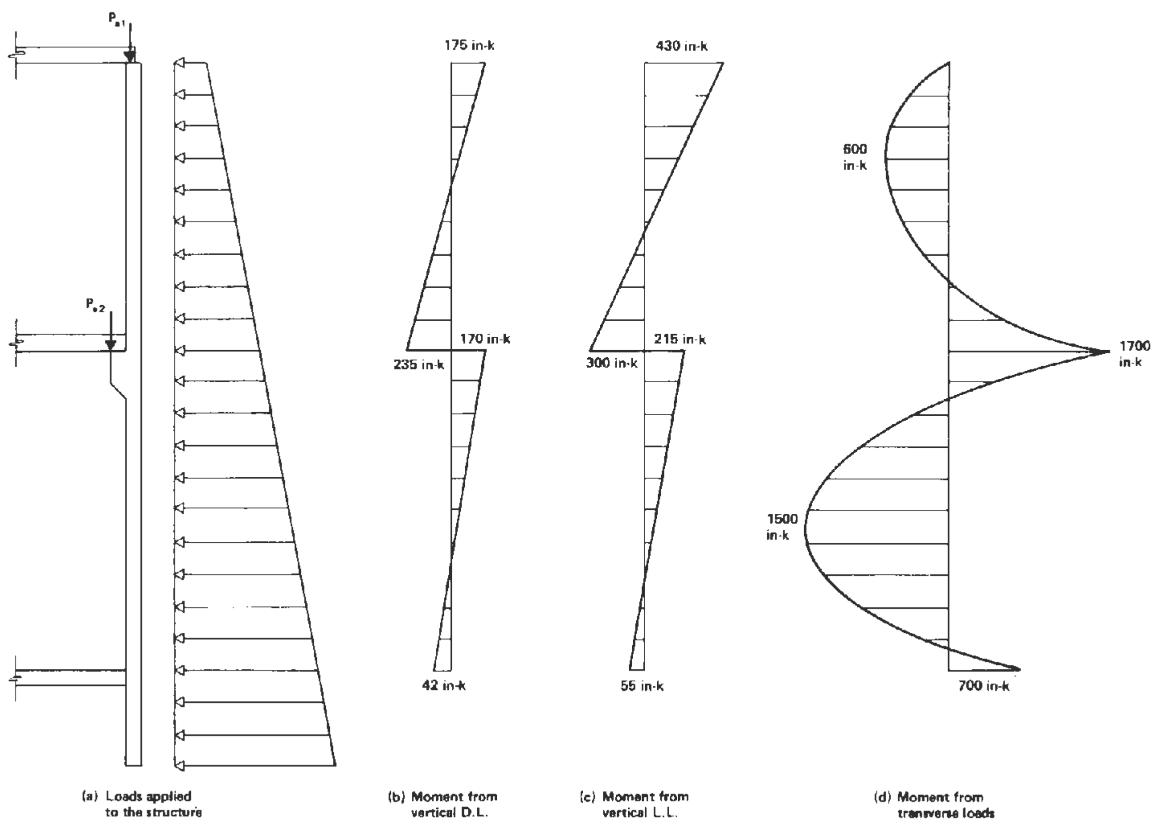


Fig. 77 Loads and moments for Example 2

A detailed analysis shows the moments to be as shown in Fig. 77 b, c, and d.

Maximum combined moments are as follows:

Under horizontal loads and vertical dead load, the maximum negative moment, $M_a + M_t = -170 - 1700 = -1870$ in.-kips, and the maximum positive moment (neglecting small compensating moment from axial loads), $M_a + M_t = +1500$ in.-kips.

Under total loads, the maximum negative moment is $-1870 - 215 = -2085$ in.-kips, and the maximum positive moment is the same, 1500 in.-kips.

For horizontal loads and vertical dead load, allow zero tension. For total loads allow $6\sqrt{f'_c} = 465$ psi tension.

Check dead load and horizontal load condition:

$$P = P_p + P_a = P_p + 20 + 20 = P_p + 40 \text{ kips}$$

$$M = M_p + M_a + M_t = M_p - 1870 \text{ in.-kips or } M_p + 1500 \text{ in.-kips}$$

Prestress force can be placed eccentrically to approximately balance the positive and negative moments.

