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December 4, 1995

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THROUGH: FRANKLIN E. WHITE

FROM: KIM KIMBALL K. Kimball
EXECUTIVE OFFICER, STRATEGIC PROJECTS

SUBJECT: TECHNICAL APPENDIX TO EISENSTEIN PANEL REPORT

As promised at the November 17, 1995 Rail Workshop, attached is a copy of the Technical Appendix to the Eisenstein Panel Report. For your convenience, a copy of the Panel's report, which includes a two page executive summary, accompanies this appendix.

Attachment

Los Angeles County Metro Rail Project: Report on Tunneling Feasibility and Performance

Prepared by:

Dr. Dan Eisenstein
Dr. Geoffrey Martin
Dr. Harvey Parker

for
Los Angeles County
Metropolitan Transportation Authority

November 1995

EXECUTIVE SUMMARY

The geological and geotechnical environment along the existing and proposed corridors of Los Angeles Metro is compatible with safe and economical underground construction. The geology in Los Angeles is either equal or more favorable to tunneling than in most other cities because the ground is generally drier and more competent with a minimum of obstructions. Conditions which are less favorable include the persistent methane problem, the earthquake fault displacement potential and the unique H_2S hazard.

It has been concluded from a review of the seismological environment in the Los Angeles area, together with the seismic design and analysis methods being used by MTA and case histories of worldwide tunnel performance during past earthquakes, that earthquakes are not a governing factor in assessing tunneling feasibility in Los Angeles. However, in developing seismic design criteria for proposed new tunnels, it is recommended that results of recent research related to the seismic hazard in the Los Angeles region be reviewed.

A survey of worldwide practice of construction of shallow urban tunnels revealed that a vast majority of these tunnels are excavated using tunnel boring machines, most of them with positive face control (slurry and earth pressure balance machines). A smaller portion of urban tunnels are constructed with methods using open face, but in most of these cases the ground is conditioned before excavation. The U.S. practice in general still prefers this approach and this is why the Los Angeles Metro tunnels are in the latter category.

A worldwide survey of performance of urban tunneling has shown that in about 44 percent of the tunnels surveyed, the performance in terms of ground control was classified as Category 1 (without problems). In about 14 percent of the contracts, performance was classified as Category 2 (minor problems) and about 42 percent were classified as Category 3 (significant problems).

Applying the criteria cited above for the Los Angeles Metro, the performance levels achieved so far have placed about 50 percent of the tunneling contracts into Category 1, 12.5 percent into Category 2, and 37.5 percent into Category 3. This ratio is approximately equal to or slightly better than the worldwide performance.
The cost of soft ground tunnels in Los Angeles is low relative to prices worldwide. These low costs may be a reason for some of the construction problems that have been experienced.

It is recommended that for future tunneling, consideration be given to application of earth pressure balance tunnel boring machines capable of operating in two modes -- as an open face machine in competent ground (e.g., the Puente Formation) and with positive face control in less competent ground (e.g., alluvial soils). The choice of whether to permit an open face shield in preconditioned ground or require an earth pressure balance machine will depend on the degree of risk MTA wishes to share and on the overall costs.

It is recommended that ground control be established as the governing design and construction criterion on the existing and future tunneling contracts, with firmly set rules about monitoring and interpreting ground deformation data as well as practical steps to be taken immediately where deformations exceed the permissible limits. MTA should consider the cost and benefits of specifying less risky construction methods and methods which minimize construction impact to the public, even if their initial cost may be somewhat more expensive.

It is recommended that fundamental technical principles and policies for the project are formulated and regularly overseen at the owner's level by a small permanent staff highly experienced in tunneling assisted by a small independent technical review board reporting directly to MTA.

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PREFACE

PREFACE

In response to questions posed by Mayor Riordan and the Los Angeles County Metropolitan Transportation Authority (MTA) Board regarding the feasibility of tunneling in Los Angeles ground conditions, MTA convened a three-member Geotechnical Tunneling Panel in August 1995. The members of the panel are:

Dr. Z. Dan Eisenstein
Professor of Civil Engineering
University of Alberta
Edmonton, Alberta

Dr. Geoffrey R. Martin
Professor and Chairman of Department of Civil Engineering
University of Southern California
Los Angeles, California

Dr. Harvey W. Parker
Senior Vice President
Shannon & Wilson, Inc.
Seattle, Washington

Dr. Eisenstein served as Chairman of the Panel. The panel met for the first time with their MTA Project Manager, Mr. Jerry Baxter, in Los Angeles on August 9, 1995. The panel was commissioned with the following two primary objectives:

1. Evaluating and comparing problems encountered in tunneling projects worldwide, with those problems encountered in constructing MTA tunnels to date.

2. Assessing whether the geological and geotechnical conditions of the Los Angeles region are inconsistent with tunneling.

Additional details on the scope and methodology of the investigation are contained in the appendix to the report.

The panel met in Los Angeles four times during the course of the study documented in this report. The panel visited tunnel projects that are still under construction and were briefed by many parties to obtain input for this report. In addition, the panel made independent inquiries and initiated requests for information that they felt was required in the course of their work. Too many people assisted the panel to be named individually but their unselfish input is gratefully acknowledged. Following the retirement of Mr. Baxter, Mr. Michael Gonzalez was assigned as the MTA Project Manger.

In order to make the report readable to a broad cross section of readers, the panel decided to keep the main report short and concise, putting the technical detailed backup in the appendix to the report. The main report contains brief summaries of the various issues investigated by the panel together with all of the conclusions and recommendations.

The appendix follows the same organization as the report and it contains detailed descriptions of the issues and discussions which support the conclusions and recommendations of the report. Finally, all tables, figures, and references are bound in the back of the appendix.

REPORT OF THE GEOTECHNICAL PANEL

Background

The purpose of this report is to (1) assess the construction feasibility of future tunneling corridors for the Los Angeles Metro; (2) evaluate the geotechnical and construction performance of the tunnels constructed so far, including comparisons with similar projects worldwide, and (3) to recommend measures to improve performance in relation to future construction.

The assessment of tunneling feasibility was initiated by conducting a review of the geological, geotechnical and seismological conditions along the existing and future lines of the Metro project to establish a baseline for comparative studies. It is understood that future planned tunnel alignments include those of the Eastside Extension, the Mid-City Extension, the San Fernando Valley Extension, and possible extensions to the intersection of Wilshire Boulevard and the 405 freeway. The performance evaluation is based on comparison of the Los Angeles tunneling contracts with published data from similar projects worldwide. The criteria used in the comparison are the surface settlement levels and the occurrence of major ground instabilities or failures. Also considered are cost comparisons, risk factors, and earthquake performance. Finally, recommendations for potential improvements are made in the technical and management areas.

Geological and Geotechnical Considerations

A wide range of geologic and geotechnical conditions are encountered along existing and proposed alignments of the tunnel segments. In the Los Angeles Basin and the San Fernando Valley areas, conditions encountered include both recently deposited and older alluvial soil deposits (primarily silts, sands, and gravels) and consolidated claystones, siltstones and sandstones ("soft" rocks) of the Fernando, Puente and San Pedro Formations. Hard rock tunneling conditions are encountered in the Topanga Formations in the Santa Monica Mountains. Overall, such ground conditions are not unique to Los Angeles, and are encountered in many tunneling projects throughout the world. The general absence of extensive deposits of soft clays and unstable running sands is a positive factor.

The absence of conditions leading to groundwater inflow into tunnels during construction over much of the alignment is also a positive factor in relation to tunneling costs and risks. Groundwater conditions possibly requiring dewatering are encountered, however, in a few localized areas associated with the Los Angeles River in downtown areas of the city (including a short western portion of the Eastside Extension) and over small sections of the Hollywood Boulevard and North Hollywood tunnels. The Santa Monica Mountain Range is not a groundwater basin, although the extensive joint and fracture systems are expected to transmit considerable quantities of water into the tunnel for short periods of time. Along the proposed San Fernando Valley Extension, groundwater is higher than the planned invert in only localized sections.

The presence of gas is always a concern where tunnels are planned. The presence of methane is not new to tunneling projects, and has been successfully overcome in the Segment 1 tunnels. However, the presence of H_2S is unique to the MTA tunneling projects. Whereas, H_2S has not been encountered in tunnel segments constructed to date, a potential H_2S presence is impacting the planned Mid-City Extension. To eliminate this problem, various options are presently being evaluated to overcome an odor concern.

Earthquake Issues

As is well known, Los Angeles is located in an area of high seismic potential that has experienced ground shaking from numerous large earthquakes in historical times. Widely recognized surface faults having major impact on the MTA alignments in terms of ground shaking potential include the Hollywood, Santa Monica, and Malibu Coast faults, and the Raymond and Newport Inglewood faults. As a result of research over the past decade, active blind thrust fault systems that do not cut the earth's surface are known to underlie the San Fernando Basin, the Santa Monica Mountains, and the Los Angeles Basin (Elysian Park thrust fault). These fault systems are also now recognized as being a major threat to the Los Angeles metropolitan area. A blind thrust fault was responsible for the Northridge earthquake.

The Maximum Design Earthquake (MDE) adopted for MTA tunnels and stations designed to date (a ground shaking level corresponding to a peak ground acceleration of 0.6g) recognizes the threats described above from surface faults. However, the more recent studies documenting the importance of blind thrust faults were not available at the time earthquake design criteria were established. A new study to re-visit blind thrust earthquake scenarios is under consideration for the Eastside Extension, and could lead to changes in design earthquake ground shaking levels. We recommend that such a study be performed.

Based on a survey of the performance of tunnels in past earthquakes, the panel has concluded that bored tunnels (either unlined rock tunnels or tunnels in alluvial ground constructed with reinforced-concrete linings) have performed exceptionally well in past earthquakes, including very strong earthquakes with estimated peak ground accelerations up to the order of 0.8g. These levels of acceleration could be expected, for example, from a Northridge-like M6.7 earthquake beneath the City of Los Angeles. No case has been identified where structural damage to bored tunnels has occurred from ground shaking effects in ground conditions similar to those in Los Angeles.

The only cases where major damage has occurred can be related to tunnels crossing faults where the earthquake has generated relative displacement across the structure. Damage similar to that experienced by stations and cut-and-cover box tunnel sections during the Kobe, Japan earthquake is not expected to MTA structures because of the more appropriate seismic design criteria being used by MTA.

It is clear from the historical record documenting the performance of bored tunnels in past earthquakes that they are much less vulnerable to earthquakes than aboveground structures. The earthquake design philosophy and seismic design and analysis methods presently adopted for MTA tunnels reflect state-of-the-art thinking and systematically address potential hazards induced by major seismic events. Detailed analyses conducted to date on the seismic response of MTA tunnels have shown they can accommodate large earthquake-induced ground distortions without damage.

In fact, it can be argued that since transit facilities in bored tunnels have demonstrated that they can operationally survive earthquakes better than surface structures, serious consideration should be given during rail transit planning to the use of tunnels over surface facilities where possible. Tunnels have the potential of functioning as important lifelines during and after a major earthquake.

No active faults were identified along the alignments for Segments 1 and 2. However, Segment 3 crosses the Hollywood fault and future westward extensions may cross the Santa Monica fault, both identified as active. Anticipated fault displacement during an MDE event on the Hollywood fault was estimated at 4.5 feet when design criteria were established. Design philosophy for fault crossings recognizes that it is difficult to prevent damage in a strong earthquake, given the magnitude of fault displacements. It is now widely accepted to "overbore" the tunnel when passing through or over a fault zone, so that if an earthquake-induced displacement occurs, the tunnel is still of sufficient diameter to fulfill its function after repairs. Such an approach is being adopted for the Segment 3 Hollywood Fault crossing.

The Eastside Extension fault investigations have not uncovered any clear expressions of near-surface strike-slip faulting that would result in abrupt displacement across the proposed tunnel alignment. However, studies to date have indicated that the asymmetrical fold comprising the Coyote Pass Escarpment which intersects the alignment is a potentially active tectonic structure. The escarpment could be uplifted during a future earthquake on an underlying blind thrust structure. The nature of the uplift and the magnitude of possible structural effects on the tunnel when it crosses the escarpment, together with any needed remedial design measures, are still under study.

It has been concluded from the review of the seismological environment in the Los Angeles area, together with the seismic design criteria and analysis methods being used by MTA and case histories of worldwide tunnel performance during past earthquakes, that earthquakes are not a governing factor in assessing tunneling feasibility in Los Angeles.

Available Tunneling Technologies

In present day tunneling, the methods for both the soft ground (soils and soft rock) and hard rock tunnels have moved from the stage of empirical craftsmanship to high technology levels. This relatively recent development is represented by tunnel boring machines with positive face control such as the slurry and the earth pressure balance shields, where ground stability and deformations are controlled, even in the most difficult ground, by processes inherent to the machine itself. With these technologies there is little or no need to condition the ground beforehand, e.g. by grouting.

On the other hand, the older tunnel boring machines with open face or the New Austrian Tunneling Method require that unstable or excessively deformable ground is treated before tunnel excavation can proceed. Both positive face control and open face machines are used in present soft ground tunneling. However, the first approach is becoming more and more the preferred solution because it offers a "blanket" type protection against unexpected ground conditions. Sometimes, when obstructions such as the steel tiebacks left over from basement construction of high-rise buildings on Wilshire Boulevard must be dealt with, an open face shield may still be the preferred choice. This is why the EPB TBM's are sometimes designed to operate in two modes, open face and EPB.

In hard rock tunneling, the trend is also towards tunnel boring machines, replacing the traditional drill and blast methods for reasons of environmental protection and economy. Though these hard rock machines reduce risk, they too are not completely risk-free. Blasting, although also not risk-free, is a tried and proven safe construction method that is still used.

Worldwide Tunneling Trends

Tunneling in Los Angeles is carried out in soft ground (soils), soft rock, and hard rock. Safe tunneling in soft ground requires either the use of tunnel boring machines with positive face control or conditioning the ground before tunneling, usually by grouting. It is the latter approach which has been used in Los Angeles so far. An extensive worldwide review of 74 case histories of soft ground urban tunneling over the last 20 years provided a useful base on which the Los Angeles experience can be evaluated. The review shows that most of the tunnels were built using some form of a tunnel boring machine, of which the vast majority were of the types with positive face control, such as slurry and earth pressure balance machines. The rest of the tunnels were constructed with open face TBMs, often combined with ground conditioning. The worldwide trend in present day tunneling is towards methods using positive face control, because they offer more effective protection against ground deformations and instability.

The trends in the choice of tunneling methods in the United States differs from the rest of the world. While TBMs in the U.S. are clearly the preferred technology today, only less than half of the TBMs belong to the categories with positive face control. More than half of the urban tunnels in the U.S. are constructed with open face TBMs, often combined with ground conditioning.

Risk

Even though all tunnel projects involve considerable risk, many tunnels and underground transit projects have been successfully constructed and put into service by managing this risk. Tunnel costs are high relative to surface construction because they reflect the uncertainties and risks that must be borne during construction. Although this construction risk is the most visible risk, transit projects also experience risks during planning, design, construction and operation.

Construction risks can be mitigated by improved geotechnical investigations, better contract packaging, improved risk sharing contracting practices, improved design and construction management, improved quality of construction, improved project direction and management, etc. These risks, particularly construction risks, must be recognized, admitted to, and understood in order to be managed. This will set manageable levels of expectation. All decision makers for any given tunnel, such as the designer, contractor, construction manager, MTA staff or Board should become aware of the importance and significance of the risks presented by the project. Often, but not always, risk can be reduced, although not eliminated, by being proactive. Often there is a cost associated with the reduction of risk. Though the real cost and the degree of risk reduction will never be determined accurately, trends can be established by conducting "what if" cost analyses.

A balance should be established by conscious decisions between risk and the cost of risk mitigating measures as well as the degree of risk sharing MTA desires. Ultimately, the decision as to what this balance should be should rest with MTA. The same "what if" cost analysis can be conducted for each risk-sharing statement in the specifications and, particularly, in the Geotechnical Design Summary Report (GDSR).

Costs

The panel evaluated costs for existing MTA tunnels together with a few other projects in North America and worldwide where acceptable cost data were available. The panel evaluated the cost of the few tunnels for which other bid costs (such as stations) did not obscure the evaluation. However, it was not possible during the time available to evaluate the overall project costs including stations or special structures or substantial ground improvement in lieu of underpinning.

It was found that the cost of soft ground tunnels in Los Angeles is low to average relative to costs in North American and very low relative to prices worldwide. On the other hand, the very low prices for tunnels may be partly responsible for some of the problems MTA is facing. These low bids are inherently associated with construction methods that may be more risky than the owner may now wish to accept. The design and contracting concept currently used by MTA results in attractive low bids. While contractors must be competitive, they should also be entitled to make a fair profit.

However, the low bidder possibly cannot afford the equipment for a special TBM that can guarantee minimization of settlement, or for positive full face control, or for digital alignment controls, or for special ground treatment methods. Instead, ground treatment, usually by grouting from the surface, has been paid for by MTA on an "as needed" basis to assure satisfactory settlement control is achieved. Unfortunately, it has been required more on a routine basis rather than on an "as needed" basis. Though this may have been a reasonable way to contract in the past, it may now be in the best interests of MTA to look at the cost and benefits of requiring or specifying more elaborate and less risky measures, especially for alignment and ground control.

Further, MTA should consider the cost benefit of requiring tunneling methods which have a lower impact on the public or adjacent property owners, such as minimizing the disruptive practices of grouting from the surface, by substituting chemical and/or compaction grouting from within the tunnel or by replacing them by use of the earth pressure balance method. These measures would result in higher bids but perhaps a lower final cost by minimizing change order and claims-type payments. A "what if" analysis should be conducted or reviewed in detail at a senior technical level by MTA so that there is a clear understanding by the owner what the risks are and what the likely cost of reducing some of the risks may be.

Local and MTA Tunnel Experience

A review of previous local tunneling projects indicates that tunneling in the local Los Angeles area is feasible, that tunneling conditions are generally favorable, and that tunnels can be constructed relatively inexpensively. However, several of these earlier tunnels experienced difficulties similar to or worse than difficulties experienced on MTA projects. This indicates to the panel that, though regrettable and not to be tolerated, the problems experienced by MTA tunnel projects are similar to or less serious than those previously experienced by the industry in the past. Problems experienced on previous tunnels included boulders, caving, sinkholes, methane, as well as an explosion and deaths which occurred in

1971. These problems and the current problems associated with methane, H_2S, and abandoned oil wells were taken in to account during planning and design.

In spite of the problems cited in the press, MTA can be satisfied with much of its tunnel construction. However, there have been several problems on a few contracts that have raised cause for concern. Many of the problems to date were unique in themselves, or have a common cause, and are not always directly related to geotechnical aspects of tunneling.

The several claims and change orders on MTA projects indicate several undesirable conflicts between the owner and the contractor. Some of these resulted in (or from) construction tolerances exceeding the specifications. There are many cases throughout the industry where tunnel specifications have been relaxed or were safely exceeded every once in a while. It is an informal form of partnering but this usually is permitted only if it is clear that no extra costs or damage can possibly result from the relaxation. Such did not appear to be the case in Los Angeles. Many of the problems may not have resulted if either the specified tolerance or the intent of the specifications had been met. For instance, the thin linings, the remining of various sections and, thus, the sinkhole were the ultimate result of being too far out of alignment. High technology digital guidance control systems should be specified on future projects.

To obtain fair bids, the specified tolerances must be achievable under the conditions given for the contractor to bid and it must be clear that the contractor will be required by the owner to achieve the tolerances. These more stringent controls should be required in the future, especially for settlement and alignment. The Geotechnical Design Summary Report (GDSR) plays a major role in setting the tone and criteria in a manageable way.

These problems are industry problems as well as an MTA problem and the industry is actively trying to improve the ways tunnels are specified and managed during construction, particularly with respect to GDSRs. The MTA project could be and should be a leader in these developments.

Comparison of L.A. Tunneling Problems to Worldwide Experience

Among the tunneling difficulties which occurred at the Los Angeles Metro, there are four single incidents which stand out in terms of their significance and impact. They are the reduced liner thickness on Segment 1, the excessive settlement on Hollywood Boulevard, the sinkhole on Hollywood Boulevard, and the excessive settlement in North Hollywood. In order to put these four incidents within a broader context, a summary of worldwide urban tunneling has been prepared for the same collection of case histories already introduced. A classification of performance has been prepared for these case histories based on settlement data and the occurrence of major ground failures. Classified as Category 1 were tunnels with settlement up to 1 inch, as Category 2 tunnels with settlement up to 2 inches, and as Category 3 tunnels with settlement over 2 inches or with major ground failure or structural incidents. Applying this classification, 44 percent of the tunneling projects worldwide could be classified as Category 1, 14 percent as Category 2, and 42 percent as Category 3.

Applying the same classifications to the Los Angeles Metro Rail construction history, which includes eight separate tunneling contracts so far, one obtains 50 percent of cases in Category 1, 12.5 percent in Category 2, and 37.5 percent in Category 3. A conclusion then can be drawn that the Los Angeles Metro Rail tunneling is in its performance about equal to or slightly better than the world average.

Feasibility of Tunneling in Los Angeles

In relation to construction feasibility, the geological and geotechnical environment along the existing and proposed corridors of Los Angeles Metro is clearly compatible with safe and economical underground construction. Dozens of cities in various countries have successfully developed underground transportation systems in similar or even more difficult ground conditions. About half of these cities have experienced difficulties comparable to or even worse than those which occurred in Los Angeles. Comparing this with the Los Angeles construction history, which includes eight separate tunneling contracts and four major problems on these contracts, a conclusion can again be drawn that the Los Angeles subway tunneling is about equal to or slightly better than the world average. However, the other half of the case histories shows that shallow urban tunneling can be carried out entirely without accidents or undue interference with normal urban life and still at reasonable cost. It is this latter half of the subway tunneling spectrum that should become the target for Los Angeles now.

The Los Angeles Metro tunneling contracts carried out so far utilized tunnel boring machines with either open face or face partially protected with breasting plates. In ground conditions where the open face or the partial protection was not sufficient to control the ground, this tunneling method was complemented by ground improvement, mostly in the form of grouting. This approach was adequate for the majority of the tunneling, except for a few instances where excessive settlement occurred for reasons discussed before.

As reflected by the documented trends in soft ground tunneling worldwide, the risks involved in methods depending on ground conditioning are increasingly eliminated by turning to tunneling methods using positive face control. These methods offer a "blanket" type protection against ground deformation and instability and are much less dependent on factors such as ground variation or workmanship which play an important role with grouting.

In Los Angeles, where there are relatively few problems with groundwater, the optimal tunnel boring machine to be considered might be an earth pressure balance machine (EPBM) capable of operating in two modes. The first mode would be an open face mode, to be applied in competent ground (e.g., the Puente Formation). The second mode, to be used in the alluvial soils, would be the earth pressure balance mode, with the face under active pressure. Should groundwater become a serious problem, the earth pressure balance machine in closed mode is well-equipped to handle such a situation. In addition to considering an open-face shield, MTA should at least consider the advantages and disadvantages and cost implications of EPBM as one means of reducing risk of significant settlement and minimizing public disruption on future projects. Such tunnel methods may or may not have a higher cost.

Conclusions and Recommendations

The geological and geotetechnical environment along the existing and proposed corridors of Los Angeles Metro is compatible with safe and economical underground construction. The geology in Los Angeles is either equal to or more favorable to tunneling than in most other cities because the ground is generally drier and more competent with a minimum of obstructions. Conditions which are less favorable include the persistent methane problem, the earthquake fault displacement potential and the unique H_2S hazard.

Proper planning, design, and construction methods can be expected to overcome all of the less favorable conditions. A technical solution could be found to satisfy the H_2S problem, which should include a full scale test facility to confirm the technical feasibility and public relations acceptance of the chosen solution.

It has been concluded from a review of the seismological environment in the Los Angeles area, together with the seismic design and analysis methods being used by MTA and case histories of worldwide tunnel performance during past earthquakes, that earthquakes are not a governing factor in assessing tunneling feasibility in Los Angeles. However, in developing seismic design criteria for proposed new tunnels, it is recommended that results of recent research related to the seismic hazard in the Los Angeles region be reviewed.

It can be argued that since transit facilities in bored tunnels have demonstrated they can operationally survive earthquakes better than surface structures, serious consideration should be given during rail transit planning to the use of tunnels over surface facilities where possible. Tunnels have the potential of functioning as important lifelines during and after a major earthquake. In the event that a major earthquake disrupts the freeway system, the Metro tunnel system could provide the means for continued mass transit.

A survey of worldwide practice of shallow urban tunnels revealed that a vast majority of these tunnels are excavated using tunnel boring machines, most of them with positive face control (slurry and earth pressure balance machines). A smaller portion of urban tunnels are constructed with methods using open face, but in most of these cases the ground is conditioned before excavation. The U.S. practice in general still prefers this approach and this is why the Los Angeles Metro tunnels are in the latter category.

A worldwide survey of performance of urban tunneling has shown that in about 44 percent of the tunnels surveyed, the performance in terms of ground control was classified as Category 1 (without problems). In about 14 percent of the contracts, performance was classified as Category 2 (minor problems) and about 42 percent were classified as Category 3 (significant problems).

Applying the criteria cited above for the Los Angeles Metro, the performance levels achieved so far have placed about 50 percent of the tunneling contracts into Category 1, 12.5 percent into Category 2, and 37.5 percent into Category 3. This ratio is approximately equal to or slightly better than the worldwide performance.

The cost of soft ground tunnels in Los Angeles is low relative to prices worldwide. These low costs, however, may be a reason for some of the construction problems that have been experienced.

Many of the major problems may not have occurred if the specified tolerances or at least the intent of the specifications had been met. For instance, the thin linings, as well as the remining of the various sections and, thus, the sinkhole, were the ultimate result of being too far out of alignment. Accordingly, more stringent controls should be required in the future, especially in settlement and alignment. High technology digital guidance systems should be required on future tunnels.

All tunnel projects involve considerable risk, but risk has been successfully managed on many projects. A balance should be established between risk and the cost of risk mitigating measures. MTA should ultimately make the decision as to what this balance should be as well as how much risk MTA wishes to share with the contractor.

It is recommended that for future tunneling, consideration be given to application of earth pressure balance tunnel boring machines capable of operating in two modes -- as an open face machine in competent ground (e.g., the Puente Formation) and with positive face control in less competent ground (e.g., alluvial soils). The choice of whether to permit an open face shield in preconditioned ground or require an earth pressure balance machine will depend on the degree of risk MTA wishes to share and on the overall costs.

It is recommended that ground control be established as the governing design and construction criterion on the existing and future tunneling contracts, with firmly set rules about monitoring and interpreting ground deformation data as well as practical steps to be taken immediately where deformations exceed the permissible limits. MTA should consider the cost and benefits of specifying less risky construction methods and methods which minimize construction impact to the public, even if their initial cost may be somewhat more expensive.

It is recommended that fundamental technical principles and policies for the project are formulated and regularly overseen at the owner's level by a small permanent staff highly experienced in tunneling, assisted by a small independent technical review board reporting directly to MTA.

The project so far has involved highly experienced companies with competent professionals. However, the project seems to have unnecessary difficulties in communication and coordination between companies. All projects of this nature have similar problems, but these intergroup coordinations and communications, together with risk-related decision-making, would be improved if MTA would develop a small, highly experienced technical staff that would coordinate these communications and ultimately make the decision whenever differences of opinion develop.

The MTA project could be and should be a leader in the current industry quest to improve the ways tunnels are specified (especially with respect to Geotechnical Design Summary Reports) and managed during construction.