Then $M_p \approx \dfrac{1870 - 1500}{2} = 185$ in.-kips, and

$$M = 1500 + 185 = 1685 \text{ say } 1700 \text{ in.-kips}$$

From interaction diagram, Min $P \approx 500$ kips (pt A)

$$P_p = P - P_a = 500 - 40 = 460 \text{ kips}$$

$$\text{Optimum eccentricity} = \frac{185}{460} = 0.40 \text{ in.}$$

Select prestressing strands:

P_p (per strand) = [0.7 (270) - 35] (0.153) = 23.6 kips per strand

No. of strands required = $\frac{460}{23.6}$ = 19.5 - Try 20 strands

Assume strands are 2" from face or 6" from center. Try 9 strands in inside face and 11 strands in outside face.

e = [11(6) - 9 (6)] /20 = 0.60 in.

P = 20 (23.6) + 40 = 472 + 40 = 512 kips

M = 472 (0.60) - 170 - 1700 = -1587 in.-kips B

or 472 (0.60) + 1500 = 1783 in.-kips C

Maximum allowable = 512/680 (2336) = +1759 in.-kips

This shows that there would be a very small amount of tension, and the designer may decide it is adequate. However, to assure no tension, add one more strand to the inside face:

e = [11 (6) - 10 (6)] /21 = 0.29 in.

P = 21 (23.6) + 40 = 496 + 40 = 536 kips

M = 496 (0.29) - 170 - 1700 = -1726 in.-kips D

or 496 (0.29) + 1500 = +144 + 1500 = +1644 in.-kips E

Check the total load condition:

P = 536 + 50 + 20 = 606 kips

$M = M_p + M_a + M_t$ = +144 - 170 - 215 - 1700 = 1941 in.-kips F

Final section is as shown in Fig. 78.

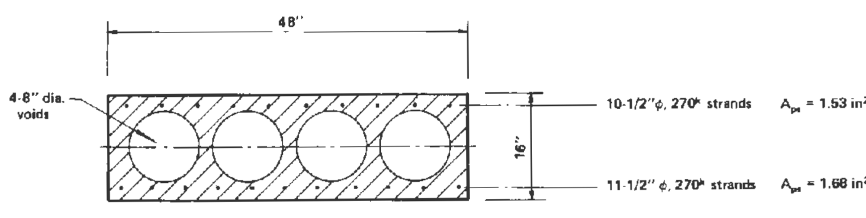


Fig. 78 Final design - Example 2

3. Construct ultimate interaction curve using method shown in Appendix B.

Given: $f_c^{'} = 6000$ psi

$f_{pu} = 270$ ksi

$E_s = 27 \times 10^6$ psi - 27,000 ksi

$f_{se} = 0.7\ (270) - 35 = 154$ ksi

$\phi = 1.0$ for precast, prestressed members

$d = 14''$

$d^{'} = 2''$

Point #1, $M_u = 0$

$$P_{uo} = 0.85\ f_c^{'}\ A_g - \Sigma\ [A_{ps}\ (f_{se} - 0.003\ E_s)]$$

$$= 0.85\ (6)(567) - 3.21\ [154 - 0.003\ (27{,}000)] = 2892 - 234$$

$$= 2658 \text{ kips}$$

Point #2, $P_u = 0$ (bending only)

$$f_{ps} = f_{pu}\ (1 - 0.5\ A_{ps}\ f_{pu}/bd\ f_c^{'}) \quad \text{(Eq. 18-3, ACI 318-71)}$$

$$= 270\ [1 - 0.5\ (1.68)(270)/(48)(14)(6)] = 255 \text{ ksi}$$

$$a = A_{ps}\ f_{ps}/0.85\ f_c^{'}\ b = 1.68\ (255)/(0.85)(6)(48) = 1.750 \text{ in.}$$

$$M_{uo} = A_{ps}\ f_{ps}\ (d - a/2) = 1.68\ (255)(14 - 1.75/2)$$

$$= 5623 \text{ in.-kips} = 469 \text{ ft-kips}$$

Note: Calculation of this point in this manner neglects the effect of the steel above the neutral axis, a slightly unconservative approximation. However, the calculation of f_{ps} by Eq. 18-3 of ACI 318-71 is conservative. The net effect of the two approximations is a negligible error.

Point #3, set a = 4 in. (Fig. 79)

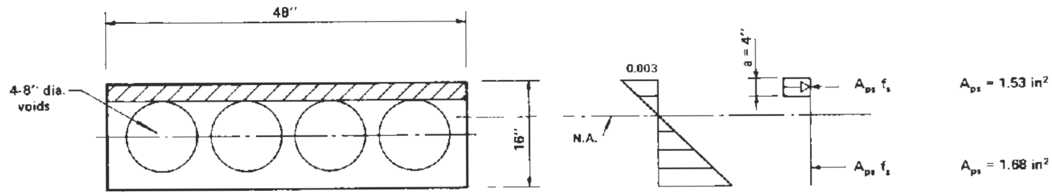


Fig. 79 Data for ultimate interaction diagram

β_1 for 6000 psi = 0.85 - 2 (0.05) = 0.75

$$c = a/\beta_1 = 4/0.75 = 5.33$$

f_{se}/E_s = 154/27000 = 0.00570

From Fig. 82:

$$\varepsilon_s = 0.00570 - \frac{0.003}{5.33}(5.33 - 2) = 0.00383$$

f_s = 0.00383 (27,000) = 103.29 ksi (See Fig. 83)

$$\varepsilon_{ps} = 0.00570 + \frac{0.003}{5.33}(14 - 5.33) = 0.01058$$

f_{ps} from stress-strain curve, Fig. 83 = 249 ksi

$$P_u = (A_{comp})\ 0.85\ f'_c - A_{ps}\ f_s - A_{ps}\ f_{ps}$$

$$= (4 \times 48)(0.85)(6) - 1.53\ (103.29) - 1.68\ (249)$$

$$= 979 - 158 - 418 = 403 \text{ kips}$$

$$M_u = (A_{comp})(y_t - y')\ 0.85\ f'_c - A_{ps}\ f_s\ (y_t - d') + A_{ps}\ f_{ps}\ (d - y_t)$$

$$= 192\ (8-2)(0.85)(6) - 1.53\ (103.29)(8-2) + 1.68\ (249)(14-8)$$

$$= 5875 - 948 + 2510 = 7437 \text{ in.-kips} = 620 \text{ ft-kips}$$

Point #4, set a = 12 in. (Fig. 80)

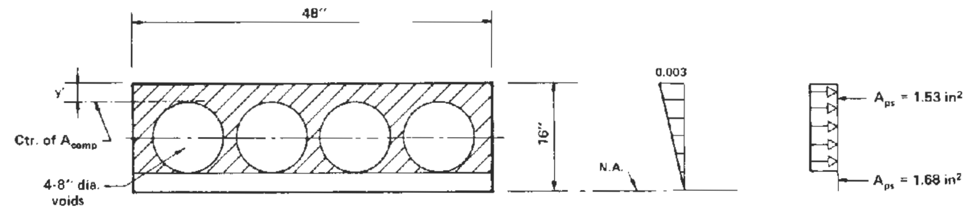


Fig. 80 Data for ultimate interaction diagram

$$A_{comp} = 12 \times 48 - 4\pi\ (4)^2 = 576 - 201.06 = 374.94 \text{ sq in.}$$

$$y' = \frac{576\ (6) - 201.06\ (8)}{374.94} = \frac{1847.5}{374.94} = 4.93 \text{ in.}$$

$$c = 12/0.75 = 16 \text{ in.}$$

From Fig. 82:

$$\varepsilon_s = 0.00570 - \frac{0.003}{16}(16 - 2) = 0.003075$$

$$f_s = 0.003075\ (27{,}000) = 83.0 \text{ ksi (Fig. 83)}$$

$$\varepsilon_{ps} = 0.00570 - \frac{0.003}{16}(16 - 14) = 0.005325$$

$$f_{ps} = 0.005325\ (27{,}000) = 143.8 \text{ ksi (Fig. 83)}$$

$$P_u = 0.85\ (6)(374.94) - 1.53\ (83.0) - 1.68\ (143.8) = 1544 \text{ kips}$$

$$M_u = 0.85\ (6)(374.94)(8 - 4.93) - 1.53\ (83.0)(8 - 2)$$
$$+ 1.68\ (143.8)(14 - 8)$$

$$= 5870 - 762 + 1450 = 6558 \text{ in.-kips} = 546 \text{ ft-kips}$$

Point #5, set a = 8 in. (Center of section)