APPENDIX

APPENDIX TO
REPORT ON FEASIBILITY OF TUNNELING
FOR THE LOS ANGELES COUNTY METRO RAIL PROJECT

November 1995

1.0 SCOPE, METHODOLOGY, AND BACKGROUND

This project was commissioned by the Los Angeles County Metropolitan Transportation Authority (MTA) on August 8, 1995, with the objectives of:

1. Evaluating and comparing problems encountered in tunneling projects worldwide, with those problems encountered in constructing MTA tunnels to date.

2. Assessing whether the geological and geotechnical conditions of the Los Angeles region are inconsistent with tunneling.

In fulfilling these objectives, the panel:

1. Reviewed and summarized the geological, geotechnical, and seismic environment along existing and proposed tunnel corridors to determine potential related hazards with respect to tunnel construction and design.

2. Reviewed and summarized worldwide case histories related to seismic performance and tunnels in similar environments to those encountered in Los Angeles, and reviewed current MTA seismic design philosophy.

3. Reviewed and summarized the status of available tunneling methods and technologies as used worldwide today, including an evaluation of their advantages and limitations.

4. Reviewed and summarized 74 worldwide case histories of urban shallow tunneling in geological and geotechnical conditions similar to those encountered

in the Los Angeles region to provide a base on which Los Angeles experience can be evaluated.

5. Reviewed and evaluated existing concepts available for assessing risk in tunnel projects to enable broad guidelines for risk parameters and management systems appropriate for MTA tunnels to be developed.

6. Evaluated costs for existing MTA tunnels and for other projects in North America where cost data were available, together with costs of risk mitigative measures.

7. Reviewed local and MTA tunnel construction experience to date.

8. Compared and assessed construction difficulties encountered to date with MTA tunnels in relation to worldwide case histories.

9. Determined, given the data collected, reviewed and evaluated, whether tunnel construction is a feasible and viable option for future tunneling corridors proposed by the MTA.

10. Recommended quality control measures for future planning, design, construction, and management of tunnel projects.

With respect to existing tunnels and planned future tunnel projects, Figure 1.1 summarizes current MTA tunnel alignments and terminology. Segment One is presently operational and Segment Two is under construction. Construction has also commenced on the Segment Three alignment through the Santa Monica Mountains into North Hollywood. Tunneling in North Hollywood is underway and a contract has been awarded to construct the tunnel through the Santa Monica Mountains.

The Segment Three 6.6 mile Eastside Extension to Whittier Blvd. is at a planning and design stage. The Segment Three Mid-City extension to Pico Blvd. is also still in preliminary design and environmental review stages. Further route alternatives extending west of the Pico/San Vicente station to the intersection of Wilshire Blvd. and the 405 freeway are undergoing preliminary study.

Conceptual studies for the final alignment and configuration of the proposed San Fernando Valley East-West Rapid Transit Project are also ongoing. This project presently involves both aboveground and underground sections, and will commence at the North Hollywood station and extend 13.9 miles west through the Valley.

The subsequent sections of this appendix document the material reviewed and summarized in fulfilling the above objectives, leading to final conclusions and recommendations given in the basic report.

2.0 GEOLOGICAL AND GEOTECHNICAL CONSIDERATIONS AND EARTHQUAKE ISSUES

This section of the report summarizes the geological/geotechnical and seismic environments along both existing and proposed tunnel alignments. Worldwide case histories related to the historic performance of tunnels in past earthquakes are also reviewed and summarized along with comments on the potential seismic vulnerability of existing and proposed tunnels in the Los Angeles area. This database establishes the groundwork for technical comments on tunnel design, construction and related risks, documented in subsequent sections of the report.

2.1 Regional Geologic and Geotechnical Framework

2.1.1 Geologic Formations

The existing and proposed MTA tunnel alignments traverse portions of three major physiographic features as shown in Figure 2.1, namely the Los Angeles Basin, the Santa Monica Mountains, and the San Fernando Valley. As described by Converse et al. (1981), the Los Angeles Basin, once a marine embayment, accumulated sediments eroded from surrounding highlands during the Miocene and Pleistocene epochs beginning about 25 million and one million years ago, respectively. Uplift of the Santa Monica Mountains provided much of the sediment filling the basin. Volcanic activity also produced extensive accumulations of basalt in the Santa Monica Mountains during the Miocene epoch. The Los Angeles Basin and the San Fernando Valley were uplifted during the Pleistocene epoch. Rapid uplift and erosion was in early Pleistocene time, filling the Los Angeles Basin with about 1,300 feet of sandy sediments (San Pedro Formation). Holocene time (beginning with the last melting of the Ice Sheets 11,000 years ago) resulted in alluvium (coarse gravels and sands) being deposited in stream channels extending into the Los Angeles Basin. The San Fernando Valley has been filled with considerably thicker deposits of alluvial sediments than the northern part of the Los Angeles basin.

Geologic units encountered by existing or proposed tunnel alignments in order of increasing age, are shown in Table 2.1. With reference to this table, the geologic materials ranging from Alluvium through the Puente Formation can be regarded as being associated with soft ground or soft rock tunneling methods. The harder rock formations associated with the Topanga Formations through the granitic rocks encountered in the Santa Monica Mountains, require hard rock tunneling techniques. The surface variations of the various geologic units are shown in the Geologic Map of Figure 2.2.

2.1.2 Groundwater Basins

Many of the serious difficulties encountered during construction of a tunnel, are related to the presence of water. The characterization of the groundwater environment is hence a critical factor in tunnel studies. Existing and proposed MTA tunnel alignments traverse four hydrologic units, as described by Converse et al., (1981):

1. Los Angeles Forebay area
2. Hollywood Basin
3. Santa Monica Mountains
4. San Fernando Valley Basin

The Los Angeles Forebay Area extends southward from the narrows of the Los Angeles River, and is an area of unrestricted infiltration of surface water. Groundwater in this area occurs in Young Alluvium and in older Pleistocene Alluvium, with some aquifers being semi-perched. The Hollywood basin is located near the Southern margin of the Santa Monica mountains. Both the Young Alluvium and the Older Alluvium sediments contain known aquifers. The Santa Monica Mountain Range is not a groundwater basin, and the older crystalline rocks have a limited capacity for transmission of water. However, the extensive joint and fracture systems can produce groundwater in considerable quantities for short periods of time. In the San Fernando Valley Basin, groundwater is present in Young and Old Alluvium. However, groundwater levels have declined since the 1940s due to heavy pumping, albeit wet winters can recharge levels by several tens of feet over a few years.

A generalized map (After Proctor, 1981) showing geologic and groundwater features in the Los Angeles Basin is shown in Figure 2.3.

2.1.3 Oil And Gas

The presence of oil or gas clearly requires special tunnel construction precautions and design measures. Oil was first discovered in the Los Angeles Basin in 1880, being produced primarily from thick deposits of lower Pliocene and Upper Miocene rock strata. The location of oil fields is shown in Figures 2.2 and 2.3. The La Brea Tar Pits are associated with the Salt Lake Oil Field. Existing and proposed tunnel alignments pass over or near several oil fields, and are discussed in detail by Converse et al. (1981). Appreciable quantities of gas (particularly methane and H_2S) have been detected in borings and from gas probes along sections of segment routes, as described in detail for example, by Proctor and Monsees (1985) for Segment 1. To date, design measures to mitigate risks associated with methane have included route re-alignment, the use of high density polyethylene (HDPE) liners and gas monitoring systems, as described by Navin (1991).

2.2 Segments 1 and 2: Geologic and Geotechnical Conditions

Whereas the focus of this report relates primarily to future tunneling plans, conditions encountered during planning and construction of the existing Segment One and Two tunnels are briefly summarized for comparative and reference purposes in later sections of the report.

2.2.1 Segment 1

The following summary is primarily drawn from a paper by Escandon et al. (1992). Segment 1 involved construction of 4.4 miles of subway in the downtown area of Los Angeles. The geologic conditions along the alignment are shown in Figure 2.4. Holocene Younger Alluvium associated with the Los Angeles River flood plain underlies most of the downtown area and was encountered in over 33 percent of all tunnel and station excavations. The younger alluvial deposits consist of dense granular sands and gravels with cobbles and boulders near the Los Angeles River and numerous stiff fine-grained silt and clay interbeds. Pleistocene Older Alluvium, consisting of dense fine to medium sands and gravelly sands with some silt and clay interbeds, also blankets portions of the alignment but was encountered in less than 2 percent of all excavations.

Siltstone, claystone, and sandstone bedrock of the Fernando and Puente Formations underlie the alluvial deposits in the downtown area. The bedrock was encountered in approximately 65 percent of the all tunnel and station excavations and is generally thick-to-thin-bedded and contains local hard cemented beds and concretionary nodules. Both bedrock units have the engineering properties of hard cohesive soils and are considered soft-ground tunneling materials due to their general excavation character and relatively low unconfined compressive strength. The transition from Fernando to Puenete Formation is recognized by a nearly 100-foot-thick zone of diatomaceous shale with interbedded siltstone at the stratigraphic top of the Puente Formation.

Groundwater encountered on the Project is associated with the Los Angeles Forebay Area and occurs primarily within the younger and older alluvial deposits. The alluvial deposits serve as a pervious free groundwater reservoir resting upon the underlying relatively impermeable Fernando and Puente Formations. Groundwater levels at the eastern portion of the project near the Los Angeles River were measured within 20 feet to 25 feet below the ground surface prior to construction. Site dewatering was performed in this area prior to tunnel and station excavation. Groundwater levels throughout the remainder of the segment were generally below the tunnel invert elevation except for perched groundwater at the base of the older alluvium on Contracts A171 and A175 and local seeps within the bedrock along joints and interbeds.

Due to construction in "gassy" or potentially "gassy" ground, tunnels are completely encased in a high-density polyethylene membrane, as described by Navin (1991). Chemical grouting was used on Contract A 146, where excessive ground losses through the tunnel faces were observed to be occurring in cohesionless sand deposits at or above the tunnel crown. The chemical grouting program is described in detail by Robison and Wardell (1991).

2.2.2 Segment 2

Segment 2 of the Metro Red Line was divided into two construction phases, namely the Wilshire Corridor and the Vermont/Hollywood Corridor. The Wilshire Corridor is discussed in detail by Stirbys et al. (1992). A brief summary of subsurface conditions encountered in the Vermont/Hollywood Corridor extracted from a report by the Earth Technology Corporation (1990), is given below.

The Phase II alignment can be subdivided into two alignment portions, the Vermont Avenue alignment and the Hollywood Boulevard alignment. The Vermont Avenue alignment portion extends from the Wilshire/Vermont Station, and extends north along Vermont Avenue to the Vermont Avenue/Hollywood Boulevard intersection (near Barnsdall Park). The Hollywood Boulevard alignment extends from the end of the Vermont Avenue alignment, west along Hollywood Boulevard to the Hollywood/Highland Station. Generalized cross sections are shown in Figures 2.5 and 2.6.

In general, the subsurface stratigraphy along the Vermont Avenue alignment consists of shallow fill zones and Old Alluvium overlying the Puente Formation bedrock. The depth of Old Alluvium ranges from approximately 1 foot to 40 feet, except in the vicinity of the Vermont Avenue/La Mirada Street intersection where an alluvial valley with a depth of about 80 feet or more is encountered. The Old Alluvium generally consists of interspersed layers of dense to very dense granular soils (gravelly, silty, and clayey sand, sandy silt, and thin zones of gravel) and stiff to very hard fine-grained soil (silty and sandy clay, and clayey silt.

Along the Vermont Avenue alignment, a major portion of the station and tunnel excavation is located within the Puente Formation. This formation consists predominantly of stratified silty claystone, clayey siltstone, and sandy siltstone with thin, weakly cemented sandstone interbeds. In general, the materials, exhibit more clay content, fewer sandstone interbeds, thicker highly weathered and oxidized zones, and become weaker and less permeable as the alignment moves north.

The subsurface conditions near the east end of the Hollywood Boulevard alignment are similar to those encountered in the Vermont Avenue alignment portion. Because of a gentle down-dipping of the Puente Formation bedrock to the west, the stratigraphy within the station and tunnel depths and in the area west of the Hollywood Boulevard/Kenmore Avenue intersection consists of shallow fill zones and Young Alluvium overlying Old Alluvium. The fill and Old Alluvium materials are similar to those encountered in the Vermont Avenue alignment. The Young Alluvium consists of interspersed layers of medium-dense to dense sand, silty and clayey sand, with occasional zones or lenses of gravelly sandy and sandy gravel, and stiff to very stiff clayey silt and silty clay.

Groundwater levels along the Phase II alignment generally reflect the ground surface topography. Groundwater levels in the Vermont Avenue alignment were reported to be approximately 4 feet to 35 feet below the ground surface and are above the planned tunnel inverts. The groundwater levels within the Hollywood Boulevard alignment are generally located within the Old Alluvium and are generally at or below the planned station and tunnel inverts, except in the vicinity of the east end of the alignment, where groundwater levels are either within the Puente Formation or above the tunnel invert.

The known boundaries of the Los Angeles City Oil Field traverses the alignment (Vermont Avenue between Second and Fourth Streets). The Earth Technology report comments that available data indicate that the Vermont Avenue alignment and the alignment portion in the vicinity of Barnsdall Park can be classified as "gassy" and elsewhere along the Hollywood Boulevard as "potentially gassy."

2.3 Segment 3 - Geologic and Geotechnical Conditions

Segment 3 comprises three legs, the North Hollywood, Mid-City, and Eastern Extension Segments. The North Hollywood leg is described in two parts, namely (1) the hard rock portion from the Hollywood/Highland station through the Santa Monica Mountains to the Universal City station, and (2) the San Fernando Valley leg from Universal City to North Hollywood (see Figure 2.7).

2.3.1 Segment 3: Santa Monica Mountains

The geologic conditions likely to be encountered during tunnel construction through the Santa Monica Mountains are described in detail in a Geotechnical Investigation Report prepared by The Earth Technology Corporation (1993) and in the Geotechnical Design Summary Report (Parsons Brinckerhoff Quade and Douglas, 1994). This brief summary is based on material presented in these reports.

As-built construction records for two existing tunnels in the near vicinity to the proposed MTA tunnel (namely, the Los Angeles Sewer Tunnel and the Metropolitan Water District (MWD) Hollywood Tunnel) assist considerably in the assessment of geologic conditions, and provide valuable insights to construction conditions likely to be encountered, including groundwater inflows.

The tunnels will be driven through eight different bedrock units encompassing three different rock types namely plutonic (granitics), volcanics (basalt), and various sedimentary rocks (conglomerates to siltstone/shale). The stratigraphic sequence of rock units and their relative thickness are shown in Figure 2.8. Contacts between rock strata dip to the north/northeast at angles of about 40 to 60°. Boundaries between units include conformable, nonconformable, and fault contacts.

In general, the subsurface conditions along the tunnel alignment can be divided into six reaches based on similar geologic units, rock types, and anticipated ground behavior. These reaches (as described in the Earth Technology report) are defined from south to north (oldest to youngest) in Table 2.2. For tunneling consideration, geologic conditions and features of each of the reaches were compiled and tabulated by Earth Technology as shown in Tables 2.3 through 2.5. The geologic formations in the Santa Monica Mountains are not expected to contain oil or gas. Monitoring programs during drilling investigations, produced no traces of organic vapors.

With reference to Tables 2.3 through 2.5, the following specific fault zone features are noted:

Reach 1: The Hollywood Fault zone is encountered at the south end of the reach. Nearly decomposed and intensely sheared rock is anticipated for a few hundred feet north of the zone. The fault forms a groundwater barrier generating at least 186 feet of groundwater elevation difference across the fault.

Reach 2: Contains about a 15 feet wide highly sheared fault zone. The groundwater table is about 700 feet above the tunnel crown.

Reach 6: The Benedict Canyon Fault zone is crossed at the north end of the reach. The width of the sheared and gouge zone is unknown. Bedrock overburden rapidly reduces in the area. The possibility of encountering young alluvium and groundwater beneath the Hollywood freeway is noted.

The presence of large boulders within the Chico, Simi, and Topanga Formation conglomerates is also noted, leading to possible problems with TBM cutters.

Measured groundwater levels in 1991 to 1993 (as reported by Earth Technology, 1993) indicated elevations ranging from a minimum of 53 feet and 128 feet above the tunnel crown on the north and south flanks of the mountains to a maximum of 758 feet near the mountain crest. It is also noted that the groundwater system responds rapidly to recharge from rainfall, and that near surface recharge is in hydraulic communication with groundwater at tunnel depth. Hydrostatic pressure measurements indicated vertical continuity throughout the rock mass, most likely through the frequent rock discontinuities.

2.3.2 Segment 3: North Hollywood Tunnel

The geological and geotechnical conditions likely to be encountered in the tunnel segment between Universal City and North Hollywood are described in the Geotechnical Summary Report (Converse Consultants, 1993). The brief summary below is based on this report.

Bedrock Section: 876 feet of the tunnel, leaving from the north end of the Universal City Station, will occur in the weak bedrock of the Topanga Formation and below the water table. Bedrock at tunnel grade is described as weathered and very low strength claystone, siltstone and some sandstone strata. Average thickness of bedrock above the crown is 10 feet. Permeable sandstone beds 1 inch to 1-foot-thick and dipping 20° to 40° are interspersed with the impermeable siltstone and claystone strata. Groundwater level is at 20 to 30 feet above the crown and is in recent channel deposits of the Los Angeles River.

Mixed-Face Bedrock/Alluvium Section: About 1,000 feet of the tunnel will encounter mixed face conditions. Bedrock is the Topanga Formation described above. The alluvium consists primarily of fine grained older channel deposits below the groundwater table, which averages 20 feet above the crown.

Alluvial Section: The final 8,632 feet of the tunnel (moving north) comprises heterogeneous and non-uniform younger and older alluvium. This reach is above the regional groundwater table. The upper 45 to 50 feet of alluvium consists primarily of sands, silty sands, and gravelly sands. Underlying alluvium consists primarily of gravelly sands and sandy gravels, some of which contain cobbles and boulders.

2.3.3 Segment 3: Eastside Extension

The geological and geotechnical conditions likely to be encountered during tunnel excavations for the proposed Eastside extension, have been presented in a preliminary Geotechnical Investigation Report by Geotransit Consultants (1994). We understand that further geotechnical investigations are continuing to provide additional subsurface data needed for final design. The brief summary given below is largely drawn from the above report.

The tunnel alignment (see Figure 2.9) is located along the southern flank of the Repetto Hills area of the Los Angeles Basin. Planned tunnel and station excavation will be within the Young and Old Alluvial deposits of the Holocene and Pleistocene ages and the Tertiary bedrock units of the Fernando and Puente Formations.

The Young Alluvium within the Western Portion of the alignment (Figure 2.9) is reported as heterogeneous, consisting of predominantly coarse-grained materials ranging from sand to gravels with local zones of cobbles and boulders (to 4 feet in size). Occasional layers of fine-grained soil (sandy clay and clayey silt) are also present. The alluvium in the eastern portion of the alignment is predominately Old Alluvium, with Young Alluvium occurring locally within intermittent drainage courses. The alluvium is reported as being very heterogeneous, comprising fine-grained materials (clays-silts) and sand and gravels with occasional cobbles and boulders. Bedrock units of the Fernando and Puente Formation underlying the alluvium, are found locally west of Boring PE-17 (see Figure 2.9). The bedrock will consist primarily of low strength siltstones and claystones with localized layers of hard calcareous interbeds to 4.5 feet thick and hard conretionary nodules 2 to 18 inches in size.

Groundwater levels are reported to be likely at or below planned tunnel inverts east of Boring PE-14 and within or above inverts west boring PE-17, except between borings PE-18 and PE-25. Additional investigations are planned to check the complex nature of hydrological conditions in the latter region. Available data suggests potential soil and groundwater contamination with hydrocarbons and H_2S between Union Station and the vicinity of borings PE-18 and PE-29. At this location, there is a "bedrock high" that may be acting as a geologic barrier to H_2S and groundwater contamination.

2.3.4 Segment 3: Mid-City Extension

The alignment for the proposed Mid-City extension to the Red Line (between the Wilshire/Western and the Pico/San Vicente Stations) is shown in Figure 2.10. The focus of ongoing studies related to this alignment are primarily related to the presence of H_2S. Background to ongoing studies regarding construction options and H_2S hazard evaluations is described in a report by Engineering Management Consultant (July 1994) and in a publication by Ellioff et al. (1995). The comments below are largely drawn from the above publications.

An earlier alignment along Wilshire Boulevard was abandoned following a methane explosion in a poorly ventilated store near the alignment in 1985, and resulted in the establishment of an exclusion or potential risk zone (see Figure 2.10). The present alignment was considered the best alternative following a gas monitoring program. However, environmental and geotechnical investigations conducted in 1993 found higher H_2S concentrations then previously measured. This led to an ongoing re-assessment study focusing on issues of and options for underground construction and operation in the presence of H_2S and also on determining the geographical extent of gases to the west of the Pico/San Vicente station.

The geologic profile for the Mid-City extension is shown in Figure 2.11. The profile comprises artificial fill, Younger and Older Alluvium Formations (sands, silts, clays and gravels; which include perched groundwater at various depths), the Lakewood Formation (silts and silty gravels), the San Pedro Formation (very dense fine to medium grained sands including a zone of perched groundwater), and the Fernando Formation (massive siltstones with sandstone interbeds). The Fernando Formation is below the depth of potential tunnel inverts. The San Pedro Formation is the major source of H_2S along the alignment. The less permeable Lakewood Formation provides a barrier to gas flow.

As H_2S is toxic at relatively low concentrations and has an odor threshold at extremely low concentrations, the re-assessment study has been evaluating three alternative elevation alignments to overcome the problem, as illustrated in Figure 2.12. As levels of odor are expected to reduce with distance above the San Pedro Formation, alternatives include a raised alignment, a bored/cut-and-cover tunnel with cut-and-cover stations, and a bored/cut and cover tunnel with raised stations. Additional ongoing studies include further field and laboratory testing to obtain more information on the presence and characteristics of gasses, gas reservoir modeling, handling and disposal of excavated soil, and hazard mitigation evaluations.

While the primary focus of attention has been on the Mid-City extension, other complementary studies have also addressed potential gas concerns for further alternative route extensions west of the Pico/San Vicente station to the intersection of Wilshire Boulevard and the 405 freeway. Figure 2.13 shows the Mid-City extension and alternative routes for western extensions of the Red Line. (Ellioff et al., 1995).

Figure 2.14 shows a simplified geologic cross section for the northernmost study segment. Traveling westward, the stratigraphy changes significantly and reported gas incidents diminish. Results from gas sampling using the Cone Penetrometer indicate that H_2S is not present west of Fairfax Avenue.

2.4 Proposed San Fernando Extension: Geologic and Geotechnical Conditions

We understand that conceptual studies for the final alignment and configuration of the proposed San Fernando Valley East-West Rapid Transit Project are still ongoing. To support these studies, only limited geotechnical investigations have been performed to date. The comments given below have been summarized from a report documenting a pre-preliminary engineering study by Engineering Management Consultant (September 1994).

The proposed alignment is shown in Figure 2.15 and commences from the North Hollywood station, moving west 13.9 miles through the Valley to the Topanga station. A number of options are being considered, combining both aboveground and underground segments. One of the alternative options is shown, for example, in Figure 2.16.

The alignment is dominated by Young Alluvial deposits as shown in Figure 2.17. Groundwater levels are low over the first 10 miles starting at North Hollywood, and are below the zone of influence for tunnel construction. Over the remaining western segment, groundwater levels are higher than planned tunnel inverts in localized areas. It is reported that large boulders (up to 4 feet in size) could be encountered in the eastern tunnel segment, and some mixed face conditions could be encountered near the Topanga station (planned to be cut-and-cover). No natural underground hydrocarbons have been detected in test borings.

2.5 Seismic Environment

Los Angeles is located in an area of high seismic potential that has experienced strong ground shaking from numerous large earthquakes in historical times. To provide the framework for establishing seismic design criteria for the Segment 1,2, and 3 of the tunnel alignments, a comprehensive study of the geologic structures and active faults capable of generating major earthquakes in the Los Angeles region is reported by Converse Consultants (1983). The seismicity of these structures expressed as the expected recurrence of earthquakes exceeding specified magnitudes was also studied, leading to probabilistic estimates of ground shaking level along tunnel alignments. Surface faults having the most significant impact on the tunnel alignments in terms of ground shaking potential included the Hollywood, Santa Monica and Malibu Coast Faults, and the Raymond and Newport Inglewood Faults.

As a result of research over the past decade, further insight as to the nature of geologic structures impacting the earthquake potential of the Los Angeles region has been obtained (Dolan et al., 1995). In particular, the blind thrust fault systems (that do not cut the ground surface) underlying the San Fernando Basin, the Santa Monica Mountains and the Los Angeles Basin (Elysian Park thrust fault) are also now recognized as being a major earthquake threat to the Los Angeles metropolitan area. A blind thrust fault was responsible for the Northridge earthquake. Current thinking related to the seismic hazard in the Los Angeles region is documented in publications by the Working Group on California Earthquake Probabilities (1995), Peterson and Wesnousky (1994) and Dolan et. al. (1995). The California Division of Mines and Geology have recently published draft probabilistic seismic hazard maps for Southern California based on these more recent studies. It is recommended that these studies be reviewed to update the 1983 Converse study prior to developing seismic design criteria for proposed new tunnel alignments.

No active faults were identified along alignments for Segments 1 and 2. However, Segment 3 crosses the Hollywood Fault and future westward extensions may cross the Santa Monica Fault, both identified as active. The Eastside Extension fault investigations have not uncovered any clear expressions of near--surface strike-slip faulting along tunnel alignments (Geotransit Consultants, 1994). However, studies to date have indicated that the asymmetrical fold comparing the Coyote Pass Escarpment, which intersects the alignment, is a potentially active tectonic structure. The escarpment could be uplifted during a future earthquake on an underlying blind thrust structure. Studies are continuing on the nature and magnitude of such an uplift, which could impact the design of proposed tunnels when crossing the escarpment.

2.6 Performance of Tunnels in Past Earthquakes

The potentially damaging effects of earthquakes on underground tunnel structures can attributed to either effects arising from ground shaking or those arising from ground failure. The latter effects include failures arising from permanent ground displacement due to slope instability, liquefaction-induced ground movement, fault displacement, and tectonic uplift or subsidence.

A literature survey was undertaken to compile available data on the performance of tunnels in past earthquakes. Use was made of the National Center for Earthquake Engineering (NCEER) computer-based search system and personal contacts. Often-cited publications documenting earthquake performance include papers by Dowding and Rozen

(1978), Owen and Scholl (1981), and Sharma and Judd (1991). Performance data from the above publications has been summarized by Wang (1993).

The report by Sharma and Judd documents observations from 192 tunnels and from 78 earthquakes covering a variety of ground conditions (rock through alluvium), support types (unlined through reinforced concrete), and cover depths. Cited damage categories range from slight to moderate and heavy, but are not clearly defined in the paper. It is assumed that heavy damage implies collapse or partial collapse of a portion of the tunnel, whereas slight to moderate damage implies spalling of rocks or concrete lining, or observed deformation of supports or lining.

Of the 192 tunnels, 98 were reported as undamaged. Of the remaining 94 tunnels reported as damaged, 22 are reported as suffering heavy damage. Of the 9 reinforced concrete tunnels damaged, 3 are reported as being heavily damaged. Unfortunately, the Sharma and Judd study does not differentiate damage arising from faulting and ground shaking, nor do they indicate whether underground tunnels are bored or cut-and-cover structures. Cut-and-cover tunnels are potentially more vulnerable to damage, while it is clearly difficult to prevent damage to tunnels crossing faults. As expected, the survey does shown that increased damage is correlated with increased ground shaking intensity, and that tunnels in alluvium are more susceptible to damage than those in rocks.