$$A_{comp} = 8 \times 48 - 4\ (\pi)(4)^2/2 = 384 - 100.53 = 283.47 \text{ sq in.}$$

$$y' = \frac{384\ (4) - 100.53\ [4(1 - 4/3\pi) + 4]}{283.47} = \frac{902.43}{283.5} = 3.18 \text{ in.}$$

$$c = 8/0.75 = 10.67 \text{ in.}$$

$$\varepsilon_s = 0.00570 - \frac{0.003}{10.67}(10.67 - 2) = 0.00326$$

$$f_s = 0.00326\ (27{,}000) = 88.1 \text{ ksi (Fig. 83)}$$

$$\varepsilon_{ps} = 0.00570 + \frac{0.003}{10.67}(14 - 10.67) = 0.00664$$

$$f_{ps} = 0.00664\ (27{,}000) = 179.2 \text{ ksi (Fig. 83)}$$

$$P_u = 0.85\ (6)(283.47) - 1.53\ (88.1) - 1.68\ (179.2) = 1010 \text{ kips}$$

$$M_u = 283.47\ (6)(0.85)(8 - 3.18) - 1.53\ (88.1)(8 - 2)$$
$$+ 1.68\ (179.2)(14 - 8)$$
$$= 6968 - 809 + 1806 = 7965 \text{ in.-kips} = 664 \text{ ft-kips}$$

Point #6, set a = 16 in.

$$A_{comp} = A_g = 567 \text{ in.}^2$$

$$c = 16/0.75 = 21.33 \text{ in.}$$

$$\varepsilon_s = 0.00570 - \frac{0.003}{21.33}(21.33 - 2) = 0.00298$$

$$f_s = 0.00298\ (27{,}000) = 80.5 \text{ ksi (Fig. 83)}$$

$$\varepsilon_{ps} = 0.00570 - \frac{0.003}{21.33}(21.33 - 14) = 0.00467$$

$$f_{ps} = 0.00467\ (27{,}000) = 126.1 \text{ ksi (Fig. 83)}$$

$$P_u = 0.85\ (6)(567) - 1.53\ (80.5) - 1.68\ (126.1)$$
$$= 2892 - 123 - 212 = 2557 \text{ kips}$$

$$M_u = 0.85\ (6)(567)(8 - 8) - 1.53\ (80.5)(8 - 2)$$
$$+ 1.68\ (126.1)(14 - 8)$$
$$= 0 - 739 + 1271 = 532 \text{ in.-kips} = 44 \text{ ft-kips}$$

For the sample problem:

Of the 1700 in.-kips negative moment shown in Fig. 77, 500 in.-kips is caused by soil pressure and surcharge and 1200 in.-kips is caused by the water table. See Section VI for load factors.

P_u = 1.3 (20 + 20) + 2.17 (50 + 20) = 204 kips

M_u = 1.3 (170) + 2.17 (215) + 1.7 (500) + 1.4 (1200)