NCEER has recently undertaken a major research study on tunnel performance during earthquakes for the Federal Highway Administration (FHWA), as part of a major research contract on the seismic design of highway structures. This study, Power (1995), which is presently only partially complete, is re-examining the Sharma and Judd database to clarify fault displacement versus ground shaking performance and is improving documentation on likely ground acceleration levels. The study is also documenting tunnel performance during more recent earthquakes, including the 1989 Loma Prieta, 1994 Northridge, and 1995 Kobe earthquakes.

The draft NCEER study documents severe damage to four Southern Pacific Railroad tunnels (unreinforced concrete lining) during the 1952 Kern County earthquake, where accelerations were estimated to be on the order of 0.8 g. However, this damage is considered as fault-rupture related. During the 1972 San Fernando earthquake (M 6.6) only slight or no damage was reported to several unlined rock tunnels and reinforced concrete-lined tunnels where estimated peak ground accelerations ranged from 0.57 to 0.76 g. The performance of tunnels in more recent earthquakes, as documented in the draft NCEER study, is summarized below.

2.6.1 Loma Prieta Earthquake

The NCEER draft study reports no recorded earthquake-induced structural damage to tunnels (including the BART tunnels) during the 1989 M 7.1 Loma Prieta earthquake. Estimated ground accelerations at most tunnel locations were less than 0.2 g. However, one timber-lined tunnel located 11 miles from the fault was subjected to accelerations of about 0.4 g.

2.6.2 Northridge Earthquake

The Metropolitan Transit Authority (MTA) had two trains in operation at the time of the earthquake in a pre-operation inspection mode (Schiff, 1994). Subway tunnels for the Redline have sensors installed to measure ground accelerations. At 0.1 g, a warning is issued to trains and at 0.2 g, all trains are stopped. An acceleration of 0.27 g was recorded during the Northridge event. No structural damage was observed in tunnels and there was also no damage to the Vermont/Hollywood Boulevard tunnels under construction at the time of the earthquake.

The NCEER draft study documents the performance of 53 tunnels during the 1994 6.7 Northridge earthquake, including Metropolitan Water District (MWD), Southern Pacific Railroad (SPRR), and Los Angeles Department of Water and Power (LADWP) tunnels. No significant structural damage to tunnels due to earthquake shaking has been identified in the study to date. Examples of tunnels which were severely shaken include the MWD Balboa inlet tunnels (concrete, reinforced concrete, and steel-lined sections; estimated ground acceleration 0.57 g), the LADWP LA Aqueduct #1 (concrete lining; estimated ground acceleration 0.57 g), and the SPRR Chatsworth tunnels (estimated ground acceleration 0.43 g).

2.6.3 Kobe Earthquake

The 1995 M 6.9 Kobe earthquake severely damaged the rail transportation network in Kobe, primarily the elevated rail structures and the stations. However, to date, there have been no reports of structural damage to concrete-lined, bored tunnels, including two long tunnels under Rokko mountain which were constructed around 1964 for the Bullet Train. The NCEER draft study is still compiling more specific data on the performance of bored tunnels. Power (1995) reports that an NTT phone company 4-meter diameter shield tunnel in soil performed well, and that a letter from the President of Japan's Tunneling Association indicates only "globally slight damage" to bored tunnels.

In contrast to the lack of damage reports to bored tunnels, severe damage has been reported (O'Rourke [1995], EQE [1995]) to underground sections and stations of the Kobe Rapid Transit Railway, which were constructed using cut-and-cover methods. Stations and running tunnels are reinforced-concrete, box structures with center columns separating inbound and outbound trains. The depth of the cover is about 15 to 25 feet.

The most serious damage was the collapse of the Daikai Station and structural damage to reinforced-concrete supporting columns. Ground acceleration levels have been estimated at about 0.6 g. Representative cross-sections showing the collapsed station and a tunnel section are shown in Figure 2.18. The central reinforced concrete columns at the station failed, apparently due to a combination of vertical and shear loading, causing subsidence at the street surface. Only nominal confining steel was present in the columns (O'Rourke, 1995).

The cut-and-cover construction for the Daikai Station involved a braced excavation supported by sheet piles. The sheet piles were left in place when the walls of the station were cast, leaving about a 10 inch gap between the walls and sheet piles. The narrow gap did not allow for compaction of the sandy fill which was loose-dumped in the gap.

Reconnaissance observation of damage to the running tunnels, reported by O'Rourke (1995), is consistent with the illustration (to an exaggerated scale) shown in Figure 2.18. Shear distortion of the box structure is the result of transient earthquake-induced shear deformation of the soil. This type of deformation leads to the development of plastic hinges at column and wall connections with the roof and base. The shear distortion and related hinge formation, coupled with increased loads from vertical accelerations and the lack of lateral passive pressure support from the loose back-fill (which may have liquefied), appear to have caused the column collapses at the Daikai Station. A report by EQE (1995) suggests that the station may have behaved almost as a free-standing structure due to the possible lack of lateral passive pressure support.

In summary, based on the above survey, bored tunnels (either unlined rock tunnels or tunnels in alluvial ground constructed with reinforced-concrete linings) have performed exceptionally well in past earthquakes, including very strong earthquakes with estimated peak ground accelerations up to the order of 0.8 g. No case has been identified where structural damage has occurred from ground shaking effects. The only cases where major damage has occurred can be related to tunnels crossing faults where the earthquake has generated relative displacement across the fault. In the case of tunnels (or stations) constructed using cut-and-cover methods, severe damage has been observed, such as that described above in the Kobe earthquake. However, the reasons for this damage can be related to the seismic design methods used, as discussed in paragraph 2.7 below.

2.7 Seismic Design Philosophy

It is clear from the historical record documenting the performance of tunnels in past earthquakes, that tunnels are less vulnerable to earthquakes than aboveground structures. Whereas aboveground structures may amplify ground accelerations due to dynamic response and are subject to earthquake-induced lateral inertial loads, tunnel structures undergo displacement imposed on them by the oscillations of the surrounding ground. Hence the effects of ground shaking on tunnel performance is best evaluated by considering the distortions imposed on the tunnel lining by ground motions. In addition, as for aboveground structures, the earthquake performance of tunnels must consider the potential for damage arising from active faults intersecting the tunnel alignment and from large ground deformations induced by liquefaction of saturated alluvial sediments or by slope instability.

In order to establish earthquake design criteria for tunnels which take into account the above effects, it is necessary to first define the seismic environment surrounding the tunnel alignment and, secondly, define acceptable risk levels for design. The latter risk levels in turn define earthquake ground motion parameters for design. The seismic environment for existing and proposed MTA tunnels is described in Section 2.5. The design risk levels and ground motion parameters adopted to date for the MTA tunnel alignments are described in reports by Converse Consultants (1983), Metro Rail Transit Consultants (1964) and are summarized by Monsees and Merritt (1991).

Two levels of earthquakes are being considered for the design:

(1) An Operating Design Earthquake (ODE), where damage would be minimal and the overall system will continue to operate normally. The ODE is defined as an earthquake event which has a return period of several hundred years and can reasonably be expected to occur during a 100 year facility design line. A peak ground acceleration of 0.3g has been assigned to this event (note that accelerations of this order were recorded in the Segment One tunnel during the Northridge Earthquake, with essentially no damage).

(2) A Maximum Design Earthquake (MDE), where critical items continue to function to maintain public safety and to prevent catastrophic failure and loss of life. The MDE is defined as an earthquake event which has a return period of several thousand years. Such an event has a small probability of exceedance in the facility life, and was assigned a peak ground acceleration of 0.6g.

2.7.1 Ground Shaking Hazard

In checking the seismic performance of tunnel linings for the above ground shaking levels, three tunnel deformation mechanisms induced by the action of seismic waves are considered in the design procedures adopted for the MTA project (Metro Rail Transit Consultants [1984]) as shown schematically in Figure 2.19.

(1) Axial deformations
(2) Curvature deformation
(3) Ovaling or Racking deformation

Racking deformations of box sections used for cut-and-cover tunnel stations are also considered.

In analyzing the earthquake-induced deformations, the design philosophy is based on the provision of sufficient structural ductility in the tunnel lining without losing the capacity to carry the static loading from the overburden. That it, the deformations can occur without the lining losing its structural integrity. This approach to seismic design of tunnels is presently considered the state of the art and is described in more detail by Wang (1991).

With respect to the damage to cut-and-cover box section tunnels and the Daikai Station described in Section 2.6, O'Rourke (1995) notes that seismic design procedures considered only dynamic earth pressures (coupled with a seismic coefficient of 0.1-0.15) and not racking deformations under maximum ground acceleration levels, as described above. Hence the structures which were extensively damaged, had insufficient ductility to accommodate the earthquake imposed ground distortions.

We note that the report on "Investigation of Structural Integrity of Los Angeles Metro Red Line Tunnels, Segment 1" by Cording, et. al. (1994) documents an extensive analytical study of tunnel distortions imposed by the ODE and MDE design earthquakes. This report was commissioned by the MTA to evaluate the structural integrity of tunnel linings with thicknesses less than 12 inches. The report notes that the 12 inch dimension was established by constructability considerations, and is well in excess of that required to carry static ground levels. Seismic resistance analyses are hence carried out as a performance check rather than to establish a design lining thickness.

Cording et. al. (1995) reports on seismic analyses for both unreinforced and reinforced liners with thicknesses ranging from 6 to 18 inches. Earthquake-induced longitudinal strains were shown to be at the same level as concrete shrinkage strains that have already produced transverse cracks in the lining. During ground motions, cracks would open and close, possibly resulting in local fracturing but not structural damage. The report notes that "both thick and thin lining sections are able to accommodate large distortions, well beyond those imposed by the MDE without crushing" and that "the reserve distortion capacity of the concrete lining was at least twice that occurring at the MDE." The analyses hence reinforce the conclusions drawn in Section 2.6 in relation to the lack of observed structural damage to bored tunnel linings even in very strong earthquakes with peak ground accelerations of the order of 0.8g. These levels of acceleration could be expected for example, from a Northridge-like M6.7 earthquake beneath the city of Los Angeles.

2.7.2 Fault Displacement

No active faults were identified along the alignments for Segments 1 and 2. However, Segment 3 crosses the Hollywood Fault and future eastward extensions may cross the Santa Monica Fault, both identified as active. Anticipated fault displacement during MDE events on these faults were estimated as 4.5 and 6.6 feet for magnitude 6.5 and 7.0 events on the Hollywood and Santa Monica Faults, respectively. These displacements are discussed further by Metro Rail Consultants (1984) and have been adopted for design considerations.

Design philosophy for fault crossings recognizes that it is difficult to prevent damage in a strong earthquake, given the magnitude of fault displacements. The general design philosophy now widely accepted for fault crossings is to "overbore" the tunnel, so that if the maximum design earthquake-induced displacement occurs, the tunnel is still of sufficient diameter to fulfill its function after repairs. The overbored section is taken through the fault zone with transition zones narrowing to the regular tunnel diameter. The overbored sections are backfilled with easily re-minable and crushable material such as "cellular" concrete. Such an approach was adopted for the North Outfall Replacement Sewer Project Tunnel when crossing the Newport Inglewood Fault, and is being adopted for the proposed Segment 3 Hollywood Fault crossing.

As discussed in Section 2.5, the Eastside Extension Fault Investigations have not uncovered any clear expressions of near-surface strike-slip faulting that would result in abrupt displacement across tunnels. However, the studies to date (Geotechnical Consultants, July 1994) have indicated that the Coyote Pass Escarpment is a potentially active tectonic structure. The proposed alignment crosses the escarpment in three locations. The escarpment takes the form of an asymmetrical fold, arising from underlying thrust faults. Uplift from a potential future earthquake could lead to tunnel flexure. The nature of the uplift and the magnitude of possible structural effects on the tunnel together with any needed remedial design measures are still under study.

2.7.3 Liquefaction

Ground deformations arising from potential earthquake-induced liquefaction of loose saturated alluvial sediments would clearly be of design concern. Investigation methods to identify liquefaction potential in relation to design levels of ground acceleration are well established. Fortunately, studies to date have indicated the absence of significant liquefaction concerns along existing and proposed alignments.

In summary, the earthquake design philosophy and analysis methods adopted for MTA tunnels, reflect state of the art thinking and systematically address potential hazards induced by major seismic events, including those with a low probability of occurrence in the design life.

3.0 AVAILABLE TUNNELING TECHNOLOGIES

Though in many ways tunneling remains an art, over the last three or four decades tunneling has developed from the stage of empirical craftsmanship to a stage closer to engineering science. This has been mainly due to the application of rock and soil mechanics principles to tunnel behavior and to the development of highly advanced tunnel boring machines (TBM).

On the theoretical side, the application of rock and soil mechanics principles to the behavior of ground around a tunnel has enabled engineers to explain the response of ground to tunnel excavation and thus to develop design approaches which are valid and applicable in general terms. On the practical side, the development of sophisticated TBMs, closely linked to advancements in mechanical, hydraulic, and computer engineering, opened up possibilities for economical and safe tunneling through adverse ground conditions previously considered unfeasible or certainly impractical or uneconomical.

The purpose of this section is to review the current state of various tunneling technologies as they exist worldwide today and to critically discuss their advantages and limitations. This will be done considering the main criteria applied in design and construction of a tunnel project, namely:

1. Compatibility of a tunneling technology with the ground.
2. The economics of a tunneling system.
3. Loads on and structural adequacy of the lining.
4. Ground control, ground displacements, and surface settlements.

Unlike with most other civil engineering structures (bridges, dams, high buildings) the design of a tunnel must inherently involve considerations about the tunneling method and technology to be applied and about its compatibility with the given ground. The economics of a tunneling system is a logical follow-up, which depends to a large degree on the right decision made about the system itself. The design criteria, whose prime purpose is to safeguard performance and safety of the structure from the owner's point of view, are also to a large degree dependent on the tunneling system selected.

In selecting the system one has to keep in mind that there are basically two approaches to soft ground tunneling.

1. The tunneling system is selected to suit the given ground conditions (e.g., by one of the tunnel boring machines which can control the face by applying a positive pressure directly to the face), or

2. The ground is conditioned (changed, altered, modified) to suit the given tunneling technology (e.g., by dewatering, grouting, freezing, etc.)

Looking at tunneling from a worldwide perspective, the trend in present day tunneling is clearly towards the first approach -- to use a system which can successfully deal with all expected variations in the ground without altering it. One of the reasons for this trend is that a permanent or even a temporary change in ground conditions (e.g., grouting, freezing, or dewatering) is often regarded as an environmental hazard or an ecologically harmful infringement. The other major reason for the recent rapid development of tunneling methods belonging to the first category listed above is their much faster rate of progress and correspondingly lower final cost, particularly with long tunnels. driven through variable ground conditions.

The trends outlined above are particularly strong in the case of subway tunnels. These tunnels are usually located at shallow depths, often in soft ground (formed by soils such as clay, sand or gravel or their mixtures and seldom by hard rock) and in an urban environment sensitive to disturbances caused by tunneling. Under these conditions, the control of ground, that is, the minimalization of settlement and other ground displacements, is a prime design and construction concern.

The present world scene in soft ground tunneling is almost entirely dominated by two technologies -- the New Austrian Tunneling Method (NATM) and Tunnel Boring Machines (TBM). Figure 3.1 schematically outlines this scene and the options it offers.

The principles of the NATM, originally developed for hard rock, were transferred to and modified for soil tunnels. The first known application was for subway tunnels in Frankfurt, Germany in 1969. In this method, rather than excavating and supporting the full face of the tunnel as done with a TBM, the excavation is carried out in sequence of smaller sections of the tunnel face. Each section is supported immediately after its excavation by shotcrete in combination with steel reinforcement and ribs. The main advantage of the NATM is its flexible response to changing ground -- if the tunnel face encounters a zone of more difficult ground, the number of sequences can be increased or additional ground control measures (such as grouting, forepoling, etc.) introduced without major interference with the tunneling operation. Also, the method is very convenient in combination with field monitoring for the ease with which any indicated new or changing support requirements can be immediately applied by changes in sequencing and tunnel support.

A major advantage of NATM is in its exceptional suitability for tunnels with unusual or noncircular profile and for all types of tunnels of short length. In the latter case, this is because the mobilization cost of the NATM is only a fraction of the cost of a TBM. However, as this method is considerably slower than TBM, for longer tunnels it becomes less economical.

Without ground conditioning, the NATM can be applied only in uniformly competent ground, i.e., in stiff to hard clays or soft rocks. In unstable ground the method must be combined with ground stabilizing measures.

The Tunnel Boring Machines for soft ground can be divided into five categories, as schematically shown on Figure 3.1. These include the Open Face TBM (OF TBM), the Closed Face TBM (CF TBM), the Earth Pressure Balance TBM (EPB TBM), the Slurry Pressure Balance TBM (SPB TBM) and the Air Pressure Balance TBM (APB TBM).

The OF TBM, in general, cannot provide any support at the face and as such can be used only in continuously competent ground where the excavated face can safely stay unsupported, or it has to be combined with extensive ground conditioning.

The CF TBMs are of two basic types. Some models are equipped with diaphragm shutter doors immediately behind the cutting wheel. These doors, which can be closed almost instantly, are supposed to prevent flooding of the machine in case of sudden encounters with waterbearing, unstable ground. But these doors can provide only very limited positive support for the face and only while the machine is stopped. Forward tunneling with the diaphragm doors closed is not possible. Other types of CF TBM try to support the face with breasting plates (orange-peel types, breasting tables, and other). However, none of these models are capable of supporting the face fully, continuously and efficiently. Thus, the CF TBMs offer only marginal improvement over the OF TBMs. CF TBMs cannot be regarded as part of the same category as the SPB and EBP TBMs. In less stable ground or below groundwater level the CF TBM has been often combined with ground conditioning or with compressed air (APB TBM).

Compressed air tunneling, which used to be a very popular method of tunneling since the last century, is becoming phased out from the current scene. This is because of the cost and technical requirements associated with use of air but mainly because of the health safety regulations that limit the time workers can work in the compressed air. The APB TBM is also extremely cumbersome in handling materials through the air locks.

The Slurry Pressure Balance TBM (SPB TBM) supports an unstable face hydraulically with bentonite slurry. The slurry is kept under controlled pressure in a chamber at the front of the TBM, sealed from the rest of the machine. The chamber, which also contains the cutting wheel, is hydraulically connected with a separation plant at the surface. The bentonite slurry is constantly circulated between the tunnel face and the plant at surface. Outgoing slurry transports the soil cuttings, which are then removed from the slurry at the separation plant. The stability and deformation at the face is maintained by adjusting the pressure in the chamber to counteract the soil and water pressures. Sometimes the pressure is maintained by both slurry and air pressure as illustrated in Figure 3.1. The bentonite slurry penetrates the soil ahead of the cutting wheel, forming a watertight membrane, against which the chamber pressure acts. This pressure not only prevents a face failure, but it also controls the soil displacements at the face, thus settlement associated with shallow tunneling can be effectively controlled and minimized. SPB TBMs have been used with considerable success at numerous difficult tunneling projects around the world (Babendererde, 1987). Their disadvantage is the relatively complex circulation and separation system for the bentonite slurry.

This problem has been to a great degree eliminated with the development of the Earth Pressure Balance TBM (Babendererde, 1991). The EPB TBM also has a separate, sealed and pressurized chamber at the front, filled with soil cuttings. The positive face pressure is maintained by pushing these cuttings against the face by pressure exerted through the cuttings from the bulkhead wall separating the front chamber from the rest of the machine.

The cuttings are removed by a screw conveyor, along which the chamber pressure is reduced to atmospheric level. The remainder of the spoil removal process is then of conventional nature. Unlike with the SPB TBM, where the face pressure is exerted hydraulically, the EPB TBM generates this pressure mechanically through the soil particles of the cuttings. Thus, the principles of earth pressure are in effect here, with the dependence of the pressure on displacement. This means that, with the EPB TBM, the pressure depends on the forward motion of the shield while with the SPB TBM it does not. This may lead to problems with face control in mixed face tunneling if one of the soil strata is markedly stiffer than the other one. Because the displacement related to the forward movement of the TBM remains the same for both layers, the softer layer generates less pressure than the stiffer one. The lower pressure may be insufficient for maintaining stability, and in the case of sand, may lead to uncontrolled ground losses. Nevertheless, in uniform face tunneling the EPB TBMs have been proven highly successful in even the most difficult ground conditions. Also, the EPB TBM can handle the final spoil removal with greater ease than the SPB TBM. For these reasons this category (EPB TBM) is gaining greater acceptance than the SPB TBM.

To design and operate a tunneling system with a positive face control (particularly modern EPB and SPB TBMs) one has to understand the complex interaction between the ground and the pressurizing medium (Eisenstein and Ezzeldine, 1994). However, there is no question that TBMs with positive ground control are superior to other tunneling techniques in controlling stability and deformations of the ground above the tunnel and as such are applied more and more often on subway projects around the world (see Section 4.0).

In tunneling through hard rock, the methods prevailing currently in practice are the Drill and Blast (D&B), the New Austrian Tunneling Method (NATM), and the Tunnel Boring Machines (TBM) as schematically outlined in Figure 3.2.

The D&B Technique, although not risk-free, is a tried and proven safe construction method that is still widely used. Only a few decades ago it was prevalent in rock tunneling but is now being increasingly replaced by the TBMs. There are two reasons for this trend: TBMs are more economical while eliminating harsh construction conditions (noise, smoke, vibrations, and vapor) associated with blasting.

The NATM, since its invention in the sixties, has become the dominant method for constructing tunnels of noncircular cross-sections and for regular tunnels requiring special ground treatment. With this method, the excavation is done by powerful roadheaders rather than by blasting.

At least four NATM projects around the world have experienced major collapses during the last year or two. At one time, there was concern that the method itself might be flawed based on the circumstantial fact that so many problems had occurred with the NATM within such a short period of time. However, it is now generally recognized that these problems were related to how the NATM was implemented on each project rather than a problem with the method itself. Thus, although also not risk-free, NATM is still considered a safe, viable method under the proper circumstances.

The method which currently shows the most rapid development and progress is the hard rock TBM. The reasons for this are obvious: fast rate of advance, continuous operation, minimum disturbance of the surrounding ground and the possibility of installing ground support as a part of the TBM operation.

The rate of advance is the single most important factor in controlling the cost of a tunnel, particularly of a long one. A most significant improvement in the rate of advance has been achieved by the introduction of the double shield TBM. Unlike the more conventional single shield machine, where the rate of advance is controlled by the "stop and go" nature of alternate drilling and support installation, the double shield advances continuously, almost without interruption. This is made possible by separating the front part of the TBM (the drilling) from its tail (the lining installation). The reaction needed to advance the drilling front is derived from a system of grippers installed between the front and tail sections of the TBM. The grippers are formed by a pair of symmetrical pads hydraulically expanded and pressed against the tunnel wall. Some possible configurations are shown in Figure 3.2. Thus the advance of the front is more or less independent from the lining installation and there is no "stop and go" cycle. Thus the rate of advance is almost double, compared to single shield TBM. However, to be an effective tunneling system, these grippers must be designed to be compatible with geotechnical ground conditions (Eisenstein and Rossler, 1995).

The current tunneling methods and technologies, as briefly outlined, above are often associated in design and construction with the Observational Method (OM). In this method field observations of a tunnel under construction are used as a feedback for continuous improvement or modifications of design for the remaining sections of the tunnel or for similar tunnels to be excavated in the future (Peck, 1969). The necessary prerequisites for an application of OM are the existence of a thorough monitoring system and of special contractual arrangements, which allow the changes in design to be incorporated during construction. The result of a judiciously applied OM is a safer and more economical tunnel.

Tunnels, being line structures, are exceptionally well suited for the application of and the benefits resulting from the OM. This is particularly true for longer tunnels excavated in relatively homogeneous ground. Observations made in the early parts of the tunnel excavation can be evaluated and utilized for the forthcoming sections.

The Observational Method can be applied with all the tunneling methods and technologies described above. Traditionally, the OM has been associated with the NATM. However, close monitoring and feedback is required with the other methods as well, in particular with the SPB and EPB TBMs, for more close pressure and soil displacement control.

The importance of these methods in today's urban tunneling is the subject of Section 4.0.

4.0 WORLDWIDE TUNNELING TECHNOLOGIES

The purpose of this section is to summarize examples of urban, shallow tunneling in geological and geotechnical conditions similar to those encountered in the Los Angeles area. Examples are selected worldwide, however, with special emphasis on projects from the United States, Canada, Europe, and Japan. Comparisons will be made particularly for conditions along the completed, currently constructed and planned segments of the Red Line.

The geological, geotechnical, and seismological conditions of the Los Angeles area and along the Red Line alignment are reviewed and evaluated in Section 2.0 of this appendix. Here, the ground conditions will be categorized from the tunneling point of view into three categories:

A. Soft ground formed by soils of the Old and Young Alluvium. This category consists of interspersed layers of granular and fine-grained soils. The granular soils are represented by sands and/or gravels with medium to high density, with variable content of fine-grained components (silt and clay). The fine-grained soils are represented by clays and/or clayey silts of firm to hard consistency.

B. Soft rock of the Puente, Fernando, and Topanga Formations. This category consists of stratified layers of claystone, siltstone, and sandstone. These layers are characterized by low strength, with SPT blow count below 100. The upper zone of about 20 feet is usually weathered up to a soil-like condition resembling the category of soft ground.

C. Hard rock consisting of three different types -- granitic rocks of plutonic origin, basalt of volcanic origin, and sedimentary rocks (conglomerates, siltstones, and shales). This ground category has not been encountered on the L.A. Metro project so far, but is expected to dominate tunneling through Santa Monica mountains.

The groundwater level varies between 4 to 35 feet below surface. This means that along some sections of the alignment the water level lies within the tunnel profile or slightly above, but seldom results in significant pressure. As with most stratified sediments there are also occasional perched water horizons. From the tunneling point of view the three ground categories can be characterized as follows:

In Category A, tunneling can be done with fully satisfactory results using either TBMs with positive face control or using the open face approach coupled with ground conditioning (GC). The ground conditioning may include chemical or cement grouting, mechanical spiling, compaction grouting, or forepoling. Only in the case of a homogeneous layer of stiff to hard clay can tunneling proceed with unsupported full face. Positive face control or ground conditioning is necessary for all granular soils and for clays of medium or softer consistency.

Category B is generally considered to be tunneling "friendly." The soft rocks offer sufficient stability and minimum deformation even in the case of open full face and, at the same time, allow the excavation to proceed with relatively low resistance to the cutting mechanism. Because the sedimentary rocks are of very low permeability there are usually no problems with groundwater control. Fortunately, a dual mode EPB TBM has been developed that can excavate Category B ground quickly and efficiently and yet can be quickly converted to full earth pressure mode whenever Category A ground is anticipated. Unfortunately, it takes some time to make the conversion and it is not automatic.

Category C -- the hard rocks -- seldom pose any problems in terms of stability or deformation, however, the excavation process becomes more demanding on cutting tools and energy. The traditional approach here is the drill and blast method, which is nowadays increasingly replaced by special hard rock TBMs. In the case of mixed tunneling, that is tunneling partially in hard rock and partially in softer ground, the modern TBMs come with cutting wheels equipped with more than one set of discs and scrappers in order to handle these versatile conditions.

An extensive set of examples of urban tunneling in soft ground has been compiled for the purpose of comparison with the Los Angeles conditions, and is presented on Table 4.1 (Summary of Urban Tunnels in Soft Ground).

The set contains 74 case histories of urban tunnels constructed approximately in the last 20 years. The tunnels were built in 18 countries including North and South America, Europe, Asia, and Africa. While the set does not claim to be completely exhaustive in the sense that it contains each and every project ever occurring, it is certainly large enough and does contain all the relevant and known projects to be of statistical significance.

By sorting the case histories according the type of soil, groundwater conditions, tunneling method, and the type of ground conditioning, one can obtain a picture of the current state of the art of urban tunneling.

Table 4.2 has been prepared to facilitate such a picture. In this table, the soil types have been grouped into three basic categories: granular (sand, gravel, and their mixture), fine-grained (clay and silt) and mixed face of granular and fine-grained. The tunneling methods will follow the categorization as developed in previous Section 3.0. Of the 74 case histories of urban tunnels, 43 were for subways and 31 for other purposes (roads, sewers).

An analysis of Table 4.2 reveals a number of important trends in current urban tunneling and offers several conclusions:

1. The vast majority of the tunnels (89%) were constructed using various type TBM. Only 11% were excavated by NATM or others.

2. Of the TBMs, the vast majority (76%) were types with positive face control (SPB, EPB, and APB TBMs).

3. OF and CF TBMs form only about 24% of the all cases.

4. About two-thirds of the tunneling methods which employ unsupported face (NATM and OF TBMs) are combined with some sort of ground conditioning (grouting or freezing).

5. In granular (non-cohesive) soils the TBMs with positive ground face control accounted for 76% of the cases while 14% of the cases were OF TBMs and NATM with ground conditioning. Only 10% of cases were with unsupported face.

6. In mixed face conditions (most typical for the Old and Young Alluvia of Los Angeles) the TBMs with positive face control represented 63% of all cases, with another 24% accounting for NATM or OF TBM with ground conditioning. Only 13% were attempted without apriori ground treatment.

7. The trend in present day urban soft ground tunneling is clearly toward methods using positive face control, regardless of the type of ground (granular or mixed face). The reason for this is the methods with positive face control offer a "blanket-type" protection against problems with excessive settlement or ground instability.

8. Open face tunneling (OF or CF TBMs and NATM) with ground conditioning is much less preferred, because of the additional cost and time requirements.

9. Table 4.2 includes 16 case histories of urban tunnels in the United States. The trend in the choice of tunneling methods here differs from the rest of the world. While TBMs are clearly the preferred technology (94% of all cases) only 40% of these cases employ some sort of positive face control. The remaining 60% are of the OF or CF TBM types, with about two-thirds of them combined with ground conditioning.

5.0 RISK

5.1 Introduction

The panel reviewed some of the existing concepts available for assessing risk in tunnel projects to develop broad guidelines for risk parameters and risk management systems which are, at the same time, appropriate to the L.A. subway. It was not possible within the time frame and resources available for this study to develop a new, project-specific set of risk parameters since such studies require months of effort and substantial budget to undertake.

5.2 Types of Risks and Parties Sharing Risk on Transit Projects

Risks come in all sizes and shapes. Generally, for a transit tunnel project, risks can be categorized in a variety of ways, one of which is by the stage of the project in which the risks might be triggered. These include the following stages:

- Planning and Financing
  - Conceptual Alignment Selection and Design
  - Scheduling
  - Budgeting and Financing
  - EIS/Community/Public Involvement
  - Voter Approval

- Design
  - Preliminary Alignment Selection and Design
  - Final Alignment Selection and Design
  - Plans, Specifications, and Estimates (PS&E)

- Construction
  - Bidding
  - Mobilization
  - Pre-treatment of Ground
  - Excavation
  - Initial Support
  - Waterproofing and Final Lining
  - Finishing Contract and Startup

- Service Period

Another useful method of evaluating or categorizing risk is according to the person or entity that is taking or sharing that risk. For transit projects, the following categories might be useful:

- Federal/State Agencies (Funders and Regulators)
- MTA (Owner) as a Planner and as a Financier
- MTA (Owner) as an Engineer (Planner, Designer, Construction Manager)
- General Design Consultant
- General Geotechnical Consultant
- Section Design Engineer
- Contractor
- Contractor's Subconsultants
- Construction Manager
- Third parties (such as adjacent property owners or overlying utilities) affected by construction
- Users or Riders

5.3 General MTA Risk Management Issues

Many of the planning decisions have already been completed by MTA but the impact of the risk of planning decisions still exists. For instance, the alignment has been selected for Segments 1 and 2. Though there are absolutely no known reasons to be concerned, there is always a risk that unanticipated or unappreciated problems may arise in terms of ridership or public acceptance. There is an equal risk that ridership will be far greater than anticipated and the system will become overcrowded far earlier than expected. Such are the risks of all transit projects.

MTA is currently addressing design decisions and the inherent risks associated with those decisions for the design of the mid-city and east-side extensions. MTA is currently also addressing planning and alignment decisions and the inherent risks associated with those decisions for the San Fernando Extension and for any extension from Wilshire to I-405. However, most of the visible short-term risks MTA currently faces involves design and construction for the sections which have been designed (Contracts B-251, C-331, and C-311, the Santa Monica Mountains tunnel).

Risks are generally not clear cut. Usually, some compromise has to be made in order to achieve any decision and thus the risks are different for each side of the compromise. For instance, a compromise usually has to be made between the method of construction (which determines the cost of construction) and the desired risk of settlement of the ground surface. The amount of expected settlement usually would be less and the risk of unexpected excessive settlement would be less if a better, yet more expensive, construction method would be used.

However, then there is a risk that the more expensive method might become too expensive. With the contracting methods used in the United States, that risk is not known until bids are received, at which time it is too late. Saddled on top of this simple example is the risk that the quality of construction may not be adequate, resulting in adverse settlements even when more expensive methods of construction are employed. For instance, the use of an Earth-Pressure Balance TBM does reduce the risk of adverse behavior but it does not guarantee excellent settlement behavior; EPB TBM's have been known to get into trouble too, particularly in mixed-face conditions.

The concept of risk as discussed above is one of a complex balance. There is also a risk of being too prescriptive in specifying more expensive measures to reduce the chances of adverse behavior. There is a chance that the more expensive measures may not be needed and this translates to a risk of spending money when less expensive construction methods could have done almost as well.

Finally, there is a positive side of risk. Though risk management is a major consideration on every project, many do get constructed on budget, on schedule, and with either none or a minimum of unexpected adverse behavior.

5.4 Examples of Risk Mitigation Measures

During the course of the panel's study, several examples of risk mitigation were identified. These include some measures that are currently being employed on MTA projects.

5.4.1 Los Angeles Metro Projects

On MTA projects, tunnel shields with full face breasting have been specified. This is more of a minimum requirement than a risk management measure. However, in addition, depending on the expected difficulty of tunneling and the resultant risk of ground settlement with the specified equipment, additional measures have been provided in the specifications. Generally this involves some type of ground improvement or GC, either by (1) chemical grouting or (2) compaction grouting from the ground surface. Chemical grouting conditions the ground by strengthening it, thus reducing any tendency for loss of ground into the tunnel heading. Compaction grouting involves the emplacement of grout bulbs into the ground which replaces the volume of ground already lost into the tunnel opening. These grouting measures are currently being paid for by MTA on a unit price basis.

Thus, there are several measures that reduce risk currently being taken on L.A. MTA projects. These measures, including the way they are paid, are traditional measures consistent with the state-of-practice in the United States.

5.4.2 Washington D.C. (WMATA) Projects

The panel contacted WMATA to learn their risk management practices. Generally, WMATA is contracting for their tunnels in much the same way as L.A. MTA except that it appears that WMATA tends to be more aggressive than some transit agencies by requiring certain construction methods to be used. They also tend to favor slightly more prescriptive-type specifications which specify certain equipment or procedures rather than leaving the decisions up to the low bidders.

WMATA reports that they consciously allocate additional money toward preventing or at least minimizing the risk of adverse ground movements. WMATA has specified the use of an earth-pressure balance machine on several of their projects. It should be noted, however, that the soil and groundwater conditions in Washington D.C. are generally more difficult to tunnel than those in Los Angeles, so the use of an earth-pressure balance machine on those projects may have been an engineering requirement as well as a way of reducing risk of adverse behavior rather than just a pure risk-mitigating measure. Nevertheless, WMATA has specified EPB TBMs several times. In addition, the panel understands that risk-mitigative measures such as chemical and/or compaction grouting are also specified on their projects. It is reported that provisions for grouting are also sometimes provided even when an EPB TBM is specified. It is estimated that about 3% of total project cost (including station costs) is spent on risk-mitigation measures such as grouting for improved ground control (Mergelesberg, 1995).

5.4.3 Edmonton Transit Project

Another example of risk management through the specification of certain equipment or construction methods is the case history of the most recent extension of the Edmonton, Canada subway in the early 1980s. On this project, ground and groundwater conditions varied markedly throughout the project and the transit agency elected to split the project into three different contracts, each having significantly different ground conditions. In the judgement of the transit agency and its engineers, it was decided to minimize risk of adverse behavior by specifying a different specific type of tunneling method for each of the three different ground conditions. The methods included (1) traditional shield, (2) NATM, and (3) Earth-Pressure Balance tunnel boring machine (EPB TBM). The projects were completed successfully, although one of the projects did experience a small sinkhole that was quickly repaired without major cost or undue attention.

5.4.4 Toronto Transit Commission (TTC)

There were several unique aspects of contracting practices recently undertaken by the Toronto Transit Commission (TTC) which are described and evaluated in the next section to estimate the cost of risk-mitigative procedures. They provide perspective into how various transit agencies might address risk management for tunnel construction.

The TTC decided that it would be in its best technical and financial interest if the TTC itself would purchase two tunnel boring machines directly from the manufacturer and provide them for use by the contractor selected as the lowest responsive bidder in a competitive bid. This was made financially attractive because the TTC had at least two long tunnel projects that could use the same EPB TBMs, thus allowing amortization over both projects rather than just the first project as would be in traditional contracting. The TTC thus elected to take all of the risk of the performance of the TBM in exchange for its own assurance that the machines would be well-suited to the ground conditions as determined by the TTC and its engineers. Furthermore, the TTC could purchase the EPB TBM early in the project, thus reducing the risk that a suitable tunnel boring machine might not be available in time to meet TTC's schedule. Moreover, for similar reasons and for economy of scale, the TTC also purchased the entire single-pass concrete lining for their two projects. Again, the lining is to be installed by the low bidder.

Such a departure from traditional contracting practice is unusual in North America but Canada has recently led the way. On another project near Toronto, the EPB TBM for the recently-completed Sarnia railroad tunnel was also provided by the Owner.

Though some types of risk are minimized when an owner provides a machine, new risks are introduced. For instance, there is some risk that an owner, in purchasing such equipment, may not, in fact, select the right machine or that the contractor is not as efficient operating a machine that was not selected by him to fit with his own equipment. Finally, there is a risk that the project will be delayed so that some of the advantages of early purchase are diminished. Such was the case in Toronto, since the Eglinton Line, for which the TBMs were purchased, was postponed and the TBMs will have to be stored until the startup of the next section, the Sheppard Line.

5.5 Discussion of Construction Risk

As can be seen, there may be some advantages for MTA to take a stronger position in the evaluation of the risks associated with different construction methods and in the selection of the methods as well as the degree to which MTA shares risk with others. The panel is not implying that previous contracting methods are incorrect since they are consistent with other US practices and since the practice has resulted in low tunneling prices. However, the panel believes that, considering the soil conditions on future alignments and the recent performance of tunnel projects, that MTA should also consider the use of earth pressure balance TBMs even if the cost is somewhat greater.

A very strong method of evaluating the risk of these new, and possibly more expensive, methods is to conduct a "what if" analysis of the various alternatives together with a careful evaluation of the likely cost of the project using the various alternatives. Thus, MTA can at least estimate what the likely cost of any required or specified item may be and compare that cost to the likely advantages and disadvantages. However, the cost analysis should take into account the total project cost, not just the cost of the item itself. This is because there is enough uncertainty in any cost estimate already without having to compound the uncertainty by not taking into account the total effect of any change. For instance, the cost of the use of EPB TBMs must consider the fact that EPB TBMs may be slower than other TBMs in certain ground conditions, thus possibly increasing the cost of the labor to construct the tunnel (see Section 6.0).

6.0 COSTS

6.1 Nature of Tunnel Cost Data

One of the most difficult tasks in the tunnel design process is developing a realistic cost estimate for the project. Characteristically, it is done two ways: (1) by a detailed contractor's type estimate or (2) by some system of correlating to past projects, usually on the basis of cost per foot taking into account similarities and differences between the various case histories. The difficulty with the second method is that the cost factors of a tunnel are extremely complex. The cost is not only a function of rational factors, such as quantities and unit prices, but it includes abundant intangible variables including bidding strategy, unbalanced bidding factors, bid strategy to improve a contractor's cash flow, whether or not the bidders are busy or idle (how hungry they are), the tone of the specifications, previous history with the owner and his construction manager, and other emotional issues.

Even seemingly rational factors like the amount and cost of labor to do a given amount of work are critically dependent on productivity and the duration of the job. The overall rate of advance is the primary factor that governs total cost as illustrated in Fig 6.1. Obviously, the rate of advance, and thus the cost, is highly dependent on the method of construction, the contractor's techniques, the cost of equipment (including TBMs) and materials, and the highly important variable of productivity.

In spite of these difficulties, the panel did evaluate the cost of MTA tunnels as well as transit tunnels elsewhere in North America and around the world on a unit price-per-foot basis. The convention used in this section is for the unit prices to be given as the unit cost per route foot for two tubes.

6.2 Los Angeles MTA Tunnel Costs

Cost data for the tunnels on the MTA project have been collected and are presented on Table 6.1 together with data on the performance of the tunnel. Some of the contracts involved stations or were complex projects having substantial building protection costs so that a pure cost per route foot of tunnel can not be extracted reliably. Where known, change orders have been reflected in the final price. In some cases, contracts have not been completed and thus the final change orders and final costs are unknown. In these cases, the data reflects the bid price and all change orders to date. The costs shown do not have the closeout costs for Contract B-251, which are unknown at this time, and they contain only the portion of the $4,100,000 grouting budget for C-331 which had been approved by change orders as of October 15, 1995.

Only a few projects are straightforward enough to extract a pure tunnel cost per route foot. On these projects, data on the cost of tunneling alone appear to be available from the following contracts.

Contract	**Project**	**Tunneling Cost Per Route Foot**	**Geology**
A-171	7th/Flower to Wilshire	$5,916/ft	Puente Fm.
B-251	Vermont/Hollywood	$5,556/ft*	Puente Fm & Alluvium
C-331	Univ. City to N. Hollywood	$6,343/ft*	Alluvium
C-311	Santa Monica Mountains	$9,440/ft*	Hard rock

*Cost based on bid price and all approved change orders as of October 15, 1995.

As will be shown, these costs are low to average compared to other transit projects in the United States and are quite low compared to tunneling costs outside of North America. Even though the costs for three of the tunnels are not finalized, the fact that they were bid at those prices confirms the fact that the MTA has received low bid prices for tunneling. This is a function of the favorable tunneling conditions, the highly competitive bidding climate and many other factors.

6.3 Cost Data for Transit Tunnels in North America

Cost data for other projects were collected from a variety of other sources to compare to MTA costs. Some data came from published data on original bid prices. Other data was made available from owners or other sources of data that had been collected by others for studying tunnel costs.

However, as on MTA, it must be realized that many tunnel projects are packaged with a station structure or system ventilation structure or other structure that prevents calculation of a pure unit price for a running foot of tunnel. Whenever it was known that the cost includes a non-tunnel structure, that fact is shown on the tables. Where possible, estimates of non-tunnel costs (stations, ventilation buildings, architectural treatments and track) are deducted from the bid price to obtain a more realistic unit price per foot for "tunnel." However, even if the bid sheets are available to obtain line item costs, there is frequently too much unbalancing of the bids for early cash flow generation, resulting in the true costs being masked. Finally, wherever the cost of change orders and claims were known, they were included in the estimated price, but such data was seldom available.

Cost data for other North American transit tunnel projects over the last few decades is given on Table 6.2. Again, the secrecy surrounding claims often prevents disclosure of these extra costs; extra costs regarding change orders and claims are not published as frequently as are the initial bids. Sometimes the initial bid price is all that is available. Since these data come from projects in different parts of the country and were built at different times, the basic costs have been adjusted according to the cost indices published by the Engineering News Record (ENR) to reflect inflation. On Table 6.2, even the MTA costs have been adjusted to bring the prices up to 1995. Selected soft ground or soil tunnel projects are compared graphically on Figure 6.1. It can be seen from Table 6.2 that the cost for tunnels around North America vary considerably from about $6,000 to $64,000 per route foot, but that MTA costs, which vary from about $6,000 to $14,000 per route foot are on the low side for soil tunnels.

Available hard rock tunnel costs are summarized in Table 6.3. Selected hard rock tunnel costs are compared graphically on Figure 6.2. The one rock tunnel at MTA (Contract C-311) is a difficult tunnel that will traverse an active fault, requiring special enlargement, and through several hard rock formations, including granite and basalt, with potential of considerable groundwater inflows as it passes through the Santa Monica Mountains. The current cost of C-311, based strictly on the bid price, is $9,440 per route foot. The most comparable project might be the Portland TRIMET project which is also through hard basalt and whose cost, as of July of this year, is $8,400 per route foot. The Dallas DART project is through very favorable, relatively low-strength chalk rock which is reflected in their lower costs of about $4,500 per route foot. Though MTA costs are slightly higher than some, it is believed that these costs reflect the difficulty of the project and are not out of line. Considering the fact that only $300,000 (0.25%) was left on the table, the bid price for C311 should be considered reasonable.

Finally, cost data for other transit systems in the world were briefly addressed by the panel. Because cost indices are not readily available for worldwide differences in regional prices and for different rates of inflation in different parts of the world, a direct comparison is extremely difficult. The project costs, not including adjustments for inflation, ranged from $20,000 to over $100,000 per foot, including the cost of stations. Rules of thumb previously developed by panel members put worldwide tunnel costs at about $12,000 to $15,000 per route foot (two tunnels but not including the cost of stations) depending on the difficulty. Station and completion costs typically doubles the cost of the tunnels alone. Thus on a project where the tunnel alone costs about $12,000 per route foot, the total cost of the project, including stations, might be on the order of $24,000.

6.4 Analysis of Tunneling Costs & Risk Mitigative Measures

The cost per unit length of tunnel can be classified into two basic components: (1) fixed costs and (2) variable costs. The fixed costs include all those items which do not vary with the duration of the construction project. These include items such as mobilization, the equipment (such as the tunnel boring machine, tunnel lining forms, etc.), the materials for initial and final linings, etc. The total bid price for these items can be divided by the length of the tunnel to obtain a fixed unit price per unit length of tunnel (dollars per foot) as shown in Figure 6.3.

The variable costs include all those which depend upon productivity and thus are related to the duration of the project. These would include all items of labor, overhead, daily operating costs, etc. The total variable costs equals the sum of the daily operating costs which, in its most simplistic terms, is the average daily operating costs averaged out over the duration of the project times the total number of days to construct the tunnel. Naturally, if the total time to construct the project is shorter, say, because of a faster average rate of advance, then the total amount of this variable component is less than a project of a longer duration. The total operating costs divided by the length of the tunnel gives a value for the variable cost which is the apparent average unit price of operating costs per foot of tunnel.

The effect of rate of advance on the variable component of tunnel costs can be shown as illustrated in the upper portion of Figure 6.3 where the unit costs are dramatically governed by the rate of advance.

This is an extremely simplistic description of the cost factors for a tunnel. In actual fact, the daily costs are not uniform since they start low, build to a crescendo during the highest production periods, then taper off for the close-out of the tunnel. Further, this simplified description of cost factors is more directly related to construction of a tunnel with a single-pass lining although the concepts are still valid for two-pass linings, except that the average rate of advance then relates to some average of excavation, final lining, and mob/demob rates. Nevertheless, the concept is sound and can be used to illustrate the effects of various factors, particularly rate of advance, on the total cost of tunnels, and in the risks of these costs and of sharing some of the uncertainties and risks in costs of tunnels.

Figure 6.3 also illustrates how two bidders who have essentially the same fixed costs, might bid a different price solely on the basis of their average anticipated rate of advance. Finally, the uncertainties associated with geology or with other unknowns affecting the project (i.e., the risks) are reflected primarily in the variable costs of the job. This is illustrated in Figure 6.3 where the potential effect of reducing uncertainties through the use of improved exploration and the use of a GDSR in the contract documents is shown by the arrow. Generally anything that reduces uncertainties, such as an improved geological exploration, the use of a GDSR in the contract documents, or risk sharing with the owner will reduce contingencies and thus reduce the low bid.

Figure 6.4 illustrates the costs and the potential risk sharing for improved technology and/or ground improvement specifically designed to reduce the amount of risks. The fixed costs in Figure 6.4 include a relatively small premium for the extra costs associated with an earth pressure balance machine (EPB TBM). If specified by the owner, this extra premium is indeed paid for by the owner in the bid price from all of the bidders. Figure 6.4 also shows the costs for pre-grouting ground (although this might involve any kind of ground conditioning). Once again, there is an extra cost associated with the grouting that would be done. If the pre-grouting were to be considered for these illustrative purposes to be a lump sum, then it too could be converted into an apparent fixed cost in these rate-of-advance diagrams. Accordingly, depending on how the owner includes risk sharing in the specifications, the pre-grouting may or may not be paid for by the owner or it may be part of the risk shared with the contractor.

Thus, the rate of advance diagrams described in the previous sheets can be used to illustrate many points regarding sharing of risk on tunnel projects. Similar diagrams may be used when evaluating the true cost of risk-mitigative measures in terms of total project cost.

6.5 Assessment of Cost of Risk Mitigative Measures

Frequently, special excavation methods or ground support techniques will be specified to achieve less risk to a tunnel project. These manifest themselves in several ways. Sometimes special excavation methods are specified to reduce risk. For instance, all MTA tunnels are excavated using a tunnel shield which is a large steel cylinder at the face which protects the working area and the workers temporarily until the initial lining is installed. Though this has become the minimum requirement for tunneling in soil, there is a cost to this equipment. The next level of ground support at the face might be a fully breasted face, which adds cost in the fact that there is extra work but, more importantly, the rate of advance likely decreases, all other conditions being the same.

As discussed in Section 3.0, two methods can be used to reduce risk associated with excavation: (1) improve the excavation technology and (2) improve the ground. There are extra costs associated with both methods; some improvements of both are listed below in Table 6.4 according to increasing costs.

Table 6.4

Excavation Methods and Ground Conditioning or Improvement

Excavation Technology
No Shield
Shield
Open Shield with breasting tables or doors

Ground Improvement
Dewatering
Compaction Grouting
Pre-grouting - Chemical Grouting

Excavation Technology
Wheel-type TBM with doors
Earth Pressure Balance TBM (EPB TBM)
Slurry Pressure Balance TBM (SPB TBM)

Ground Improvement
Freezing
Combinations of above

Two methods that might be considered by MTA are (1) EPB TBM and (2) pre-grouting. In recent years, WMATA has required one or, in a few cases, both of these two methods to reduce ground and building settlement or, in other words, to improve the likelihood of a successful project.

There is substantial extra cost of the equipment needed for an EPB TBM operation. However, it will be shown that, considering the cost of the whole project, the premium cost for EPB TBM may not be that great. The unit price for the initial outlay for improved equipment is, of course, lower for longer tunnels than for shorter tunnels.

The Toronto Transit Commission has purchased two EPB TBMs and all of the single-pass lining directly from the manufacturers. This gives a unique opportunity to evaluate the costs of the various elements of a tunnel on a very recent case history.

Based on the Toronto (TTC) experience, the earth pressure balance tunnel boring machines (EPB TBM) are estimated to cost about $7,000,000 each while traditional TBM machines would cost about $5,000,000 or $2,000,000 less each. Thus the extra cost for the use of two EPB TBMs for this project would be $4,000,000. It has been estimated that the total price of the tunnel would be about $70,000,000, including excavation and lining. Thus, on this project, the cost of the risk-mitigative measure of requiring an EPB TBM to reduce the risk of settlement would be $4,000,000, or only about 6% of the tunnel cost. Initially, it will be assumed that the rate of advance for the EPB TBM would be similar to that for traditional digger shields or for wheel type excavators. This assumption depends on the type of ground and many other factors. However, some of the EPB TBMs can be operated in a dual mode; open mode while driving through favorable ground and EPB mode when driving through unfavorable ground, thus increasing the potential for favorable advance rates.

On the other hand, if it is assumed that the use of an earth pressure balance machine will reduce the rate of advance significantly, the EPB TBM cost would be considerably more. Again, similar types of "what if" cost analyses can be conducted to evaluate the likely cost implications of any risk mitigation measure, including special equipment or additional pre-grouting of the ground. It is essential to take into account the effect of special equipment and/or grouting on the actual rate of advance. If there is a risk that a new technology would result in a considerably slower rate of advance, the cost of risk mitigation could be substantial. This premium can be evaluated by evaluating the cost impact of different rates of advance by cost analyses that should be done as part of the "what if" evaluation of the risk.

7.0 LOCAL AND MTA TUNNEL EXPERIENCE

7.1 Introduction

A large amount of information exists regarding the past tunnels that have been constructed in Los Angeles. An important beginning was made in the early 1970s by Richard Proctor who has continued to point out the favorable aspects of tunneling in the Los Angeles area (Proctor, 1973). Case histories on the previous tunnels were assembled and evaluated during planning and design phases of this project (Converse, et al., 1981). In addition, information regarding the construction and behavior of the tunnels constructed for Segment 1 and even for some of Segment 2 tunnels have been published. Many of these were published in the proceedings of the Rapid Excavation and Tunneling Conferences (RETC). A significant contribution was the field book guide published by Association of Engineering Geologists on the occasion of its 35th Annual Meeting in October 1992 (Stirbys, Radwanski, Escandon, and Daugherty, 1992).

As with many tunnel projects, uncertainties and unknowns as well as potentially hazardous situations exist and several of the previous local projects experienced difficulties that were taken into account during the design of MTA projects. Many of those problems are summarized in the sections below to illustrate that past tunnels in the area have experienced and have overcome significant problems. Thus, the problems at MTA are typical of tunnel projects elsewhere and are, by no means, isolated or are a direct function how the MTA project is being designed and constructed.

In spite of the problems experienced by previous tunnels as well as the problems associated with MTA tunnels, these hazards are manageable and the fact that there have been problems on previous local tunnels and on MTA tunnels should not be used as reasons for avoiding tunneling. For instance, many surface and aboveground construction projects have just as many or more problems and an even worse safety record than tunnels.

7.2 Previous Local Tunnel Experience in Soft Rock or Soil

The panel has briefly reviewed past local tunnel history in order to more fully understand the soil and groundwater conditions that must be tunneled as well as to appreciate the behavior of the ground during tunneling by the various methods used.

There is considerable experience in both soil and rock tunnels in Los Angeles. Most of the MTA tunneling to date has been in soil or soft rock which was of low enough strength that it could be excavated by heavy soil-type tunnel boring machines. Several of these previous case histories in the soft-rock/soil category plus a more recent case history of the NORS project are summarized in Table 7.1.

These case histories reflect the experience of the tunnels in the Puente or similar formations that were constructed prior to the MTA program. Many experienced considerable difficulties such as the fatal gas explosion in the San Fernando Tunnel and the several sinkholes using a CF TBM in dune sand and the one sinkhole using an earth pressure balance machine on the NORS Tunnel. Many tunnels experienced difficulty with methane but only one reported an explosion. On the other hand, the NORS tunnels benefitted by the techniques developed during Segment 1 for ventilation of gassy tunnels and for the use of magnetometer probing ahead of the tunnel which were adopted for the NORS design.