= 221 + 467 + 850 + 1680 = 3218 in.-kips = 268 ft-kips

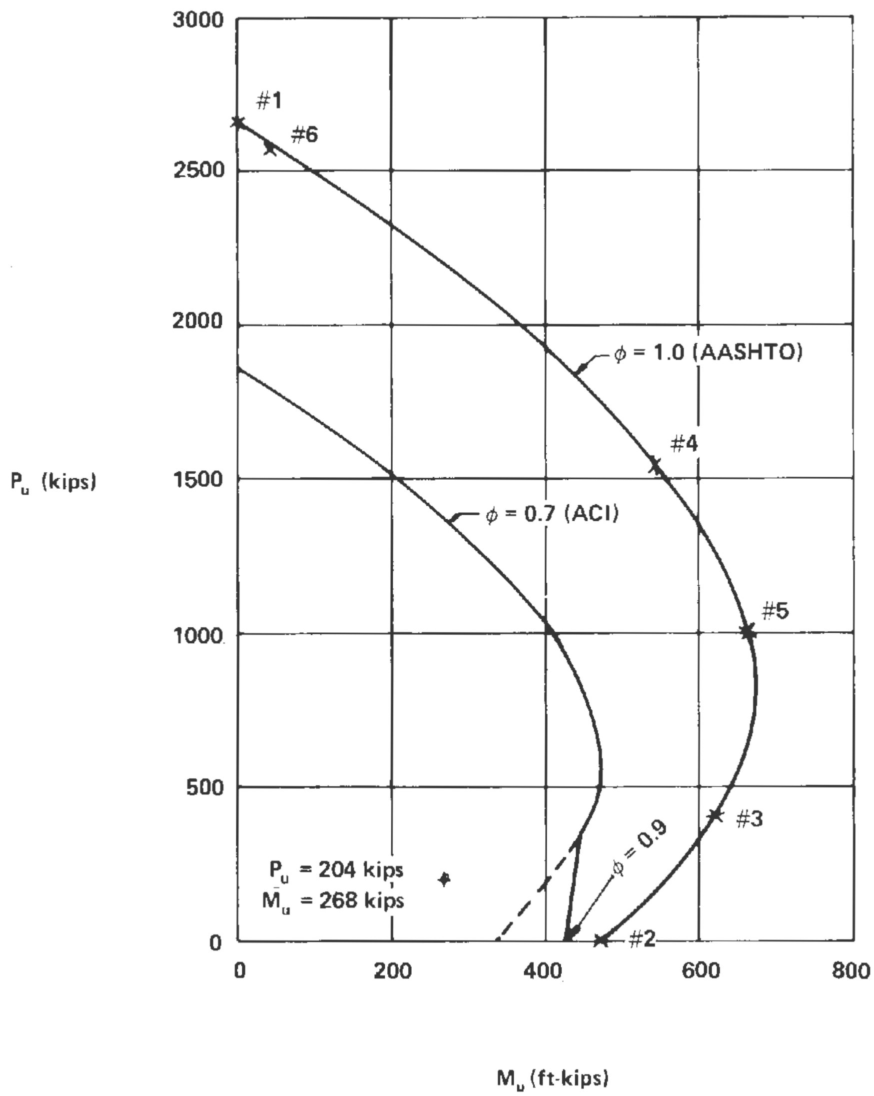


Fig. 81 Ultimate interaction diagram

APPENDIX B

CONSTRUCTION OF STRENGTH (ULTIMATE) INTERACTION CURVES FOR PRESTRESSED CONCRETE BEAM-COLUMNS

Note: Assumptions are based on Sections 10.2 and 18.3 of ACI 318-71

Step 1. Select a value of "a" or "c" for each point on the interaction curve. Determine the corresponding "a" or "c" from the equation $a = \beta_1 c$.

Step 2. Determine the value of A_{comp} from the geometry of the section (shaded portion in Fig. 82).

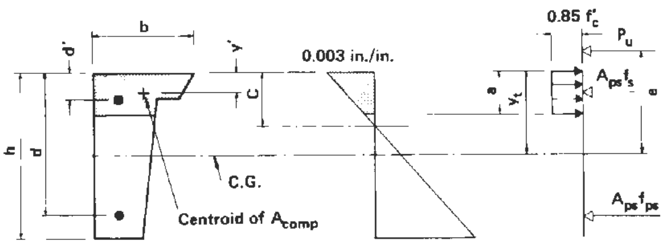
Step 3. Determine the initial strain in the strand caused by the prestressing = f_{se}/E_s.

Step 4. Determine the strain in the strand caused by external loading by similar triangles as shown in Fig. 82 (a). The total strain is then the initial strain ± the strain causec by external loading.

Step 5. Determine the stress in the strand from the stress-strain curve (Fig. 83).

Step 6. Calculate the values of P_u and M_u for each point selected by statics as shown in Fig. 82 (a).

Note that in the special case shown in Fig. 82 (b), the neutral axis can be outside the section, and the relationships are still valid. In the case of axial load only, the value of P_u is calculated by the equation shown in Fig. 82 (c).



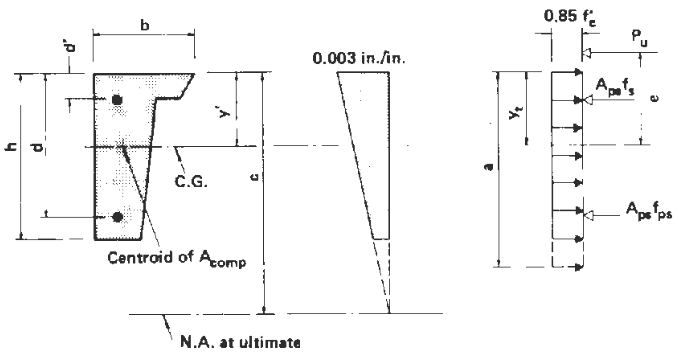
$$\epsilon_s = \left[f_{se}/E_s - \frac{0.003}{c}(c-d')\right] \leq 0.035 \text{ in./in.}$$

$$\epsilon_{ps} = \left[f_{se}/E_s + \frac{0.003}{c}(d-c)\right] \leq 0.035 \text{ in./in.}$$

$$P_u = (A_{comp})\, 0.85 f'_c - A_{ps} f_s - A_{ps} f_{ps}$$

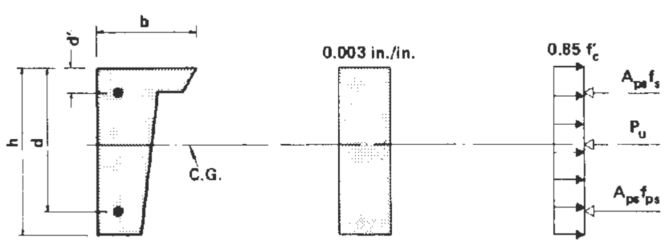
$$M_u = P_u e = (A_{comp})(y_t - y')\, 0.85 f'_c - A_{ps} f_s (y_t - d') + A_{ps} f_{ps} (d - y_t)$$