It should be noted that these case histories, some of which included record tunnel advance rates, attest to the good tunneling conditions in the Los Angeles area. They also confirm that many of these earlier tunnels experienced problems which they overcame. The fact that the remarks on the case histories given in Table 7.1 primarily relate to the problems experienced by the project should not be viewed as being negative but rather a confirmation that many of them experienced problems that were overcome and that these problems were taken into consideration during the planning and design of the MTA system. The fact that all the problems have essentially been forgotten is evidence that the project successfully overcame the problems and the tunnels are quietly continuing to perform their service quite satisfactorily.

Some of the problems experienced by these previous tunnels include:

- Methane
- High groundwater flows
- Boulders
- Hard cemented layers
- Caving
- Sinkholes
- Running ground
- Tar and oil seeps
- Explosion & Deaths
  - 17 died in the San Fernando Tunnel Explosion in 1971
- Abandoned and uncharted oil wells
- Stuck and abandoned TBM's

Although not specifically described in the previous tunnel case histories, Hydrogen Sulfide (H_2S) was identified early on the MTA project as a very important issue that certain MTA tunnel sections must resolve.

On the other hand, the previous local tunnel projects demonstrated that tunneling is feasible, that tunneling conditions are generally favorable, and that tunnels are relatively inexpensive in the Los Angeles area. In particular, the projects demonstrated that the tunnels behaved quite satisfactorily during earthquakes. In fact, while the San Fernando tunnel was under construction, it experienced the Richter Magnitude 6.4 earthquake of February 9, 1971 without any major damage in spite of the fact that the tunnel was in a fault block that tilted the east portal up 6 feet up relative to the west portal which, itself, shifted up some 1 1/4 feet. In fact, none of the tunnels in the Los Angeles area have experienced any significant damage during an earthquake.

Other favorable tunneling conditions that were demonstrated by the previous tunnels include the following:

- Very high rates of advance with open face shields
- Development and refinement of unbolted segmental precast concrete lining techniques for initial support (two pass system)
- Ability to cope with tar and oil seeps
- Ability to develop and routinely implement procedures to overcome the methane hazard while in revenue service as well as during construction

- Special tunnel designs for tunnels that cross active faults
- Development of magnetometer probe techniques to locate unknown oil well casings
- Development of improved construction ventilation methods

7.3 Previous Local Tunnel Experience in Hard Rock

Some of the local case histories were appropriate for the hard rock of the proposed Santa Monica Mountains tunnel. Two of these are summarized on Table 7.2. Fortunately, the cases were well documented by the construction teams and much valuable information was collected on the behavior of the rock in response to traditional drill and blast techniques.

These hard rock case histories are particularly important since it is very difficult, certainly not very cost effective or practical to drill deep borings on close centers for mountainous tunnels. Thus the tunnel case histories supplement the traditional exploration programs. More importantly, at least one case history is located approximately on the alignment of the MTA tunnels but some 120 feet higher than the MTA tunnels. Further, a parallel tunnel case history could possibly be far better than a series of widely spaced borings since borings only provide a 3-inch-diameter core some 500 to 1,000 feet apart. Thus, not all of the rock is seen in borings and the effect of scale is not fully appreciated. On the other hand, a full-size tunnel parallel to the proposed MTA tunnel exposes the rock and its behavior to tunneling at full scale, including size effects along the entire length of the tunnel.

The previous tunnels are not at the same depth as the MTA tunnel and thus do not go through the same weathering profile nor will contacts between strata occur at the same location. Finally, these case histories are sewer and water supply tunnels that are about half of the size of the MTA tunnel so extrapolation of behavior is still necessary as is an interpretation of how the rock behaved in response to drill and blast techniques in a smaller diameter tunnel when its response to a TBM needs to be predicted.

Clearly, significant groundwater inflow can be expected in the Santa Monica Mountains tunnel project C-311. Further, difficulties may be expected in the many fault and/or gouge zones that the tunnel has to cross. Special geotechnical considerations for design of TBM grippers should be addressed on C-311 (Eisenstein and Rossler, 1995).

7.4 Discussion of General Tunneling Conditions in L.A.

These case histories of successful previous projects document the fact that the previous tunnels and tunneling conditions were very good. The panel thus concludes that with a few exceptions, the geology in Los Angeles is generally favorable for tunneling than other projects worldwide because of :

- Low water table and/or dry soils
- Essentially no liquefaction potential
- Generally uniform ground conditions. Few mixed-face conditions
- Most of alignment is in competent ground
- Few obstructions, either natural or manmade

On the other hand, the reasons why Los Angeles tunneling conditions are less favorable than other projects around the world include:

- Unusual H_2S Hazard
- Seismic design and fault displacement potential
- Persistent methane problem

7.5 Experience of MTA Tunnels Constructed to Date

Table 7.3 summarizes the salient points of the MTA tunnel projects to date.

Table 7.3

Summary of MTA Tunnel Projects

A-130 East Portal to Union Station

- Contractor-Tutor-Saliba-Perini
- Bid Price: $37,677,803 on 5/20/88
- Final Price including changes: $37,417,116
- 1692 Route feet under Santa Anna Freeway and Brewery
- Geology: Alluvium: Coarse sands & gravels with some fine-medium sand and silt
- Excavation Method: Shield

- Initial Support: Ribs and lagging
- Surface Settlement:
  - Less than 1 inch except in one short section
  - Reportedly minimal settlement in grouted zone
  - One 10 cu yd (7.6 cu m) face loss entering into ungrouted area showed up as settlement at surface in vacant lot.
- Remarks:
  - Dewatered soils
  - Extensive chemical grout program from surface and from inside tunnel
    -- Innovative long hole drilling under freeway; holes up to 318 feet long
    -- 2 million gallons of Geoloc-4 chemical grout
    -- Largest grout project of its kind in United States
  - Fire in tunnel on July 13, 1990 resulted in collapse of ribs and lagging in areas that had not been grouted.
    -- Note, grouted areas did not collapse
  - Encountered residue from old coal gasification plant
  - Rate of Advance; Ungrouted ground: 21 to 29 feet per 20 hour day. Grouted ground: 12 to 16 feet per 20 hour day

A-141 Union Station through Civic Center Station to 5th/Hill Station

- Contractor-Tutor-Saliba/Groves
- Bid Price: $61,471,225 on 1/14/87
- Final Price including changes: $89,195,906
- 5,871 Route feet
- Geology: Puente rock and Alluvium
- Excavation Method: Robbins digger shield
- Initial Support: Ribs and lagging
- Surface Settlement
  - Little surface settlement; less than 1 inch everywhere and no grouting required
- Alignment Tolerance: Alignment out of tolerance; several adjustments to alignment were required

A-146 5th/Hill to 7th/Flower

- Contractor-Shank/Ohbayashi
- Bid Price: $18,221,820 on 2/12/87
- Final Price including changes: $24,970,552
- 2,150 Route feet all on a curve
- Geology: Alluvium
- Excavation Method: Mitsibushi digger shield
- Initial Support: Ribs and lagging
- Surface Settlement: Less than 1 inch everywhere despite some ground losses
  Zone 1 (600 feet) = 0.4 inch

Two ground losses (10 cu m and 2 cu m)
Zone 2 (660 feet) = 0.4 inch
Zone 3 (60 feet) = 0.3-0.5 inch
Significant number of ground losses up to 28 cu m
Zone 4 (840 feet) = 0.3inch
One ground loss of 31 cu m; one smaller of 7 cu m
Zone 5 (290 feet) = 0.5 inch

- Remarks:
  - Two compaction grouting test sections to confirm designer's concept; later elected to use chemical grouting
  - Developed a chemical grout canopy grout concept from inside the shield
  - Did compaction grouting from 720 holes
  - Rate of Advance: 40 ft/day

A-171 7th/Flower to Wilshire/Alvarado

- Contractor-Shank-Ohbayashi
- Bid Price: $26,340,078 on 12/12/86
- Final Price including changes: $29,669,697
- 5,015 Route feet
- Geology: Pliocene Fernando Formation in 95% of tunnel overlain by Older Alluvium; small section of old alluvium on east end
- Excavation Method: Mitsibushi digger shield
- Initial Support: Precast concrete segments
- Surface Settlement
  - Settlement less than 1 inch everywhere
- Remarks:
  - Encountered numerous tiebacks
  - Hard 3-foot-thick cemented bed slowed ROA to 85 and 95 feet for two days
  - Magnetometer survey 50 feet ahead-No wells detected
  - H2S was detected during construction; some gasoline fumes
  - Rate of Advance: AL=128 feet Ave 71.6 AR=136 feet Ave 97.4 (AR was 2nd tunnel)
  - Groundwater well below invert but some perched (1-2 gpm); mostly relatively dry

B-201 MacArthur Lake to Wilshire/Vermont Station

- Contractor - Tutor/Saliba/Perini
- Bid Price = $44,577,273 on 3/28/91 (Includes Pocket Track and cut-and-cover)
- Final Price = $50,683,155
- 4,536 Route feet
- Geology: Puente Rock
- Excavation Method: Digger shield
- Initial Support: Precast concrete segments

- Settlement
  - 70% less than 1 inch, 24% between 1 and 1.5 inch, 6% greater than 1.5 inch

B-221 Wilshire/Western to Wilshire Normandie

- Contractor - Tutor/Saliba/Perini
- Bid Price = $79,812,793 on 3/1/91 (Includes Cut-&-Cover & Station)
- Final Price = $92,333,199
- 4,894 Route feet
- Geology: Puente Rock & Alluvium
- Excavation Method: Digger shield
- Initial Support: Precast concrete segments
- Settlement
  - 96% less than 1 inch

B-251 Vermont/Hollywood

- Contractor - Shea/Kiewit/Kenny
- Bid Price = $129,504,695 on 5/28/92 (Includes Barnsdale Shaft)
- Final Price = $173,377,697 to date (includes increased tunnel length to 31,205 ft)
- 31,205 Route feet
- Geology: Puente Rock & Old & Young Alluvium
- Excavation Method: Digger shield
- Initial Support: Precast concrete segments
- Settlement
  - 51% less than 1 inch, 35% between 1 and 2 inches, 14% greater than 2 inches, Hudson Avenue settlement about 9 inches
- Alignment tolerance: Several sections out of alignment; Three sections remined
- Remarks
  - Increased tunnel length to improve contract packaging
  - Hudson Avenue 9+ inches settlement
  - Dewatering and Pillar Problems coming out of the shaft
  - Remining required in three sections
  - Sinkhole on Hollywood Boulevard
  - Contract terminated by MTA

C-331 North Hollywood

- Contractor - Ohbayshi
- Bid Price - $65,400,000 on 10/16/93
- Final Price: NA (in construction; current contract cost about $66,700,000)
- 10,515 Route feet
- Geology - Alluvium

- Excavation Method - Digger shield with breasting tables
- Initial Support - Precast concrete segments
- Settlement - Greater than 3 inches
- Remarks:
  - Excessive settlement on Lankershim. Stopped contractor Now chemical grouting
  - MTA approved $4,100,000 budget for chemical grouting from the street surface

C-311 Santa Monica Mountains Tunnel

- Contractor - Traylor Brothers/Frontier Kemper
- Bid Price $124,421,000 on 10/18/94
- 13,180 Route feet
- Geology: Hard Rock/basalt, granite etc
- Excavation Method: TBM

7.5.1 Discussion

MTA's tunneling program has not only experienced problems but it also has many successes to its credit. This section briefly addresses the overall tunnel performance with particular reference to the favorable aspects of the tunnel performance. The major problems that have been experienced are discussed separately in Section 8 of this appendix.

Many challenges and difficulties have been successfully overcome during planning, design, and construction of the MTA system. They represent appreciable improvements in the state of the art of tunneling and they include the following:

- Development of an alignment that avoids major obstacles.
- Development of HDPE barriers to eliminate methane and to minimize H_2S hazards
- Special development of seismic design criteria and earthquake design methods
- Full-scale demonstration of resistance to earthquakes by withstanding the Richter Magnitude 6.8 Northridge earthquake on January 17, 1994 with essentially no damage
- Development of design techniques for tunnels crossing active faults
- Recovered from major fire and collapse of A-130
- Implementation and refinement of compaction grouting methods to restrict settlement

- Conceptual design of mined station concept
- Development of construction ventilation techniques for gassy tunnels
- Development of magnetometer probe methods to search for uncharted oil wells
- Record tunnel advance rates
- Relatively low unit prices for tunnel construction

8.0 COMPARISON OF L.A. TUNNELING PROBLEMS TO WORLDWIDE EXPERIENCE

The purpose of this section is to assess the construction difficulties in tunneling which occurred at the Los Angeles Metro Red Line. The assessment will be done from the following points of view:

A. Geological and geotechnical conditions.
B. Tunneling methods and construction techniques.
C. Management system.
D. Comparison with similar projects in other cities.

It should be noted that the panel received data and was briefed on the issues discussed in this section of the Appendix but a detailed, all-inclusive independent investigation was not part of the scope of the panel. Such detailed investigations are being or have been conducted by others. Thus, the following discussions are a consensus of our current understanding of the sequences and causes of the problems discussed herein.

The problems experienced on MTA projects are neither of equal significance nor of equal impact on performance and cost of the project. Here, only those which are considered relevant from a geotechnical point of view shall be discussed in more detail. For instance, it is recognized that a large portion of A-130 collapsed after a fire in the tunnel which was already excavated but not lined. Though the fire and resulting collapse led to the prohibition of wood lagging, the fire did not start as a result of geotechnical reasons and thus and problem is not considered in the following section or in geotechnical settlement classifications.

The first major geotechnical tunnel concern was with the variable thickness of the cast-in-place concrete liner on Segment One. An extensive independent investigation and analysis has shown that the variation in thickness does not affect the structural adequacy or integrity of the liner, both for static and seismic loadings (Cording, 1994). The first question arising here is why the liner could not be designed thinner (and thus less expensive) in the first place. In determining liner thickness the designer has to consider two aspects. One of them is the thickness needed to carry all the expected loads. The other aspect is the constructability of the liner -- the minimum thickness required for pouring the concrete into the forms.

The latter aspect often is, and most likely was, the controlling factor in this case. The second question, then, is why the liner could not be installed in the first place with uniform thickness as originally specified. Then it must be realized that it is not possible in practice to install cast-in-place concrete liners with absolute accuracy and certain tolerances are always permitted. If these tolerances are exceeded, as they were in this case, the immediate reasons for this usually is inadequate steering control during the excavation of the tunnel. Problems of this nature sometimes occur at tunneling projects, normally as a result of either a lack of contractor's quality control or his selection of too small an excavated diameter for the tolerance that could be practically obtained using the alignment control method at the rates of advance being achieved.

The second incident which triggered widespread attention was the excessive settlement which occurred in August 1994 over the twin tunnel along Hollywood Boulevard near Hudson and Whitley Avenues. The surface settlement here eventually reached about 9 inches, a value almost an order of magnitude larger than what should be acceptable. As is often the case with construction accidents, there was not a single cause for this occurrence, but rather a chain of contributing events. A pattern of ground settlements normally considered acceptable along this section of the tunnel apparently caused a leak from an adjacent water main. The leaking water saturated the alluvial soils above the tunnel and altered their properties by decreasing shear strength and increasing compressibility. This contributed to more settlement above the tunnel, which now reached about 4 inches and to a loss of arching ability within the soil mass. The loss of arching led to a significant increase in soil loading on the primary liner in the tunnel. This increase in loading overstressed the wooden wedges used to support the gap in the expanded joints of the primary liner and these became eventually crushed. The gaps tended to close with additional 5 inches of settlement occurring on surface. Problems of this nature are relatively rare in the tunneling practice, but are not without precedence. This problem might have been prevented by more strict construction quality control and its seriousness reduced by more prudent design of the support of the liner gap or by filling the liner gap with dry-pack grout more quickly as specified.

The sinkhole which occurred above the south tunnel along Hollywood Boulevard in June 1995 received the greatest deal of publicity, both from the media and from the profession. It was again, like in the case above, a chain of events which ultimately led to this dramatic occurrence. Difficulties with steering the TBM resulted in the tunnel being off alignment by a distance which would no longer permit rectification by adjusting the position of the final cast-in-place liner. The tunnel thus had to be remined for a distance of 80 feet or 20 rings. When the remining procedure reached almost its end, the tunnel overburden had changed from a competent rock of the Puente Formation to a highly disintegrated rock, which was no longer compatible with the chosen remining process. This incompetent rock collapsed into the tunnel through a temporary opening in the roof. The collapse then propagated upwards to the street surface creating a chimney failure in the overlying Alluvium. The reasons for this incident can be found in the selection of a remining procedure that did not work and ultimately in the fact that the tunnel was driven too far out of alignment.

Finally, the fourth event which has received very little publicity so far but should be included in this review because of its importance is related to the excessive settlements experienced along the Lankershim Boulevard in North Hollywood. Here, although specified in the contract at 1 inch maximum, the settlement gradually increased as the tunnel progressed to 2 and 3 inches. At this point, further tunneling was stopped until appropriate measures would be taken. The problem here was the contractor's inability to control the face of the tunnel in the potentially unstable soils of the Young Alluvium. The contractor has selected to use a CF TBM with breasting tables to support the face. This technique is not sufficient for adequate ground control and must be combined with some other form of stabilization of the ground, e.g. grouting. This is, in fact, what the contractor is attempting to do at the time of writing this report.

In order to put these four isolated incidents within a broader context of construction difficulties occurring at urban tunneling worldwide, a summary of performance has been prepared for the same collection of case histories as presented in Table 4.1. In Table 8.1 (Performance of Urban Tunnels in Soft Ground) are summarized data about surface settlement and about other failures, wherever they had occurred. Also added in the table are data about the rate of progress achieved at these tunnels. Since only 36 case histories of the 74 cases presented in Table 4.1 had performance data included, the statistical set will be somewhat smaller, but nevertheless still significant.

For evaluation of the settlement data the following criteria shall be assumed, based on semiempirical analysis of allowable slope of the surface settlement trough:

1. As **Category 1** will be considered settlements with maximum value up to 1 inch (25 millimeters) which corresponds to a settlement trough slope of about 1:500. This deflection generally causes no visible harm to normal structures, buildings, or utilities.

2. As **Category 2** will be classified settlements with maximum values between 1 and 2 inches (25 to 50 millimeters). The corresponding settlement trough will reach about 1:300, which may cause visible, but not structurally dangerous damages to adjacent buildings.

3. As **Category 3** will be classified settlement with maximum value over 2 inches (50 millimeters). The resulting damage could be severe and might require structural repair. As Category 3 will also be classified all tunnels, regardless of their settlement levels, which experienced major difficulties, such as roof or wall failures or other forms of ground instability or structural insufficiencies.

To allow some inherent variations, greater settlement can be experienced in any category for a very short length of tunnel at a non-critical section, but any sinkhole or collapse automatically moves the tunnel to Category 3.

The following table presents an analysis of the settlement performance data (out of 36 cases total):

Category	**No. of Cases**	**Percent of Total**
1	16	44%
2	6	14%
3	15	42%

An immediate conclusion which can be drawn from the table above appears to be that slightly less than half of urban tunneling projects are completed without any difficulties while almost the same percentage ends up with serious problems.

In order to compare the Los Angeles history to the global performance levels derived above it is necessary to understand that the worldwide case histories summarized in Tables 4.1 and 8.1 do not refer to entire subway projects but rather to individual segments of these projects, usually represented by separate tunneling contracts. In fact, in several instances the tables contain more than one case history (tunneling segment) from the same subway system in the same city (e.g., Washington DC, Paris, Tokyo, London, Lille, Baltimore, Lyon, Edmonton). Thus, to make a meaningful evaluation of Los Angeles Metro performance, one has to look at the tunneling history here in terms of individual contracts rather than in terms of the entire system. There have been eight separate tunneling contracts completed or under construction in Los Angeles so far and these are listed in Table 6.1. One can compare the worldwide levels of performance with those at the MTA contracts by combining the two tables:

Category	**Worldwide (36 cases)**	**MTA (8 cases)**
1	16 cases (44 %)	4 cases (50%)
2	5 cases (14%)	1 case (12.5%)
3	15 cases (42%)	3 cases (37.5%)

The conclusion which can be derived from comparison of the simple statistical comparison above is that the Los Angeles Rail Metro project in terms of its overall performance is about equal to or slightly better than the world average.

This conclusion does not mean that there is no room for improvement on this project. In fact, there are several technical and management aspects which emerged from this study as desirable for improvement or change.

9.0 FEASIBILITY OF TUNNELING IN LOS ANGELES

In relation to construction feasibility, the geological and geotechnical environment along the existing and proposed corridors of Los Angeles Metro is clearly compatible with safe and economical underground construction. Dozens of cities in various countries have successfully developed underground transportation systems in similar or even more difficult ground conditions. About half of these cities have experienced difficulties comparable to or even worse than those which occurred in Los Angeles. Comparing this with the Los Angeles construction history, which includes eight separate tunneling contracts and four major geotechnical problems on these contracts, a conclusion can again be drawn that the Los Angeles subway tunneling is about equal to or slightly better than the world average. However, the other half of the case histories shows that shallow urban tunneling can be carried out entirely without major problems or undue interference with normal urban life and still at reasonable cost. It is this latter half of the subway tunneling spectrum that should become the target for Los Angeles now.

The Los Angeles Metro tunneling contracts carried out so far utilized tunnel boring machines with either open face or face partially protected with breasting plates or breasting tables. In ground conditions where the open face or the partial protection was not sufficient to control the ground, this tunneling method was complemented by ground improvement, mostly in the form of grouting. This approach was adequate for the majority of the tunneling, except for a few instances where excessive settlement occurred for reasons discussed before.

As reflected by the documented trends in soft ground tunneling worldwide, the risks involved in methods depending on ground conditioning are increasingly eliminated by turning to tunneling methods using positive face control. These methods offer a "blanket" type protection against ground deformation and instability and are much less dependent on factors such as ground variation or workmanship which play an important role with grouting.

In Los Angeles, where there are relatively few problems with groundwater, the optimal tunnel boring machine to be considered might be an earth pressure balance machine (EPB TBM) capable of operating in two modes. The first mode would be an open face mode, to be applied in competent ground (e.g., the Puente Formation). The second mode, to be used in the alluvial soils, would be the earth pressure balance mode, with the face under active pressure. Should ground water become a serious problem, the earth pressure balance machine in closed mode is well-equipped to handle such a situation. In addition to considering an open-face shield, MTA should at least consider the advantages and disadvantages and cost implications of EPBM as one means of reducing risk of significant settlement and minimizing public disruption on future projects. Such equipment may or may not cost more.

REFERENCES

12-1-95/W7101-01.RPT/W7101-lkd/dgw

REFERENCES

Babendererde, S. (1991) "Tunneling Machines in Soft Ground: A Comparison of Slurry and EPB Shield Systems," Tunneling and Underground Space Technology, Vol. 6 No. 2, pp. 169-174.

Babendererde, S. (1987) "Underpassing the Rhone in Lyon, France With a Hydroshield and Extruded Steel Fiber Concrete Lining," RETC, Vol. 1, pp. 611-623.

Converse Consultants (1993) "Geotechnical Design Summary Report for Construction Contract Unit C331 North Hollywood Tunnel," Report prepared for Engineering Management Consultant, July.

Converse Consultants (1983) "Seismological Investigation and Design Criteria," Report prepared for Southern California Rapid Transit District, Metro Rail Project, May.

Converse Ward Davis Dixon (1981) "Geotechnical Investigation Report," November 1981, Report prepared for Southern California Rapid Transit District, Metro Rail Project.

Cording, Edward J., De Marco, Paul, and Hanson, John (1994) "Investigation of Structural Integrity of Los Angeles Metro Red Line Tunnels, Segment 1," Report prepared for Los Angeles County Metropolitan Transportation Authority, February.

Dolan, James F., Sieh, Kerry, Rockwell, Thomas K., Yeats, Robert, Shaw, John, Suppe, John, Huftile, Gary J. and Gath, Eldon (1995) "Prospects for Larger or More Frequent Earthquakes in the Los Angeles Metropolitan Region," Science, Volume 267, 13th January, pp. 199-206.

Dowding, Charles H. and Rozen, Aaron (1978) "Damage to Rock Tunnels from Earthquake Shaking," Journal of the Geotechnical Division, ASCE, Vol. 104, No. GT2, February, pp. 175-191.

Eisenstein, Z. and Ezzeldine, O. (1994) "The Role of Face Pressure for Shields With Positive Ground Control," Proceedings, International Congress on Tunneling and Ground Conditions, Cairo, pp. 557-571.

Eisenstein, Z. and Rossler, K. (1995) "Geotechnical Criteria for Double Shield Tunnel Boring Machines," Proceedings, World Tunnel Congress, "Worldwide Innovations in Tunneling," Stuttgart, Germany, May 6-11 1995, pp. 192-201.

Ellioff, M.A., Smirnoff, T.P., Ryan, P.F., Putnam, J.O. and Ghadiali, B.M., "Geotechnical Investigations and Design Alternatives for Tunneling in the Presence of Hydrogen Sulphide Gas -- Los Angeles Metro" Proceedings 1995 Rapid Excavation and Tunneling Conference, San Francisco, pp. 299-318.

Engineering Management Consultant (September 1994) "San Fernando Valley East-West Rapid Transit Project -- SP Burbank Branch Alignment Pre-Preliminary Engineering Study," Report prepared for Los Angeles County Metropolitan Transportation Authority, September.

Engineering Management Consultant (1994) "Mid-City Extension Reassessment Study," an interim report on construction options prepared for Los Angeles County Metropolitan Transportation Authority, July.

EQE (1995) "The January 17, 1995 Kobe Earthquake, An EQE Summary Report."

Escandon, Richard F., Stirbys, Anthony F., and Daugherty, Charles W. (1992) "Los Angeles Metro Rail Project: History and Construction of Phase I," Los Angeles Metro Rail System Field Trip Guide Book, Association of Engineering Geologists, 35th Annual Meeting, Long Beach, October.

Geotransit Consultants (1994) "Geotechnical Investigation for Preliminary Engineering Program, Eastside Extension Metro Red Line Project," Report prepared for Engineering Management Consultant.

Geotransit Consultants (July 1994) "Final Draft Fault Investigation for Preliminary Engineering Program, Eastside Extension, Metro Red Line Project", Report prepared for Engineering Management Consultant.

Mergelsburg, W.A. (1995) Personal Communication.

Metro Rail Transit Consultants (1984) "Supplemental Criteria for Seismic Design of Underground Structures", Report prepared for Southern California Rapid Transit District, June.

Monsees, J.E. and Merritt, J.L. (1991) "Earthquake Considerations in Design of the Los Angeles Metro", Proceedings, ASCE Conference on Lifeline Earthquake Engineering, 1991, pp. 75-88.

Navin, Stephen J. (1991) "HDPE as a Gas and Water Barrier, The Los Angeles Metro Red Line Experience," Proceedings, 1991 Rapid Excavation and Tunneling Conference, Seattle, June, pp. 253-265.

O'Rourke, T.D. (1995) Personal Communication.

Parsons Brinckerhoff Quade and Douglas Inc. (1994) "Geotechnical Design Summary Report, Part L, Contract CO311," Report prepared for Engineering Management Consultant, July.

Peck, R.B. (1969) "Advantages and Limitations of the Observational Method in Applied Soil Mechanics," Ninth Rankine Lecture, Geotechnique, June 1969, pp. 171-187.

Petersen, Mark D., and Wesnousky, Steven G. (1994) "Fault Slip Rates and Earthquake Histories for Active Faults in Southern California," Bulletin of the Seismological Society of America, Vol. 84, No. 5, pp. 1608-1649, October.

Power, Maurice (1995) Personal Communication.

Proctor, R.J. (1981) "Los Angeles Basin Geology and Local Tunneling Experience," Bulletin Association of Engineering Geologists, Symposium Issue, Vol. 18, No. 2, pp. 141-146.

Robison, M.J., and Wardell, S.R. (1991) "Chemical Grouting to Control Ground Losses and Settlements on Los Angeles Metro Rail Contract A146," Proceedings, 1991 Rapid Excavation and Tunneling Conference, Seattle, June, pp 179-195.

Sharma, Sunil and Judd, William R. (1991) "Underground Opening Damage from Earthquakes," Engineering Geology, Vol. 30, pp. 263-176.

Stirbys, Anthony F., Radwanski, Richard Z., Escandon, Richard F. and Daugherty, Charles W. (1992) Los Angeles Metro Rail System Field Trip Guide Book, Association of Engineering Geologists, 35th Annual Meeting, Long Beach, October.

The Earth Technology Corporation (1990) "Metro Rail Project, Summary of Subsurface Conditions", Report prepared for Metro Rail Transit Consultant.

The Earth Technology Corporation (1993) "Geotechnical Investigation Report, Santa Monica Mountains Segment 3, Metro Red Line," Report prepared for Engineering Management Consultant, July.

Wang, Jaw-Nan (1993) Seismic Design of Tunnels: A Simple State-of-the-Art Design Approach, Parsons Brinckerhoff Quade and Douglas Inc., New York, Monograph 7.

Working Group on California Earthquake Probabilities (1995) "Seismic Hazards in Southern California: Probable Earthquakes, 1994 to 2024," Bulletin of the Seismological Society of America, Vol. 85, No. 2, pp. 379-439, April.

CASE HISTORY REFERENCES

No.	Reference
1	Anheuser, L., 1982. Gemessene Setzungen uber mit dem Hydroschild Aufgefahrenen Tunneln. In Forchung + Praxis, Construction underground-Present and Future, STUVA, pp. 120-128.
2	Anheuser, L., 1985. Technologie et Application recentes des Boucliers a Boue Allemands. Tunneling in Soft and Water-Bearing Grounds, M. Legrand, Editor, pp. 15-20.
3	Cloquet, V, 1985. Aspects Practiques des Entrees at sorties du Bouclier a la Bentonite au Droit des Puits d'acces et Gares. Protection des ouvrages le long du Trace. Tunnelling in Soft and Water-Bearing Grounds, M. Legrand, Editor, pp. 29-36.
4	West, G., 1988. Innovation and the rise of the tunnelling industry. Cambridge University Press, publisher, 355 pages.
5	Kuesel, T., and Parker, H.W. 1981. Marta's broad street tunnels. RETC 1981, San Francisco, American Institute Min., Metallurgical, and Petroleum Engineers, Vol.2, pp. 393.
6	Desai, D.B. 1979. North American subway system-Baltimore rapid transit project. RETC 1979, Atlanta, Georgia, AIMMPE, Vol. 2, pp. 1545.
7	Edwards, C.A., and Merrill, K.D. 1995. Baltimore Metro Northeast extension. RETC 1995, San Francisco, Cal., Society for Mining, Metallurgy, and Exploration, Inc., pp. 1545.
8	Smith, R., Eisold, D.E., and Schrad, M. 1993. Compressed air tunneling through a contaminated zone. RETC 1993, Boston, Mass., AIMMPE, pp. 735.
9	Wallis, S., 1990. Redesigned hydroshield gives boulders short drift. Tunnels and Tunnelling, June, 1990, pp. 41-42.
10	Meyer, D.F. 1983. Metropolitan Boston transit authority Red Line project Harvard Square to Porter Square tunnels Cambridge, Massachusetts. RETC 1983, Chicago, Illinois, AIMMPE,. Vol. 2, pp. 941.
11	Almeraris, P.E., Peyton, P.E., Skanska Tunneling Inc 1991. Mixed type tunneling beneath metropolitan Boston. RETC 1991, Seattle, Wash., AIMMPE, pp. 275.

12 Wallis, S. 1994. Mixshields for the Cairo Metro. Tunnel, STUVA ed., 6/94, pp. 35.

13 Cairo Metro's Line 2 on course. Tunnels & Tunnelling, 1995, June, pp. 13.

14 El-Nahhas, F. 1994. Some geotechnical aspects of shield tunnelling in the Cairo area. International Congress on Tunnelling and Ground Conditions, Cairo, Balkema, April 1994, pp. 11.

15 Paulus, J.F., Avesque, A., and Pasqual, P., 1988. L'Utilisation de deux Tunneliers Lovat en Terrains Alluvionnaires et Schisteux (Metro de Caracas). Collectivites territoriales et utilization de sous-sol, Bordeaux, M. Legrand, editor, pp.351-355.

16 Romo, L.A., Ferrer, J.A., and Silva, J. 1979. Metro tunnels, Caracas, Venezuela. RETC 1979, Atlanta, Georgia, Vol. 2, pp. 355.

17 Cologne - Tunnel 3/94, pp. 10.

18 Mohkam, M. 1991. Combining proper confinement pressure and adequate slurry for passing beneath existing structures securely with slurry shield - case study. RETC 1991, Seattle, Washington, AIMMPE, pp. 309.

19 El Nahhas (1980) Ph.D. thesis U of A.

20 Eisenstein (1981) Pressure - displ. relations in two system of tunnel lining. Soft ground Tunnelling, Failures and Displacements (ed. D. Resedniz and M.P. Rommo) A.A. Balkema (Rotterdam) pp. 85.

21 Branco (1981) M.Sc. thesis, U of A.

22 Eisenstein, Branco (1985) Canadian Tunnelling (TAC) p. 51.

23 Krywiak, G.J. and Morison, R.M., 1990. Analysis and design aspects of the precast concrete tunnel installed by the hydroshield method. Unpublished report.

24 Jones, I., 1990. SLRT tunnel construction-hydroshield method. Unpublished report.

25 Eisenstein, Z. and Ezzeldine, O., 1992. Ground control by shields with positive soil support. Cairo 2nd International Symposium on Tunnelling; Current Experiences in Tunneling, Jan. 27-28, pp. 75-88.

26 EOLE - Tunnel 1/95, p. 6.

27 Melzer, K.H., Lauruschka, K. 1995. Recent experience with the mixshield under difficult geological conditions. T&T-Bauma'95, Special Issue, April 1995, p. 21

28 Hollstegge, W., and Meseck, H., 1989. This shield-tunneling with steel lining in a mining subsidence region. 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, pp. 1493-1496.

29 Steiner, W., 1992. Dual-mode machine for the Grauholz tunnel. Tunnels and Tunnelling, Jan., 1992, pp. 58-60.

30 Steiner, W., 1989. Grauholz tunnel, tunnelling with a slurry shield. Tunnels and Water (J. Manuel Serrano, editor), Balkema Rotterdam, pp. 1381-1385.

31 Steiner, W., and Becker, C., 1991. Grauholz tunnel in Switzerland: Large mixed face slurry shield. RETC, pp. 329-347.

32 ITA, International Tunnelling Association's Working Group on Research, 1981. Development and future trends on shield tunnelling. Advances in Tunnelling and Subsurface Use, 1, (3), pp. 199-365.

33 Japan Tunneling Association 1995. "Challenges and Changes" Tunneling activities in Japan 1994. TUST, Vol. 10, No. 2, p. 203.

34 Tamai, T., Chiba, Y., and Yoshimura, M., 1989. The soil plasticizing shield driving method using a super-absorbent polymer. International Congress on Progress and Innovation on Tunnelling, Toronto, Canada, pp. 431-438.

35 Lile - Tunnel 1/95, p. 46.

36 Lisbon - Tunnel 4/94, p. 13.

37 Chaffois, S. Lareal, P,. Monnet, J., and Chapeau, C., 1988. Study of tunnel face in gravel site. Numerical Methods in Geomechanics, Innsbruck, pp. 1493-1498.

38 Bouyat, C., Mohkam, M., and Mocrette J.P., 1985. Le Soutainement Liquide, une Recherche Appliquee pour le Bouclier a Pression de Boue du Metro de Lyon. Revue Francaise de Geotechnique, N. 32, pp. 59-69.

39 Ferrand, J. and Morcrette, J.P., 1988. Metro Ligne D: Premiers Resultats et Bilan des Chantiers de Genie Civil. Tunnels and Water (J. Manuel Serrano, Editor), Balkema Rotterdam, pp. 869-876.

40 Ferrand, J. and Bouyat, C., 1985. Le Metro de Lyon et son Bouclier a Pression de Boue. Tunnelling in Soft and Water-Bearing Grounds, Proceedings of an International Symposium organized by AFTES (Association francaise des Travaux en Souterrain), Lyon, France, pp. 21-28.

41 Babendererde, S., 1987. Underpassing the Rhone in Lyon, France with a Hydroshield and Extruded Steel Fiber Concrete Lining. RETC, Vol.1, pp. 611-623.

42 Morcrette, J.P., and Bouyat, C., 1986. Metro de Lyon. Les Difficultes du Creusement sous le Rhone aves le Bouclier a Pression de Boue. International Congress on Large Underground Openings, Firenze, Italy, pp. 822-827.

43 Morcrette, J.P., 1988. La Traversee sous Fluviale de la Lgne D du Metro de Lyon. Collectivites territoriales et utilization du sous-sol, Bordeaux, M. Legrand, editior, pp. 335-339.

44 EPBM to start Lyon bore. T&T, Dec. 1994, p. 13.

45 Schmitter, J.M., Lopez Portillo, M.V., and Orozco, J.C., 1988. Slurry shield tunnelling for Mexico City. Tunnelling 88', 5th International Symposium organized by the Institution of Mining and Metallurgy, London, England, pp. 289-294.

46 Axhausen, K. 1995. Compressed-air tunnel shield with roadheader boom for the Munich underground (Line U2 East Lot 1). T&T-Bauma'95 Special Issue, April 1995, p. 17.

47 Sasanabe, M. and Matsubara, H., 1986. Planing and results of large diameter tunnel driven by soil plasticizing shield. Chichibu section of Nagoya municipal subway. International Congress on Large Underground Openinings, Firenze, Italy, pp. 925-932.

48 Kuesel, T.R., 1972. Soft ground tunnels for the BART project. RETC, Chicago, Vol. I, pp. 287-313.

49 Hashimoto, K., Kato, T., Watanabe, Y., Matsuda, M., Tamiya, Y., Shima, M., Fujihara, Y., and Tadao, Y., 1988. Underground storm water reservoir constructed by 11.22 m slurry shields. Tunnels and Water (J. Manuel Serrano, editor), Balkema Rotterdam, pp. 473-488.

50 Hashimoto, K., Takeyama, T., and Kurosawa, S., 1985. Tunnel excavation with the world's largest slurry shield. Tunnelling in Soft and Water-Bearing Grounds, M. Legrand, Editor, pp. 45-52.

51 Duddeck, H., Meister, D., Werner, E., Schlegel, R., and Theurer, M. 1979. Road tunnel in very soft soil in terrace deposits in the Harz Mountains, Germany. Tunnelling '79, Inst. Min. Engineers and the Transport and Road Research Laboratory, p. 205.

52 Marcheselli, P., and Peron, J.Y. 1995. Construction of the "Passante Ferroviario" link in Milano, Lots 3P - 5P - 6P. Excavation by large earth pressure balanced

shield with chemical foam injection. RETC 1995, San Francisco, Cal., Society for Min. Metall. and Exploration, Inc., pp. 549.

54 Cassinis, C., Nisio, P., and Capata, V., 1985. Settlement and face stability boring a large tunnel. 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco, pp. 2057-2062.

55 Pandolfo, A., 1986. Il Sottoattraversamento Ferroviario di Centri Urbani con Scarsa Copertura ed in Ambienti Idrogeologici Particolrmente Difficili. Internationa Congress on Large Underground Openings, Firenze Italy, pp. 252-263.

56 Cappellari G. and Ottaviani, M., 1982. Predicted surface settlements due to shield tunneling with compressed air. Proceedings of the 4th International Conference on Numerical Methods in Geomechanics, Edmonton, Vol. 2, pp.531-536.

57 Wallis, S., 1984. Hydroshield Alleviates the pressure on Rome's urban railway drives. Tunnels and Tunnelling, Oct., 1984, pp. 11-16.

58 Kuesel, T. 1972. Soft ground tunnels for the BART project. RETC 1972, Chicago, Illinois, AIMMPE, Vol. 2, pp. 287.

59 Sutter, D.A., Frobenius, P., and Ching Wu 1995. Tunnel design of MUNI metro turnback project. RETC 1995, San Francisco, Cal., Society for Mining, Metallurgy, and Exploration, Inc., pp. 703.

60 Clough, G.W., Shirasuna, T., and Finno, R.J., 1985. Finite element analysis of advanced shield tunneling in soils. 5th International Conference on Numerical Methods in Geomechanics, Nagoya, pp. 1167-1174.

61 Clough, G.W., Sweeney, B.P., and Finno, R.J., 1983. Measured soil response to EPB shield tunneling. Journal of Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 109, pp. 131-149.

62 Rowe, R.K. and Kack, G.J., 1983. A theoretical examination of the settlements induced by Tunnelling: Four case histories. Canadian Geotechnical Journal, Vol. 20, pp. 299-314.

63 Finno, R.J., 1983. Response of cohesive soil to advanced shield tunneling. Ph.D. Thesis, Stanford University.

64 Kasali, G., and Clough, G.W., 1983. Development of a design technology for ground support for tunnels in soil-Volume III-Three dimensional finite element analysis of advanced and conventional shield tunneling. Report No UNTA-MA-06-0100-82-2 submitted to the U.S. Department of Transportation, Washington, D.C., 20590, 232 pages.

65 Clough, G.W., Finno, R.J., Sweeney, B.P., and Kavazanjian, E., 1983. Development of a design technology for ground support for tunnels in soil-volume III-Observed behavior of an Earth pressure balanced shield in San Francisco Bay Mud. Report No UNTA-MA-06-0100-82-3 submitted to the U.S. Department of Transportation, Washington, D.C., 20590, 196 pages.

66 Negro, A. Jr., Eisenstein, Z. 1981. Ground control techniques compared in three Brazilian tunnels. Part 1. T&T, 1981 Oct., pp. 11.

67 Negro, A. Jr., Eisenstein, Z. 1981. Ground control techniques compared in three Brazilian tunnels. Part 2. T&T, 1981 Nov., pp. 52.

68 Negro, A. Jr., Eisenstein, Z. 1981. Ground control techniques compared in three Brazilian tunnels. Part 3. T&T, 1981 Dec., pp. 48.

69 Sao Paulo - Simondi, Negro, Kuperman (1982) Proc. Symp. on Concrete for Foundation and Underground Structures, Sao Paulo, p. 42.

70 Amaral, L.R., and Frobenius, P. 1974. Tunnel construction for the Sao Paulo subway. RETC 1974, San Francisco, Cal., Vol. 2, pp. 1213.

71 Robinson, R.A., Kucker, M.S., Parker, H.W. 1991. Ground behavior in glacial soils for the Seattle transit tunnels. RETC 1991, Seattle, Wash., AIMMPE, pp. 93.

72 Critchfield, J.W., and MacDonald J.F. 1989. Seattle bus tunnel construction. RETC 1989, Los Angeles p. 341.

73 Maidl, B., Hou, X., and Yi, X., 1990. Field measurement of ground movements and variations of pore pressure caused by EPB shield construction. Tunnel and Underground Works Today and Future. International Congress, The International Tunnelling Association Annual Meeting Chengdu, China, 1990, pp. 335-349.

75 Shimizu, R., and Maeda, M., 1989. Design and construction of the Shinkawa tunnel using the mudwater-type shield method. International Congress on Progress and Innovation in Tunnelling, Toronto, Canada, pp. 431-438.

76 Shirlaw, J.N., and Doran, S., 1991. Ground movements and settlements caused by tunnelling for the Singapore Mass Rapid Transit System. Tunneling 88', 5th International Symposium organized by the Inst. Min. and Metall., London, England, pp. 295-314.

77 Dietz, W., and Kohle, R. 1994. The Taipei Metro - Comparison of findings using different earth pressure balance shield systems. Tunnel, STUVA ed.,6/94, p. 5.

78 Stuva: tunnels stand up to Kobe's earth shaker. T&T, June 1995, p. 15.

79 Masuzaki, S., Yamada, K., and Tanaka, H., 1986. An example of observation for risk reduction against existing underground structures in shield tunnelling. International Congress on Large Underground Openings, Firenze, Italy, pp. 630-639.

80 Matsumoto, H., Takamatu, Y., Kwada, H., and Arai, T., 1989. Results and future prospects of MF shield tunnelling. International Congress on Progress and Innovation in Tunnelling, Toronto, Canada, pp. 1011-1016.

81 Matsumoto, Y., Arai, T., Ohta, H., and Ooishi, Y., 1991. An experimental study on the multi-circular face shield. RETC, pp. 815-828.

82 Magata, H., Nakauchi, S., and Sogabe, H., 1982. Subsidence measurements associated with shield tunneling in soft ground. Tunnelling 82', 3rd International Symp., Inst. Min. and Metall., London, England, pp. 231-240.

83 Seychuck (1977) Ph.D. thesis, U of Illinois.

84 DeLory, F.A., Crasford, A.M., and Gibson, M.E.M. 1979. Measurements on a tunnel lining in very dense till. Canadian Geotechnical Journal, v. 16, no. 1, p. 190.

85 Schmidt, B. 1974. Prediction of settlements due to tunneling in soil: Three case histories. RETC 1974, San Francisco, Cal., AIMMPE Vol. 2, pp. 1179.

86 Deix, F., and Braun, B. 1987. The use of NATM in combination with compressed air and ground freezing during Vienna subway construction. RETC 1987, New Orleans, Lousiana, Society of Mining Engineers, Inc., pp. 488.

87 Leblais, Y. and Bochon, A., 1991. Villejust tunnel: slurry shield effects on soil and lining behavior and comments on monitoring requirement. Tunnelling 91', 6th International Symp. organized by the Institution of Mining and Metallurgy, London, England, pp. 65-78.

88 Guillaume, J., 1989. The Vilejust Tunnels for the new very high speed train link to the Atlantic. International Congress on Progress and Innovation in Tunnelling, Toronto, Canada, pp. 679-685.

89 Wallis, S., 1986. TGV bores its way through France to the Atlantic coast. Tunnels and Tunnelling, June, 1986, pp. 19-22.

90 Hansmire, W.H., and Cording, E.J. 1972. Performance of a soft ground tunnel on the Washington metro. RETC 1972, Chicago, Illinois, AIMMPE, pp. 371.

91 Daugherty, C.W., Ware, K.R., and Gould, J.P. 1976. Selection of the vertical alignment of rapid transit tunnels. RETC 1976, Las Vegas, Nevada, AIMMPE, pp 311.

92 Heflin, L., and Higgins, M. 1993. Lessons learned: WMATA NATM design and construction. RETC 1993, Boston, Mass., pp. 341.

93 Heflin, L.H., Irshad, M. 1987. Soft ground NATM tunnel designs. RETC 1987, New Orleans, Luisiana, Society of Mining Engineers, pp. 111.

94 Mergelsberg, W. A., Leech, W.D., and Viner, R.L., Jr. 1987. Washington metropolitan area transit authority underground projects. RETC 1987, New Orleans, Louisiana, pp. 1112.

95 Mergentime, C.E., and Marshal, J.D. 1985. Construction of section E-ld Greenbelt route tunnels, Washington, DC. RETC 1985, New York, AIMMPE, pp. 631.

96 Rackstraw, N., and Meaza, M. 1991. Mixed face metro tunnels in free air. RETC 1991, Seattle, Wash., AIMMPE, pp. 529.

97 Kuesel, T.R. Washington Metro's topless tunnels. RETC 1976, Las Vegas, Nevada, AIMMPE, p. 296.

98 Ishii, T., Ishii, S.,and Kobayashi, M. 1994. Construction of large cross-sectional diversion tunnels through confined water-bearing sandy layers in urban areas: Katarbiragawa River Diversion Channel Project (Lot No. 2, Yokohama). International Congress on Tunnelling and Ground Conditions, Cairo, Balkema, April 1994, pp. 135.

99 Mizuno, E., and Hirano, I., and Katoka, K. 1994. Excavating tunnels of large cross-section with a thin overburden beneath densely populated areas (Konan Tunnel on Ring Road No. 2 in Yokohama City). International Congress on Tunnelling and Ground Conditions, Cairo, Balkema, April 1994, pp. 59.

100 Piquand, J.L., and Gesta, P., 1986. Les Tunnels de Villebon-Villejust du TGV Atlantique. International Congress on Large Underground Openings, Firenze, Italy, pp. 264-268.

101 STUVA, ITA, 1995. Subsurface Construction in Germany. Tagung '95, Stuttgart, May, pp. 53.

102 STUVA, ITA, 1995. Subsurface Construction in Germany. Tagung '95, Stuttgart, May, pp. 55.

103 STUVA, ITA, 1995. Subsurface Construction in Germany. Tagung '95, Stuttgart, May, pp. 59.

104 STUVA, ITA, 1995. Subsurface Construction in Germany. Tagung '95, Stuttgart, May, pp. 69.

105 STUVA, ITA, 1995. Subsurface Construction in Germany. Tagung '95, Stuttgart, May, pp. 71.

106 STUVA, ITA, 1995. Subsurface Construction in Germany. Tagung '95, Stuttgart, May, pp. 75.

107 STUVA, ITA, 1995. Subsurface Construction in Germany. Tagung '95, Stuttgart, May, pp. 197.

108 Oatman, M., Byers, M., and Larson, A. 1994. West Seattle Tunnel - Risk or Opportunity. Canadian Tunnelling 1994, publ. by The Tunnelling Association of Canada (TAC), Lo, K.Y., ed., pp. 183.

109 Tunnelling in Italy 1992, Pelizza, S., ed., Torino, pp. 94.

COST REFERENCES

Ball, J., 1995, Personal communication.

Christianson, J., 1995, Personal communication.

Edwards, C.A., and Merrill, K.D., 1995, Baltimore Metro northeast extension, in Williamson, G.E., and Gowring, I.M., eds., Proceedings, 1995 Rapid Excavation and Tunneling Conference, San Francisco, California, June 18-21, 1995: Littleton, Colo., Society for Mining, Metallurgy, and Exploration.

ENR, 1995, Cost history - inflation checked: ENR, March 27, p. 79-85.

Garrett, R., 1993, DART update: World Tunnelling, v. 6, no. 7, p. 288-293.

Kuesel, T.R., 1976, Washington Metro's topless tunnels, in Robbins, R.J., and Conlon, R.J., eds., Proceedings, 1987 Rapid Excavation and Tunneling Conference, Las Vegas, Nev., June 14-17, 1976: New York, N.Y., The American Institute of Mining, Metallurgical, and Petroleum Engineers, Inc., p. 296-310.

Kuesel, T.R., and Parker, H.W., 1981, Mixed face tunneling in Atlanta - MARTA's Broad Street tunnels, in Bullock, R.L., and Jacoby, H.J., eds., Proceedings, 1981 Rapid Excavation and Tunneling Conference, San Francisco, California, May 3-7, 1981: New York, N.Y., The American Institute of Mining, Metallurgical, and Petroleum Engineers, Inc., v. 1, p. 393-408.

Mergelsberg, W.A., 1995, Personal communication.

Mergelsberg, W.A., Leech, W.D., and Viner, Jr., R.L., 1987, Washington metropolitan area transit authority underground projects, in Jacobs, J.M., and Hendricks, R.S., eds., Proceedings, 1987 Rapid Excavation and Tunneling Conference, New Orleans, La., June 14-17, 1987: Littleton, Colo., Society for Mining, Metallurgy, and Exploration, v. 2, p. 1112-1133.

National Research Council, 1984, Geotechnical site investigations for underground projects, volume 2, abstracts of case histories and computer-based data management system: Washington, D.C., National Academy Press, 241 p.

Pozzi, R., 1995, Personal communication.

Robinson, R.A., Kucker, M.S., and Parker, H.W., 1991, Ground behavior in glacial soils for the Seattle transit tunnels, in Wightman, W.D., and McCarry, D.C., eds., Proceedings, 1991 Rapid Excavation and Tunneling Conference, Seattle, Wash., June 16-20, 1991: Littleton, Colo., The American Institute of Mining, Metallurgical, and Petroleum Engineers, Inc., p. 93-117.

Stirbys, A.F., 1995, Personal communication.

Tubs, M., 1993, 3.5-mile tunnel completed on DART's light rail transit line: Texas Construction, October 1993, p. 18-22.

Underground Space Center, 19959, Personal communication, "Minnesota Colloquium on Underground Space Use," University of Minnesota, July.

Wallis, S., 1994, Dallas city place: World Tunnelling, v. 7, no. 4,, p. N29-N33.

Table 2.1

Geologic Units Encountered by Existing or Proposed Tunnel Alignments (after Converse et al. 1981)

Formation	Map Symbol	Description
Young Alluvium	(Qal)	Silt, sand, gravel, and boulders; chiefly unconsolidated (loose) and granular.
Old Alluvium	(Qalo)	Clay, silt, sand, and gravel; chiefly consolidated (stiff) and fine-grained.
San Pedro Formation	(Sp)	Sand; clean, relatively cohesionless; locally impregnated with oil or tar (Formation not exposed at surface on geologic map).
Fernando Formation	(Tf)	Claystone, siltstone, sandstone; chiefly soft, stratified siltstone; local hard sandstone beds.
Puente Formation	(Tp)	Claystone, siltstone, sandstone; chiefly soft, stratified siltstone; local hard sandstone
Topanga Formation	(Tt)	Siltstone, sandstone, conglomerate; chiefly hard, well cemented, massive sandstone; local soft, thin siltstone beds; includes some Cretaceous conglomerate and sandstone, undifferentiated beds.
Topanga Formation	(Tb)	Basalt; includes dolerite and andesitic basalt; non-columnar flows and intrusives; deeply weathered, soft, crumbly at surface; hard, unweathered at depth.
Alluvial Fan	(Qf)	Silt, sand, gravel, and boulders; primarily semi-unconsolidated (dense) and granular.
Modelo Formation	(Tm)	Claystone, siltstone, sandstone; chiefly soft, diatomaceous stratified siltstone; local hard sandstone beds.
Granite	(Cg)	Chiefly granodiorites; deeply weathered, soft at surface; hard unweathered at depth.

Table 2.2

Tunnel Reaches in Santa Monica Mountains Tunnel

Reach	Approximate Station No.	Geologic Units (Rock Types) Within Tunnel Envelope
1	629+60 to 679+80	Plutonic Rock (predominantly granodiorite)
2	679+80 to 693+00	Chico Formation (conglomerate and sandstone) and Simi Conglomerate (conglomerate)
3	693+00 to 698+30	Las Virgenes Sandstone and Lower Topanga Formation (sandstone and conglomeratic sandstone)
4	698+30 to 716+10	Middle Topanga Formation (basalt and basalt breccia)
5	716+10 to 730+30	Lower section of Upper Topanga Formation (sandstone, partly conglomerate)
6	730+30 to 761+40	Upper section of Upper Topanga Formation (interbedded sandstone and siltstone/shale)

12-1-95/TABLE.2-2/W7101-lkd/dgw

Reach No. 1 - 5,020 Feet

GEOLOGIC CONDITIONS

General Lithology - Medium- to coarse-grained granitic rocks, blocky structure, infrequent inclusions (gneiss/schist), mildly foliated. Possible basalt dikes from few inches to several tens of feet thick and rare aplite or felsite dikes.

Discontinuities - Joint spacing variable. From 0.4 to 24 inches, dominantly 2.4 to 8 inches. Dip angles 20 to 50 degrees (common). Near Hollywood fault, inclination toward north and northeast. Mixed horizontal and vertical with random joint sets common at northern portion of reach. Joints generally tight hairline planar features.