(a) Basic relationships



$A_{comp} \approx A_g$ if $a > h$

(b) Special case with Neutral Axis outside of the section



$$P_u = 0.85 f'_c (A_{comp}) - \Sigma\, [A_{ps}(f_{se} - 0.003 E_s)]$$

$$A_{comp} \approx A_g$$

(c) Special Case when $M_u = 0$

Fig. 82 Strain compatibility relationships for prestressed concrete beam-columns

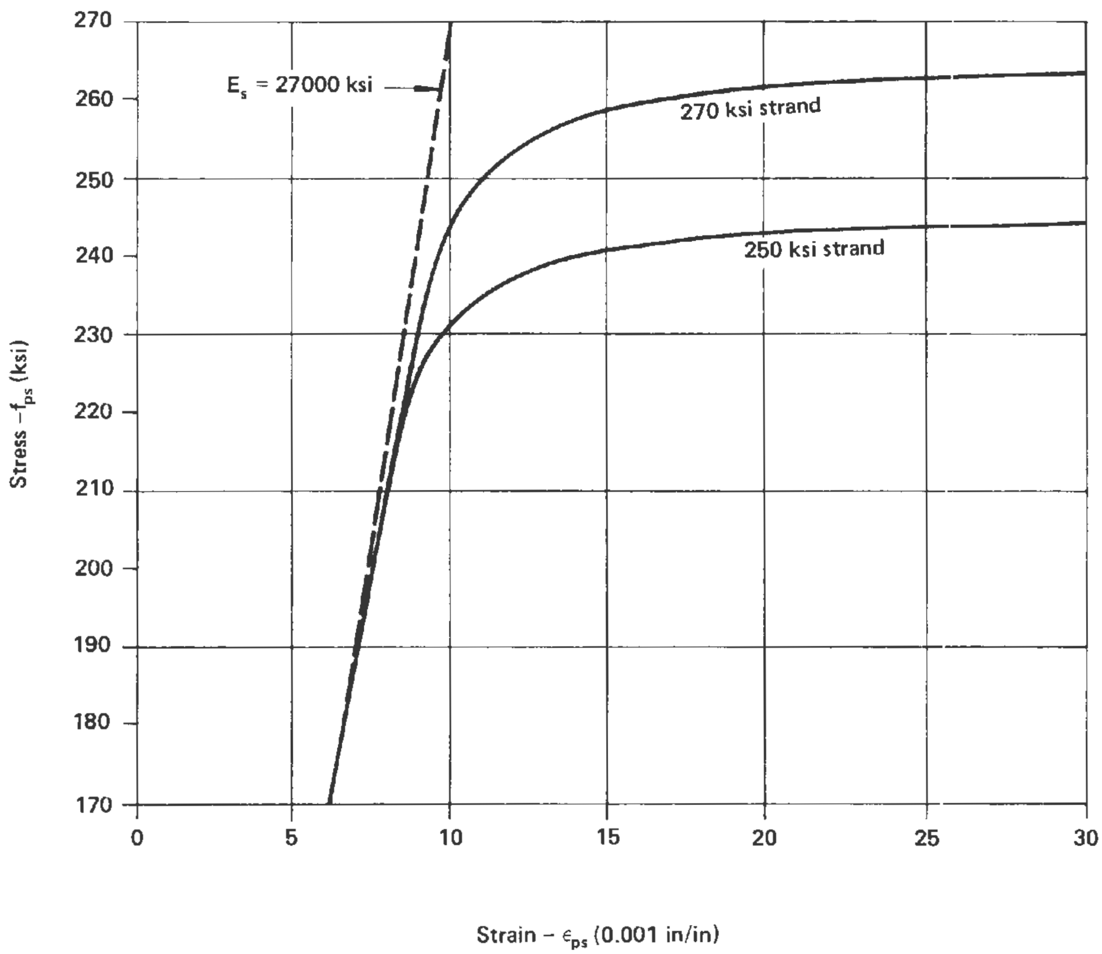


Fig. 83 Typical stress-strain curves for 7-wire prestressing strand.

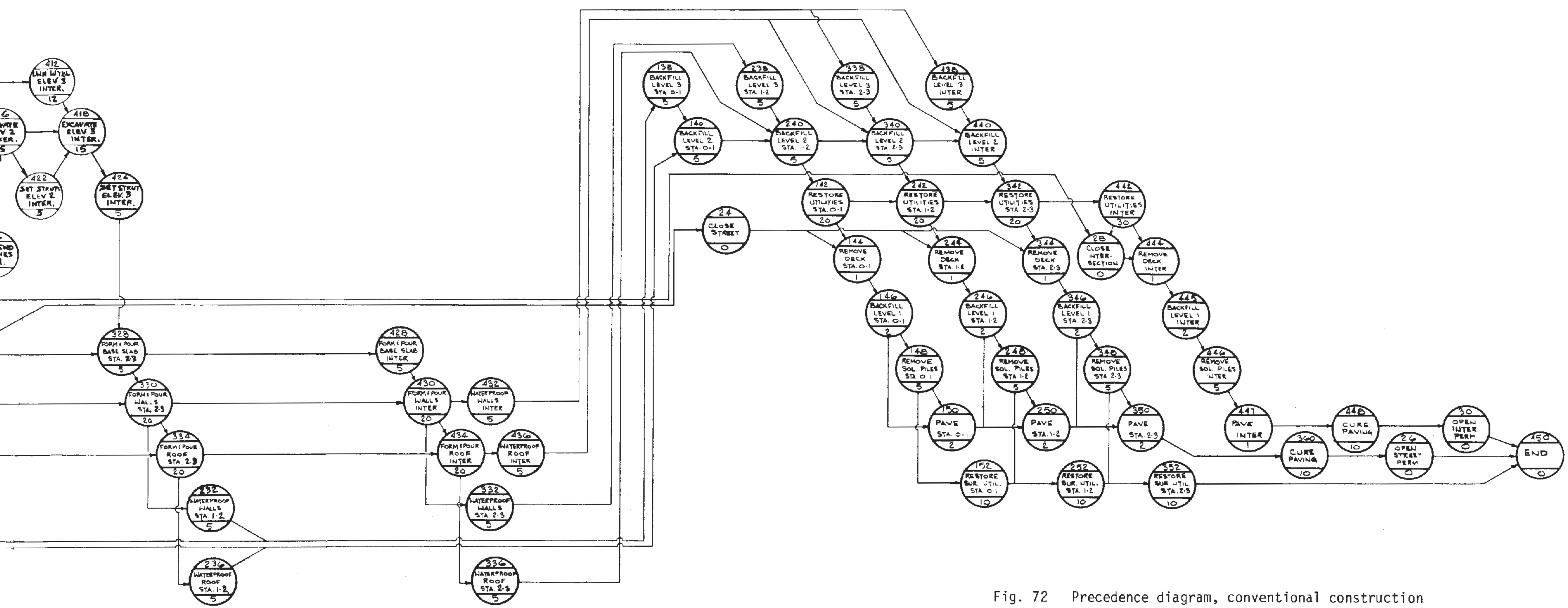


Fig. 72 Precedence diagram, conventional construction