Cementation - (igneous intrusive rocks) NA

Weathering - Completely weathered/decomposed near Hollywood fault zone (estimated 200-foot section) transitioning to moderately weathered in central reach to fresh in northern two-thirds of reach. Rock will be hydrothermally altered and brecciated in shear zones.

Percent Quartz - 19 to 33 percent of rock mass.

Groundwater Table - about 120 to 740 feet above tunnel crown and about 9 feet below ground surface. Groundwater barrier at Hollywood fault.

Other Geologic Conditions

- One or more major shear zones up to 200 feet wide (previously reported in Los Angeles Sewer Tunnel).
- Hollywood fault zone to be crossed at extreme south end of reach. Rock anticipated to be very weathered (locally decomposed) brecciated and sheared. Hollywood fault forms groundwater barrier with at least 186 feet of groundwater elevation difference across the fault.
- Minor sheared zones from 1 to 10 feet wide are common.

Reach No. 2 - 1,320 Feet

GEOLOGIC CONDITIONS

General Lithology - Conglomerate and interbedded sandstone lenses with minor (rare) thin claystone/siltstone beds. Large rounded gravel/cobbles to 8 inches and occasionally to 24 inches, matrix supported. Poorly to indistinctly bedded (massive). Simi Conglomerate contains up to 60 percent quartzite cobbles and boulders.

Discontinuities - Close joint spacing (2.4 to 8 inches) common, widely spaced random sheared zones with clay seams. Joint sets generally random and contain several intersecting sets. Bedding dips 10 to 70 degrees northeast (40 to 60 degrees dominant).
Conglomerate clasts are shattered and may fragment into smaller particles. Intact clasts should be anticipated.

Cementation - Slight to moderate (variable) should stand well at face. Cobbles and boulders usually poorly cemented to matrix and will dislodge easily.

Weathering - Fresh, no alternation.

Percent Quartz - Variable from 3 to 45 percent inclusive of sand grains and quartz rich rock fragments.

Groundwater Table - about 700 feet above tunnel crown and about 75 feet below ground surface.

Other Geologic Conditions - Interface between the granitic and conglomerate bedrock may represent a fault zone up to 15 feet-wide comprised of highly sheared and brecciated rock fragments derived from the conglomerate.

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Table 2.3** **Santa Monica Mountains Tunnel** **Anticipated Geologic Conditions** **Reaches No. 1 and No. 2**	November 1995 W-7101-01

Reach No. 3 - 530 feet

GEOLOGIC CONDITIONS

General Lithology - Dominantly thick beds of sandstone and conglomeratic sandstone, and (rare) conglomerate lenses to 3 feet thick with rounded clast to 18 inches and matrix supported. Estimated 80 percent sandstone, 10 percent gravelly sandstone, 5 percent conglomerate and 5 percent siltstone. This reach includes Las Virgenes, massive arkosic 125 feet thick, friable sandstone.

Discontinuities - Joints closely to widely spaced (2.4 inches to 6.6 inches) inclined from 30 to 60 degrees, generally healed with calcium carbonate. Bedding dips 30 to 60 degrees to the northeast.

Cementation - Sandstones are moderately well cemented (not friable). Conglomerate beds are very weakly cemented. Cementation is via calcite or clay and up to 30 percent by volume. The Las Virgenes sandstone is friable (weakly cemented).

Weathering - Generally fresh with approximately 4 percent chlorite bearing (hydrothermal alternation).

Percent Quartz - 28 to 47 percent (mostly sand sized grains).

Groundwater Table - about 560 feet above tunnel crown and abut 165 feet below ground surface.

Other Geologic Conditions - Geologic contacts are judged to be conformable at each end of reach.

Reach No. 4 - 1,780 feet

GEOLOGIC CONDITIONS

General Lithology - Extruded basalt, dominantly breccia with massive intervals of basalt flows. Breccias are coherent, matrix supported; clasts are angular to several inches across. Matrix consists of chlorite, zeolite, and smectite minerals. Infrequent depositional lenses and layers of sandstone up to 50 feet thick, fine to medium grained.

Discontinuities - Joints and shears often lined with chlorite/smectite are very closely to moderately closely spaced (0.4 to 24 inches), and predominantly interlocking and wavy. Generally two sets at moderate to steep inclination with one random set superimposed. Inclinations range from 24 to 60 degrees (44 degrees average). Trends E-W, NW, ENE, shears commonly are near vertical. Most joints and shears are healed with infilling of calcite, zeolite, chlorite, minerals, or smectite.

Cementation - Igneous rock (basalt) Not Applicable. Breccia are not granular, matrix is softer than fragments, generally coherent. Sandstone lenses may be well cemented.

Weathering - Fresh (unweathered) but much of original basalt is hydrothermally altered to serpentine and chlorite group minerals.

Percent Quartz - No quartz present but rock contains an abundance of serpentine and chlorite group minerals on fracture surfaces.

Groundwater Table - About 650 feet above tunnel crown and about 100 feet below ground surface

Other Geologic Condition - Low percentage of iron pyrite disseminated in rock mass or locally concentrated on some joint surfaces. Geologic contacts are judged to be conformable on each end of reach.

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Table 2.4** **Santa Monica Mountains Tunnel** **Anticipated Geologic Conditions** **Reaches No. 3 and No. 4**	November 1995 W-7101-01

Reach No. 5 - 1,420 feet

GEOLOGIC CONDITIONS
General Lithology - Dominantly a massive to thick bedded medium to coarse grained sandstone with widely spaced thin to thick gravelly sandstone zones. Sequence contain 80 percent sandstone, 15 percent conglomeratic sandstone, and 5 percent conglomerate. Clasts up to 24 inches (rare to 48 inches), subangular to subrounded, matrix supported. Minor thin (1 to 2 inches thick) siltstone rare.
Discontinuities - Joints closely to widely spaced (2.4 inches to 6.6 inches) and primarily moderately closely spaced (8 to 24 inches). No regular pattern of orientation or dip angle (random). Infrequent sheared clay seams.
Cementation - Moderately cemented with primarily calcite.
Weathering - Fresh (unweathered), no alteration
Percent Quartz - Quartz content of sand grains varies from 15 to 30 percent with intervals up to 55 percent quartz.
Groundwater Table - About 550 feet above tunnel crown and about 50 feet below ground surface.
Other Geologic Conditions - Geologic contacts are judged to be conformable at each end of reach.

Reach No. 6 - 3,110 feet

GEOLOGIC CONDITIONS
General Lithology - Interbedded sandstone and siltstone/shale. Laminated to thickly bedded (very distinct). Dominantly fine to coarse sandstone in the south portion and increase in siltstone/shale content towards the north portion. Bedrock is folded locally, but bedding predominantly dips northeast.
Discontinuities - Joint spacing moderately close (8 inches to 24 inches), usually one set with apparent random orientation. Bedding dips generally 50 to 90 degrees, reversals and possible overturning anticipated. Bedding parts easily on some siltstone/shale surfaces, often sheared, polished clay-lined seams present.
Cementation - Variable, ranging from slightly to moderately well cemented. Some sandstone layers are uncemented and very friable.
Weathering - Mostly fresh (unweathered). Locally highly weathered to residual soil.
Percent Quartz - Quartz content of sand grains varies from 15 to 30 percent with intervals up to 55 percent quartz.
Groundwater Table - About 50 to 200 feet above tunnel crown and 0 to 30 feet below ground surface
Other Geologic Conditions - The Benedict Canyon fault zone will be crossed at the north end of the reach. Two zones of shearing/brecciation are anticipated beneath the Hollywood Freeway area. No more than 9 feet of bedrock overlies tunnel crown beneath Hollywood Freeway. Stream alluvium may be encountered in crown of tunnel beneath freeway. Geologic contact at the south end of the reach is judged to be conformable.

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Table 2.5** **Santa Monica Mountains Tunnel** **Anticipated Geologic Conditions** **Reaches No. 5 and No. 6**	November 1995 W-7101-01

Table 4.1

Review of Urban Tunnels in Soft Ground

No	Project	Country	Year	Type of Ground	Depth of Water (ft)	Depth of Tunnel (ft)	Diameter (ft)	Tunneling Method	Lining - Initial/Final	Ground Improvement	Reference No [1]
1	Antwerp - Metro Line 2	Belgium	1977-1981	fine sand		30-59ft	21ft	SPB TBM	concrete segments		1, 2, 3, 4,
2	Atlanta (MARTA)	USA	1977	saturated silty sand, sandy silt, weathered rock		33-92ft		APB TBM		grouting to control water percolation	5
3	Baltimore (MTA)	USA	1973-1992	sandy clay, sand , gravel, near-surface water		66ft	20ft	OF TBM (backhoe)		chemical grouting	6
4	Baltimore (MTA)	USA	1973-1992	sandy clay, sand , gravel, near-surface water		66ft	20ft	OF TBM (digger shield)	concrete segm., water-proof membrane, cast-in-place concrete	compaction grouting from the surface, chemical grouting	7, 8,
5	Berlin - city metro	Germany	1986	large boulders, water-bearing sands and gravels		20ft	22ft	SPB TBM			9
6	Boston (MBTA) - Red Line Extension	USA	1978	silty clay, outwash sand and gravel, glacial till - very dense silty to coarse		39ft	23ft	OF TBM	rib and lagging, cast-in-place concrete		10
7	Boston - Wellesley Extension Sewer	USA	1989	saturated running sands		23ft	6ft	SPB TBM	jacked reinforced concrete pipe		11
8	Cairo Metro	Egypt	1993	wet sands, cobbles, occasional		72ft	31ft	SPB TBM			12, 13
9	Cairo waste water project	Egypt	1980's, 1990's	dense sand	7ft	39ft	17ft	APB TBM			14
10	Cairo waste water project	Egypt	1980's, 1990's	fine to medium sand	7ft	52ft	11ft	EPB TBM			14
11	Cairo waste water project	Egypt	1980's, 1990's	compact to dense sand, soft to very stiff clay, cobbles 200mm, occasional boulders 1m	3ft	49ft	20ft	SPB TBM	concrete segments		14
12	Caracas Metro	Venezuela	1986	loamy sand, sandy loam, gravelly sand, under ground water		16-33ft	19ft	EPB TBM (Lovat)	concrete segments		15
13	Caracas Metro (Propatria-Fuerzas Armadas)	Venezuela		clays, sands, silts, schists (decomposed)		10-33ft	19ft	CF TBM	concrete segments	compressed air, chemical grouting	16
14	Cologne's Urban Railway	Germany	1992	quaternary sands, clay-sands	30ft	30ft	19ft	SPB TBM	concrete segments		17
15	Creteil-Valenton Tunnel, north-west of Paris - wastewater diversion	France		coarse alluvial deposits, dense sandy gravel, under water		33ft	11ft	SPB TBM	concrete segments + cast-in-place concrete		18
16	Edmonton Experimental Sewage tunnel	Canada	1979	softer glacial till, above water level		8ft	74ft	CF TBM (Lovat)	concrete segments		19, 20
17	Edmonton LRT Subway	Canada	1981	stiff glacial till, above water level		30ft	20ft	CF TBM (Lovat)	steel ribs and wooden lagging, cast-in-place concrete		21, 22
18	Edmonton Subway	Canada	1989	mixed face: sand, gravel, above water level		49ft	22ft	SPB TBM	concrete segments		23, 24, 25
19	Essen subway (Baulos 34)	Germany		marl, silts	13ft	49ft	27ft	SPB/EPB TBM	concrete segments		27, 102
20	Gelsenkirchen Railway	Germany	1979	mixed face: sand, silty sand and chalk marl, under ground water		>11ft	24ft	SPB TBM	steel segments		28, 29
21	Grauholtz		1991	variable: silty clay, gravel, boulders, below ground water			38ft	SPB TBM	steel segments	fault grouting	30, 31, 29
22	Hamburg sewage tunnel	Germany	1978	variable: sand or clay with silt, below ground water		13ft	14ft	SPB TBM			32
23	Japan, sewage	Japan		alluvium gravel and clay, under ground water		20ft	11ft	EPB TBM	concrete segments		34
24	Kobe - Maiko Twin Highway Tunnel	Japan	1988-1998	Pleistocene sediment, sand, gravel, layers of clay	33ft	33-180ft	30h x 39w (ft)	NATM	forepoling, ribs + cast-in-place concrete	face bolts, chemical grouting	33
25	Lile Metro	France		Tertiary rocks, chalk, limestone, sand-bearing Cretaceous rock, clay, sand		13ft	25ft	EPB TBM	concrete segments		35

Table 4.1

Review of Urban Tunnels in Soft Ground

No	Project	Country	Year	Type of Ground	Depth of Water (ft)	Depth of Tunnel (ft)	Diameter (ft)	Tunneling Method	Lining - Initial/Final	Ground Improvement	Reference No[1]
26	Lile Metro	France		fine grained soil, sandy, clayey water-bearing gravels			25ft	SPB TBM			35
27	Lile Metro	France	1994	Sedimente de Marque with coarse gravel			25ft	EPB TBM			35
28	Lisbon Metro	Portugal		alluvial clay, sand, gravel		66ft	32ft	EPB TBM	concrete segments		36
29	London - test tunnel	England	1972	gravel, below ground water		26ft	13ft	SPB TBM	steel segments		4
30	Lyon Metro Line D	France	1984-1986	gravel under river			21ft	SPB TBM	extruded concrete with steel fibres		37, 38, 39, 40, 41, 42,
31	Lyon traffic tunnels under Saome	France	1994	hard gneiss, soft alluvial mat.				EPB TBM	concrete segments		44
32	Mexico City Sewage	Mexico	1984-1986	highly compressible clay, under ground water		32ft	13ft	SPB TBM	concrete segments	grouting	45
33	Milan - Passante Ferroviario (railway twin tube)	Italy		alluvial ground, above water table		26ft	26ft	EPB TBM	extruded concrete behind segmented lining		52
34	Munich Underground	Germany	1994	Tertiary waterbearing sands, gravels			24ft	APB TBM			46
35	Nagoya Municipal Subway	Japan		loose water-bearing gravelly soil containing boulders		43ft	24ft	EPB TBM	concrete segments		47
36	Oakland subway, California	USA		recent soft marine silty clay (Bay Mud)		52ft	19ft	APB TBM	steel segments		48
37	Osaka Stormwater Tunnel 1 and 2	Japan	1985	hard diluvium, sand, gravel		72ft	37ft	SPB TBM	concrete segments		49, 50
38	Osterode - Butterberg tunn. - 50 km southeast from Hannover		1977-1979	Quaternary terrace deposit (Pleistocene), well graded sandy-silty gravel, boulders up to 1m, no water		46ft	38ft	NATM	shotcrete 30cm, wire mesh, ribs prim.	1.3 m forepoling	51
39	Paris - East-West Crossover Line (EOLE)	France	1992-1998	marl, gravel, coarse limestone, clayey sand		98ft	39ft	Over-cutting method	shotcrete + cast-in-place concrete	face supported by falsework + shotcrete, fiberglass tubes	26
40	Paris - East-West Crossover Line (EOLE)	Paris	1992-1998	sand, clayey ground, limestone		98ft	24ft	SPB/EPB TBM			26
41	Rome - Galleria Aurelia -railway	Italy	1989	hard clay, silty sand, sand with gravel, silty sand, below ground water	43ft	98ft	35ft	SPB TBM	concrete segments		2, 53, 54, 55, 56, 57,
42	San Francisco (BART)	USA		alluvial cohesive deposits, water close to the surface		39ft	19ft	OF TBM	steel welded pan segments stiffened with ribs		58
43	San Francisco (MUNI) Metro Turnback	USA	1994	mixed conditions of fill, bay mud	23ft	33ft	19ft	OF TBM	steel welded pan segments stiffened with ribs	chemical grouting	59
44	San Francisco, water-way	USA	1981	soft sandy clay (recent Bay Mud), under ground water		30ft	12ft	EPB TBM	steel segments		60, 61, 62, 63, 64, 65
45	Sao Paulo - Alto da Boa Vista Tunnel	Brazil	1978	clayey sand (stiff - dense), above		20ft	13ft	NATM	shotcrete 10cm+3cm		66, 67, 68,
46	Sao Paulo - subway under Boa Vista street and Caixa Economica	Brazil	1973	sands, silty clays, below ground water, perched ground water			20ft	CF TBM	segment iron rings	vacuum wells, underpinning, grouting, chemical grouting from surface	70
47	Sao Paulo - subway north of Prestes Maia shaft	Brazil	1973	sands, silty clays, below ground water, perched ground water			20ft	OF TBM (manual excavation)	segment iron rings	vacuum wells, underpinning, grouting, chemical grouting from surface	70
48	Seattle Downtown Transit	USA	1987	glacial soils, flowing sand	39ft	59ft	21ft	OF TBM (digger shield)	water proof membrane (PVC), concrete segments	compaction grouting, jet grouting, eductor well, deep wells, vacuum well points, chemical grouting, underpinning	71, 72
49	Seattle -West Tunnel - wastewater tunnel	USA	1994	squeezing-fast raveling, cohesive flowing glacial consolidated soils, grounewater head(60ft) above the tunnel, methane presence, boulders		300ft	10ft	EPB TBM	bolted and gasketed concrete segments	compaction grouting, jet grouting, eductor well, deep wells, vacuum well points, chemical grouting, underpinning	108
50	Shangai, sewage	China		silty clay, silty sand, under ground		13ft	14ft	EPB TBM	concrete segments		73

Table 4.1

Review of Urban Tunnels in Soft Ground

No	Project	Country	Year	Type of Ground	Depth of Water (ft)	Depth of Tunnel (ft)	Diameter (ft)	Tunneling Method	Lining - Initial/Final	Ground Improvement	Reference No [1]
51	Shanghai Subway	China		soft clay and permeable sand, under ground water	2ft		18ft	APB TBM	concrete segments		74
52	Shikawa - Tokyo Railway	Japan		non-cohesive well compacted gravel, under ground water		72ft	27ft	SPB TBM	concrete segments		75
53	Singapore, subway	Singapore		soft clay, permeable sand			18ft	EPB TBM	concrete segments		76
54	Taipei Metro - Taiwan	Taiwan	1992	alluvial clay, silty sand	7ft	66ft	20ft	EPB TBM	concrete segments		77
55	Tokyo - Idabashi Subway Station	Japan	1995	alluvial sand, gravel		89ft	29ft	SPB TBM	concrete segments		33
56	Tokyo - railway	Japan	1980	gravel and sand, below ground water			33ft	SPB TBM			4
57	Tokyo Highway, Kawasaki-Kisarazu	Japan	1994	alluvial clay, sand	7ft	131ft	46ft	SPB TBM	concrete segments		78
58	Tokyo Sewer System, Ohta Trunk	Japan		silt, under ground water		52ft	27ft	EPB TBM	concrete segments		79
59	Tokyo Sewer System, Omori Trunk	Japan	1987-1988	sandy silt, fine sand, under ground		52ft	12ft	EPB TBM	concrete segments		79
60	Tokyo Sewer System, Shin-Ohmori	Japan		silt and sand, under ground water		52ft	19ft	EPB TBM	concrete segments		79
61	Tokyo, Keiyo, subway	Japan	1990	sandy clay, gravel, under ground water		89ft	24ft	EPB TBM	segment concrete		80, 81
62	Tokyo, Shinozaki trunk sewer	Japan		sandy soil, under ground water table		23ft	13ft	SPB TBM		dewatering, grouting	82
63	Tokyo, Shinozaki, sewer	Japan		sandy soil, under ground water		34ft	17ft	EPB TBM			82
64	Toronto - sewer tunnel	Canada	1972	very dense sand-clay till	20ft	38ft	14ft	OF TBM	ribs and concrete planks + cast-in-place concrete		83, 84
65	Toronto - sewer tunnel	Canada	1966	fine medium sands				OF TBM (hand mining)	ribs and concrete planks + cast-in-place concrete		85
66	Vienna Subway	Austria	1983	alluvial		5ft	33ft	NATM		dewatering, ground freezing, grouting	86
67	Villejust - railway tunnel	France	1984-1988	fine dense sand, at and above ground water		24ft	30ft	SPB TBM	concrete segments		2, 100, 87, 88, 89
68	Washington D.C. (WMATA) - A2 test section	USA		Pleistocene sand , gravel, silty sand, perched ground water (8ft head)	33ft	49ft	21ft	OF TBM	expanded steel ribs	deep dewatering wells, chemical grouting in running terrace deposits	90, 91
69	Washington D.C. (WMATA) - Sections E-5, 6e, 8e	USA	1987	gravel, sand, occasional boulders, clay	33ft	20ft	17h x 16w (ft)	NATM	shotcrete + cast-in-place concrete		92, 93
70	Washington D.C. (WMATA) - Anacostia River Crossing	USA	1987	Cretaceous stiff clays, dense clayey sands		33ft	21ft	EPB TBM (Hitachi)			94
71	Washington D.C. (WMATA) - Greenbelt Route	USA	1987	Cretaceous and Pleistocene deposits		39ft	19ft	CF TBM (Lovat)	concrete segments	chemical grouting	94, 95, 96
72	Washington D.C. (WMATA), Branch Rte. and Pentagon Rte. Tunnels	USA		mixed ground conditions, clay invert and waterlogged sand crown				OF TBM (Robbins)	concrete segments	chemical grouting	97
73	Yokohama - River Diversion Channel	Japan	1991	aquifer alluvial loose sand		98ft	364 ft^2	NATM	shotcrete + cast-in-place concrete	chemical grouting, drainage boring	98
74	Yokohama City - Konan Tunnel	Japan		diluvial sand and silt		16-79ft	1540 ft^2	NATM	forepoling, shotcrete + cast-in-place concrete	jet grouting in soft clay, vertical bolting from the surface	99

[1] References for the case histories are provided after the main text of the appendix.

Table 4.2

Summary of Urban Tunnels in Soft Ground

Tunneling Method and Ground Conditioning	Total Number of Cases	Type of Ground			U.S. Practice
		Granular	Fine-Grained	Mixed	
NATM all	7	5	0	2	1
NATM with GC	5	3	0	2	0
NATM without GC	2	2	0	0	1
OF TBM all	12	2	1	9	8
OF TBM with GC	8	1	0	7	6
OF TBM without GC	4	1	1	2	2
CF TBM all	4	0	2	2	1
EPB TBM all	20	6	1	13	3
SPB TBM all	25	13	2	10	1
APB TBM all	5	3	1	1	2
Others	1	0	0	1	0
TBM all types	66	24	7	35	15
TOTALS:	74	29	7	38	16

Legend:

TBM	Tunnel Boring Machine
OF TBM	Open Face TBM
CF TBM	Closed Face TBM
APB TBM	Air Pressure Balance TBM
SPB TBM	Slurry Pressure Balance TBM
EPB TBM	Earth Pressure Balance TBM

12-1-95/TABLE.4-2/W7101-lkd/dgw

Table 6.1

Summary of Tunneling Contracts for Red Line Project

Contract	Description	Route Length (feet)	Contractor	Bid Date	Low Bid (million)	2nd Low Bid (million)	Left on Table (million)	Change Orders To Date (million)	Final Cost or Current Value (million)	Cost per Route Foot (Twin Tubes) (dollars)	Geology	Single Tube Rate of Advance (Max) (ft/day)	Single Tube Rate of Advance (Average) (ft/day)
A130	East Portal to Union Station	1692	Tutor/Saliba/Perini	5/20/88	$37.7	$41.2	$3.6	($0.3)	$37.4	n/a [1]	Alluvium		10-30
A141	Union Station to Pershing Square	5871	Tutor/Saliba/Groves	1/14/87	$61.5	$66.1	$4.7	$27.7	$89.2	n/a [2]	Puente/ Alluvium	70	
A146	Pershing Square to 7th/Metro Center	2150	Shank/Ohbayashi	2/12/87	$18.2	$18.3	$0.1	$6.7	$25.0	$11,614	Alluvium	55	40
A171	7th/Metro Center to Westlake/MacArthur Park	5015	Shank/Ohbayashi	12/12/86	$26.3	$28.7	$2.4	$3.3	$29.7	$5,916	Puente	135	65-100
B201	Westlake/MacArthur Park to Wilshire/Vermont	4536	Tutor/Saliba/Perini	3/28/91	$44.6	$44.8	$0.2	$6.1	$50.7	n/a [1]	Puente	200	
B221	Wilshire/Vermont to Wilshire/Western	4894	Tutor/Saliba/Perini	3/1/91	$79.8	$81.8	$2.0	$12.5	$92.3	n/a [1,2]	Puente/ Alluvium	180	80-90
B251	Vermont/Hollywood	31205	Shea/Kiewit/Kenny	5/28/92	$129.5	$141.7	$12.2	$43.9	$173.4 [3]	$5,556	Puente/ Alluvium	320	80-110
C311	Santa Monica	13180	Traylor Bros./Frontier Kemper	10/18/94	$124.4	$124.7	$0.3		$124.4	$9,440			
C331	North Hollywood	10515	Ohbayashi	10/16/93	$65.4	$75.5	$10.1	$1.3 [4]	$66.7	$6,343	Alluvium		

[1] contract includes cut and cover section
[2] contract includes station
[3] includes additional length added to contract
[4] does not include all of $4.7 million grouting budget

Table 6.2

Soft Ground Tunneling Costs for Selected North American Transit Projects

Location	Project	Contractor	Start	Bid (million)	Total (million)	Route Feet (feet)	Tunnel Size	Excavation Method	1995 Cost/Route Foot [1]
Atlanta, GA	MARTA - Broad Street		1977	$12.0	na	959	20' 6"	shield	$25,145
Baltimore, MD	Baltimore - Bolton Hills	Fruin-Colnon Corporation	Aug-77	$29.1	na	5615	19' 1"	compressed air, drill & blast	$10,353
Baltimore, MD	Baltimore - Lexington Market	Traylor, MK & Grow (JV)	Sep-78	$11.6	$11.8	1560	19' 1"	compressed air	$13,667
Baltimore, MD	Baltimore - John Hopkins	Kiewit/Shea (JV)	Jul-89	$70.4	$106.0	7900		shield w/ compressed air	$15,849 [2]
Los Angeles, CA	LACMTA A146	Shank/ Ohbayashi	Feb-87	$18.2	$25.0	2150	22' 2"	shield	$14,358
Los Angeles, CA	LACMTA A171	Shank/ Ohbayashi	Dec-86	$26.3	$29.7	5015	22' 2"	shield	$7,311
Los Angeles, CA	LACMTA B251	Shea/Kiewit/ Kenny	May-92	$163.5	$173.4	31205	21' 10"	shield	$6,096
Los Angeles, CA	LACMTA C331	Ohbayashi	Oct-93	$65.4	$66.7	10515	21' 11"	shield	$6,526
New York, NY	NYCTA 131-D, Section 5	MacLean-Grove (JV)	Nov-75	$4.4	$4.4	164	19' 8"	shield	$64,376
New York, NY	NYCTA 131-D, Section 8	Schiavone	Sep-81	$22.6	na	1205	19' 5"	shield	$26,993
New York, NY	NYCTA 133, Section 2	Schiavone	Mar-77	$21.1	$21.0	1110	19' 5"	shield	$39,081
Seattle, WA	Seattle Downtown Transit	Guy F. Atkinson	Nov-86	$44.1	na	5000	21' 3"	shield	$10,899 [2]
Washington, D.C.	WMATA Section E-1b/c	Mergentime/ Perini	Aug-85	$50.9		1895		CF TBM	$34,127 [3]
Washington, D.C.	WMATA Section E-1d	Mergentime/ Loram	Jun-83	$25.8		1770		CF TBM	$18,921 [4]
	estimated tunnel cost								$7,400 [5]
Washington, D.C.	WMATA Section F-1b	Dravo	Mar-74	$9.9	$10.8	2653	20.7'	shield	$10,464
Washington, D.C.	WMATA Section F-3a	Harrison Western	Nov-85	$24.9		1135		EPBM	$27,784 [4]
	estimated tunnel cost								$10,200 [5]
Washington, D.C.	WMATA Section F-3c	Mergentime/ MK	Dec-86	$19.5		1695		EPBM	$14,217 [4]
	estimated tunnel cost								$10,200 [5]
Washington, D.C.	WMATA Section F-4a	Harrison Western/ Franki-Denys	Dec-84	$25.6		2540		EPBM	$12,965
	estimated tunnel cost								$9,800 [5]
Washington, D.C.	WMATA Section G-2	Healy-Ball-Greenfield (JV)	Oct-75	$18.2	$23.0	6850	20' 11"	shield	$7,989
Washington, D.C.	WMATA - Pentagon and Branch Tunnels	Traylor Bros./ S&M	Aug-74	$35.7		4400	18'	TBM	$20,893 [6]
Washington, D.C.	WMATA 1987		1987						$8,500 [5]
Washington, D.C.	WMATA 1989		1989						$8,200 [5]
Washington, D.C.	WMATA 1994A		1994						$7,000 [5]
Washington, D.C.	WMATA 1994B		1994						$11,000 [5]
Washington, D.C.	WMATA 1994C		1994						$11,600 [5]

[1] converted from bid date cost to 1995 cost using ENR Cost History
[2] includes excavation for station
[3] includes station
[4] includes cut and cover section
[5] estimated cost for tunnel only (stations, cut and cover, shafts, etc. removed))
[6] includes protection of existing structures

Note: References for the sources of cost data are provided after the main text of the appendix.

Table 6.3

Rock Tunneling Costs for Selected North American Transit Projects

Location	Project	Contractor	Start	Bid (million)	Total (million)	Route Feet (feet)	Tunnel Size	Excavation Method	1995 Cost/Route Foot [1]
Baltimore, MD	Laurens Street Tunnels	Granite Construction Co.	Mar-79	$13.6	$14.8	2603	22' x 19'	heading & bench, hand mining, drill & blast	$10,098
Baltimore, MD	Baltimore - Mondawin Line South	Clevecon, Inc.	Jul-78	$11.9	$12.0	3300	18' x 16'	drill & blast	$6,667
Baltimore, MD	Baltimore - Mondawin Tunnels	Clevecon, Inc.	Nov-77	$10.3	$10.3	3158	18' x 16'	drill & blast	$6,340
Boston, MA	MBTA Red Line - Porter to Davis Square	Perini	Apr-80	$14.2	$13.6	2550	23' 6"	drill & blast	$8,716
Boston, MA	MBTA Red Line - Porter to Harvard Square	MK, White & Mergentime (JV)	Sep-79	$25.0	na	4325	23' 6"	shield, drill & blast	$9,458
Buffalo, NY	Buffalo Section C-11	Fruin-Colnon, Traylor, Onyx (JV)	Mar-80	$28.7	na	10208	18' 6"	TBM	$4,548
Buffalo, NY	Buffalo Section C-31	S&M, McHugh, Kenny (JV)	Jan-80	$17.7	na	7449	18' 6"	TBM	$3,900
Dallas, TX	DART - City Place	S.A. Healy	May-92	$70.0	na	16750		TBM	$4,585
Los Angeles, CA	LACMTA C311	Traylor Bros./ Frontier Kemper	Oct-94	$124.4	$124.4	13180		TBM	$9,401
Portland, OR	Portland Westside Light Rail	Frontier Kemper/ Traylor Bros.	May-93	$96.8	$120.5	14411		TBM, drill & blast	$8,449
Washington, D.C.	WMATA Section A-9a	MK	Sep-75	$25.0	$27.2	7620	19' 1"	TBM, drill & blast	$8,470
Washington, D.C.	WMATA Section A-11a	J.F. Shea	Mar-77	$23.3	$22.0	11464	19' 1"	TBM	$3,957
Washington, D.C.	WMATA Section B-10a	Ilbau America	Sep-83	$51.5	na	8800		drill & blast	$7,475
Washington, D.C.	WMATA Section B-10c	Dillingham/ Ohbayashi	Jan-87	$19.5	na	2200		drill & blast	$10,963
Washington, D.C.	WMATA 1993		1993						$6,600 [2]
Washington, D.C.	WMATA Section C-4	MK	Nov-72	$15.6	$25.0	6056	20'	shield, drill & blast	$12,207

[1] converted from bid date cost to 1995 cost using ENR Cost History

[2] estimated cost for tunnel only (stations, cut and cover, shafts, etc. removed))

Note: References for the sources of cost data are provided after the main text of the appendix.

Table 7.1

Selected Previous Soft Ground Tunneling Projects in Los Angeles

(1) Metropolitan Water District San Fernando Tunnel -- Water Supply

Date		1970-1975
Diameter/Length		22 ft. O.D./29,100 ft.
Geology		Soft sandstone & siltstone
Excavation Method		Robbins Digger Shield
Initial Support		Precast concrete segments
Contractor		Lockheed Shipbuilding & Construction Company
Remarks	-	Record progress (up to 277 ft/24 hr day) in good ground
	-	Substantial water inflow (1,400 gpm) slowed rate of advance to 60 ft/24 hr
	-	Caving resulted from thousands of gallons of water in Old Alluvium
	-	Sinkhole (10 ft diameter) on Foothill Blvd.
	-	Survived 6.4 Richter earthquake 2/9/71 without damage (earthquake tilted tunnel; one portal uplifted 7 ft higher but no evidence of damage to initial lining)
	-	Fatal gas explosion 6/24/71 killed 17 workers and stopped project for 27 months
	-	Restart and completed without incident with substantially more ventilation and advance rates restricted to no more than 25 ft/day; TBM removed

(2) MWD Tonner Tunnels #1 & 2 -- Water Supply

Date		1972-1976
Diameter/Length		11 ft O.D./4.3 miles (23,000 ft)
Geology		Sandstone & shale (Puente formation)
Excavation method		Calweld rotary head CF TBM
Initial Support		Steel ribs & wood lagging
Contractor		JF Shea Co. Inc. -- $15,348,331
Remarks	-	Gassy tunnel, 1st tunnel built after fatal gas explosion in San Fernando tunnel
	-	Most of tunnel in low strength of 100-200 psi
	-	Boulders required modification of TBM
	-	Encountered 1,500 ft. of very hard cemeted sandstone with unconfined strengths of 12,000-15,000 psi so TBM was abandoned for drill & blast
	-	Oil seeps but not significant acculations were common in some formations
	-	One automatic shutdown of TBM due to methane

(3) MWD Newhall Tunnel -- Water Supply

Date	1966-1970
Diameter/length	26 ft O.D./3.5 miles (18,480 ft)
Excavation method	Calweld OF TBM
Initial Support	Steel ribs and wood lagging plus precast concrete segments
Contractor	Dixon Arundel MacDonald & Kruse; Peter Kiewit
Remarks -	Contractor abandoned one TBM due to weak sedimentary rock sloughed ahead, above and onto the machine, replacing it with another TBM pushing off the initial steel ribs and wood lagging

(4) LACFCD Sacatella Tunnel

Date	1975-1977
Diameter/length	18 ft O.D./0.6 miles (3,170 ft)
Geology	Claystone, siltstone, occasional very hard calcareous cemented sandstone (Puente Formation)
Excavation method	Digger Shield
Initial support	Precast concrete segments
Contractor	Glansville Construction Co.
Remarks -	Gassy tunnel but ventilation kept alarm from going off
-	Encountered several uncharted, uncased, abandoned oil wells full of water
-	Oil from formation seeped down sides of supports
-	Dewatering by deep wells worked satisfactorily
-	No reported overbreak; standup time was 2-3 hours
-	Encountered several hard cemented layers making excavation difficult. Some air slaking noted.
-	Ground settlement apparently not measured but no settlement noted and no noise complaints from residents except at portals
-	Encountered 21-ft diameter auger hole abandoned which chimneyed up to within 6 ft of surface but did not daylight to surface was filled with pea gravel from the surface

(5) North Outfall Replacement Sewer Tunnel (NORS) Digger Shield Portions

Date	1989
Diameter/length	18 ft O.D./8 miles (42,782 ft)
Geology	San Pedro & Lakewood Formations of dense sands, silts & clays. Under LAX airport, lightly cemented dune sand then uncemented recent dune sand; gassy tunnel, crosses active Newport-Inglewood fault and potentially active Overland Ave and Charnock faults
Excavation method	Digger Shield
Initial Support	Precast concrete segments
Contractor	JF Shea/Traylor Bros. JV $115,240,050
Remarks	- Gassy tunnel with full ventilation & automatic methane detection - Magnetometer probe ahead of tunnel was specified to detect any unknown or uncharted oil wells - No oil seeps; some methane encountered - Sinkhole on LAX property (20' diameter & 16' deep). Subsequently encountered 7 smaller sinkholes. Undertook comprehensive program of exploration & remediation with hundreds of borings & grout holes. Second round of exploration/ remediation after new sinkhole developed 1 year later.

(6) North Outfall Replacement Sewer Tunnel (NORS) EPBM Stretch

Date	1990-1991
Diameter/length	12 ft O.D./1,900 ft
Geology	Claystone, siltstone & occasional very hard calcareous cemented sandstone (Puente Formation)
Excavation Method	Earth Pressure Balance Machine
Initial Support	Steel liner plates; some reinforced with steel ribs
Contractor	JF Shea/Traylor Bros. JV
Remarks	- Gassy tunnel with full ventilation and automatic methane detection - Magnetometer probe ahead of tunnel was specified to detect any unknown or uncharted oil wells - EPBM had considerable difficulty operating in pseudo-earth pressure balance mode. Difficulty with steel supports. - Sinkhole developed to surface during pseudo-earth pressure balance operation

Table 7.2

Selected Previous Rock Tunneling Projects in Los Angeles

(1) LAC (La Cienega-San Fernando Valley) Sewer Tunnel

Date	1954-1956
Diameter/length	9 ft O.D./2.8 miles (41,400 ft)
Geology	Conglomerate sandstone, shale (Topanga Formation) 8,000 ft, granite (4,000 ft), basalt (1,200 ft), Young Alluvium (430 ft)
Excavation Method	Drill & blast
Initial Support	Steel ribs & lagging; steel liner plates for soft ground
Contractor	LE Dixon Co.; lining by Kemper Construction Co.
Remarks -	Substantial groundwater inflow at several locations (greater than 100 gpm at 7 locations)
-	Maximum water inflow for entire tunnel 800 gpm
-	Heavy ground pressure in Topanga Formation shale
-	Sinkhole to surface in wet, muddy Young Alluvium

(2) MWD Hollywood Tunnel Water Supply\

Date	1940-1941
Diameter/length	8 ft O.D./0.7 miles (854 ft)
Geology	Conglomerate sandstone, shale (Topanga Formation) (1,600 ft), basalt (2,100 ft)
Excavation Method	Drill & blast
Initial Support	Steel ribs & lagging
Contractor	JF Shea
Remarks -	Located at 400 ft above C-311
-	Water inflow of 600 gpm at sandstone/ basalt contact

Table 8.1

Performance of Urban Tunnels in Soft Ground

No	Project	Country	Year	Settlement (in)	Reported Failure	Tunneling Method	Lining - Initial/Final	Rate of Advance (ft/day)	Special features	Reference No [1]
1	Antwerp - Metro Line 2	Belgium	1977-1981			SPB TBM	concrete segments	9.2ft/day		1, 2, 3, 4,
2	Atlanta (MARTA)	USA	1977	2.0in		APB TBM				5
4	Baltimore (MTA)	USA	1973-1992		face runs, street collapse, compressed air leakage through unbolted ungasketed initial liners (two-pass lining system), gasoline ground contamination	OF TBM (digger shield)	concrete segm., water-proof membrane, cast-in-place concrete		polling plates replaced by breasting plates, explosion-proof equipment, vapor monitoring at the surface and in the tunnel, big digger replaced by road-header, vapor extraction wells, compressed air	7, 8,
7	Boston - Wellesley Extension Sewer	USA	1989		shield had to be removed through open cut after a bolder 1.5m in diameter could not be mined through	SPB TBM	jacked reinforced concrete pipe		internal rock crusher for boulders of 38.1 cm in diameter	11
8	Cairo Metro	Egypt	1993			SPB TBM		65.6ft/day		12, 13
9	Cairo waste water project	Egypt	1980's, 1990's	0.6-1.0in	one case of blow-out, compressed air->raising of groundwater table-flooding the basements	APB TBM		23.0ft/day		14
10	Cairo waste water project	Egypt	1980's,	0.8-3.5in		EPB TBM				14
11	Cairo waste water project	Egypt	1980's, 1990's	0.8-2.6in		SPB TBM	concrete segments	78.7ft/day	cutterhead modification to accommodate boulders	14
13	Caracas Metro (Propatria-Fuerzas	Venezuela				CF TBM	concrete segments	16.4ft/day	breasting plates	16
14	Cologne's Urban Railway	Germany	1992	0.6in	cave-in when old well shaft encountered (3 hours delay)	SPB TBM	concrete segments	26.2ft/day		17
15	Creteil-Valenton Tunnel, north-west of Paris - wastewater diversion	France		0.1in		SPB TBM	concrete segments + cast-in place concrete			18
16	Edmonton Experimental Sewage tunnel	Canada	1979	0.5in		CF TBM	concrete segments	42.0ft/day		19, 20
17	Edmonton LRT Subway	Canada	1981	0.4in		CF TBM (Lovat)	steel ribs and wooden lagging, cast-in-place concrete	8.5ft/day		21, 22
19	Essen subway (Baulos 34)	Germany				SPB/EPB	concrete segments	29.5ft/day		27, 102
21	Grauholtz		1991		problems with separating fine solids from the slurry	SPB TBM	steel segments	15.7ft/day	stone crusher (1.1m boulders)	30, 31, 29
28	Lisbon Metro	Portugal		3.9in		EPB TBM	concrete segments	39.3ft/day		36
29	London - test tunnel	England	1972			SPB TBM	steel segments	13.1ft/day		4
30	Lyon Metro Line D	France	1984-1986	< 0.1in	blow-out by bentonite pressure resulted in 0.8 m diameter chimney	SPB TBM	extruded concrete with steel fibres	65.6ft/day		37, 38, 39, 40, 41, 42,
32	Mexico City Sewage	Mexico	1984-1986	1.0in		SPB TBM	concrete segments	32.8ft/day		45
33	Milan - Passante Ferroviario (railway twin tube)	Italy		1.0in		EPB TBM	extruded concrete behind segmented lining	78.7ft/day	chemical foam injection in the cutting chamber	52
34	Munich Underground	Germany	1994			APB TBM		34.4ft/day		46
37	Osaka Stormwater Tunnel 1 and 2	Japan	1985	0.6in		SPB TBM	concrete segments			49, 50
38	Osterode - Butterberg tunn. - 50 km southeast from Hannover		1977-1979	0.5in		NATM	shotcrete 30cm, wire mesh, ribs prim.	6.6ft/day		51
41	Rome - Galleria Aurelia -railway	Italy	1989	0.3in		SPB TBM	concrete segments	29.5ft/day		2, 53, 54, 55,
42	San Francisco (BART)	USA		3.9in		OF TBM	steel welded pan segments			58

Table 8.1

Performance of Urban Tunnels in Soft Ground

No	Project	Country	Year	Settlement (in)	Reported Failure	Tunneling Method	Lining - Initial/Final	Rate of Advance (ft/day)	Special features	Reference No [1]
43	San Francisco (MUNI) Metro Turnback	USA	1994	5.8in		OF TBM	steel welded pan segments stiffened with ribs	11.8ft/day	compressed air, face breasting, manual excavation with pneumatic spade	59
44	San Francisco, water-way	USA	1981	1.1in		EPB TBM	steel segments	29.8ft/day		60, 61, 62,
45	Sao Paulo - Alto da Boa Vista Tunnel	Brazil	1978	0.2in		NATM	shotcrete 10cm+3cm	5.2ft/day		66, 67, 68,
46	Sao Paulo - subway under Boa Vista street and Caixa Economica	Brazil	1973	0.8in		CF TBM	segment iron rings	19.7ft/day		70
47	Sao Paulo - subway north of Prestes Maia shaft	Brazil	1973		running face where ground water control was ineffective	OF TBM (manual	segment iron rings	13.1ft/day	compressed air, breasting plates	70
48	Seattle Downtown Transit	USA	1987	2.4in (in flowing sand)	two sinkholes	OF TBM (digger shield)	water proof membrane (PVC), concrete segments		orange-peel breasting doors	71, 72
49	Seattle -West Tunnel - wastewater tunnel	USA	1994	2.4in (in flowing sand)		EPB TBM	bolted and gasketed concrete segments			108
53	Singapore, subway	Singapore		1.6in	high face pressures generated pore pressures which led to large consolidation settlements (12 cm)	EPB TBM	concrete segments			76
54	Taipei Metro - Taiwan	Taiwan	1992	1.0in		EPB TBM	concrete segments	55.7ft/day	compressed-air locks useful when working chamber has to be entered to remove an obstacle	77
56	Tokyo - railway	Japan	1980			SPB TBM		12.1ft/day	crusher at the bottom of the chamber	4
62	Tokyo, Shinozaki trunk sewer	Japan		0.9in	face collapse due to insufficient grouting	SPB TBM				82
64	Toronto - sewer tunnel	Canada	1972	0.1in		OF TBM	ribs and concrete planks + cast-in-place concrete	16.4ft/day		83, 84
65	Toronto - sewer tunnel	Canada	1966	4.0in		OF TBM (hand	ribs and concrete planks + cast-in-place concrete			85
66	Vienna Subway	Austria	1983	0.2in		NATM			compressed air	86
67	Villejust - railway tunnel	France	1984-1988	3.7in		SPB TBM	concrete segments	50.5ft/day		2, 100, 87,
68	Washington D.C. (WMATA) - A2 test section	USA		7.0in		OF TBM	expanded steel ribs	23.9ft/day	breast jacks in the top and breast plate 5ft below the crown, poling plates from crown to springline, ripper-bucket with an articulated arm	90, 91
69	Washington D.C. (WMATA) - Sections E-5, 6e, 8e	USA	1987			NATM	shotcrete + cast-in-place concrete	14.8ft/day		92, 93
70	Washington D.C. (WMATA) - Anacostia River Crossing	USA	1987	5.9in		EPB TBM (Hitachi)		23.9ft/day	settlement in stiff clays which did not form plasticized mass and gravel and sand tended to ravel in	94
71	Washington D.C. (WMATA) - Greenbelt	USA	1987	1.5in		CF TBM	concrete segments		compressed air	94, 95, 96
72	Washington D.C. (WMATA), Branch Rte. and Pentagon Rte. Tunnels	USA		1.6in	30 ft. chimney collapse	OF TBM (Robbins)	concrete segments		hydraulic claw excavator, orange-peel breasting doors	97
73	Yokohama - River Diversion Channel	Japan	1991	0.2in		NATM	shotcrete + cast-in-place concrete		drainage boring	98
74	Yokohama City - Konan Tunnel	Japan		0.7in		NATM	forepoling, shotcrete + cast-in-place concrete			99

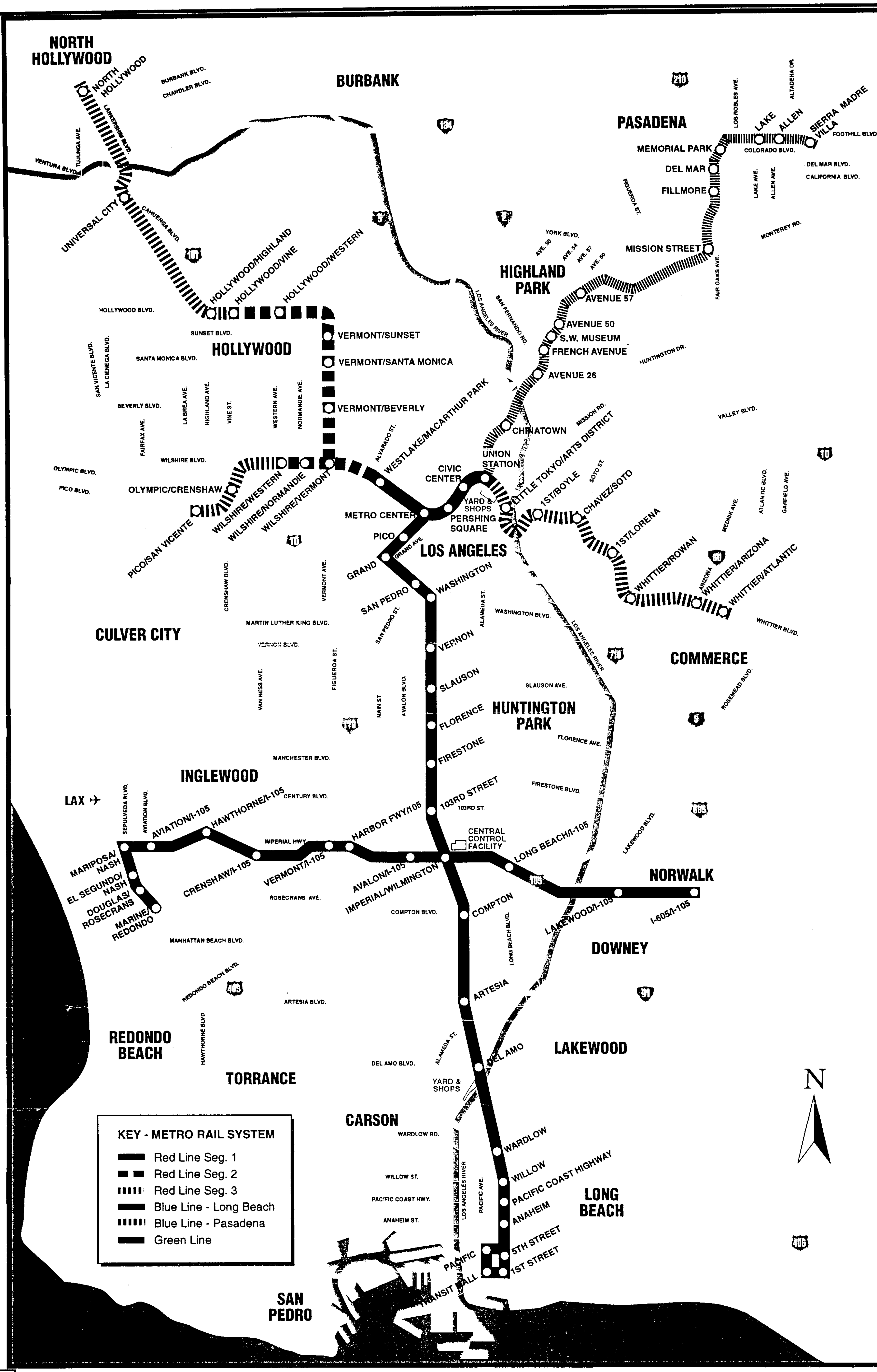
[1] References for the case histories are provided after the main text of the appendix.

Table 8.2

Geotechnical Performance on Tunneling Contracts for Red Line Project

Contract	Settlement	Geotechnical Problems	Category
A130	less than 1 inch except in one short section	none	1
A141	less than 1 inch everywhere	thin liner	3
A146	less than 1 inch everywhere	none	1
A171	less than 1 inch everywhere	none	1
B201	70% less than 1 inch 24% between 1 and 1.5 inch 6% greater than 1.5 inch	none	2
B221	96% less than 1 inch	none	1
B251	51% less than 1 inch 35% between 1 and 2 inches 14% greater than 2 inches	▸ sinkhole on Hollywood Blvd. ▸ excessive settlement at Hudson Street ▸ Remining at three locations	3
C331	greater than 3 inches	excessive settlement at Lankershim Blvd.	3

12-1-95/TABLE.8-1/W7101-lkd/dgw



10/95

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 1.1** **Los Angeles Metro Rail System**	November 1995 W-7101-01

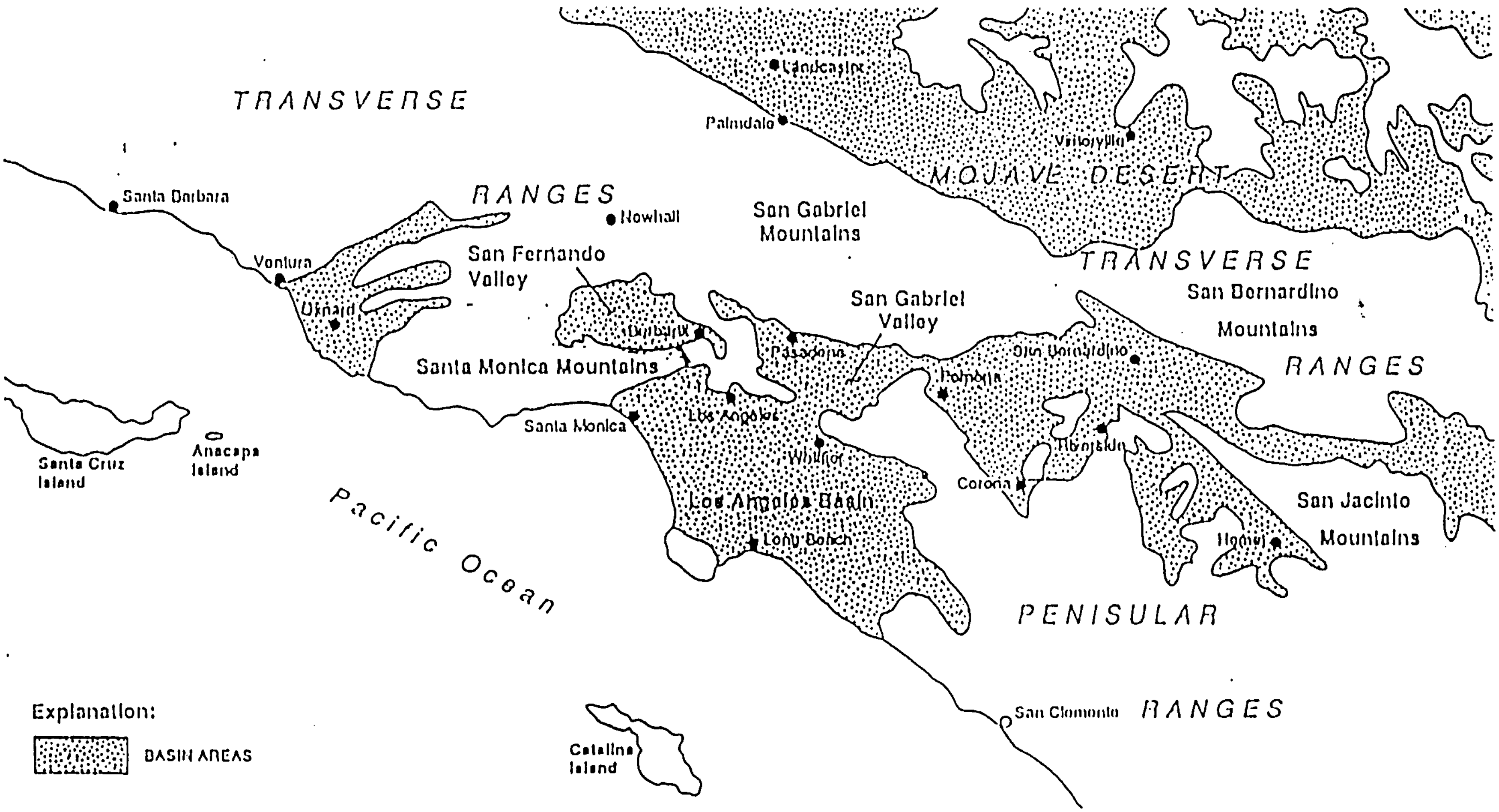
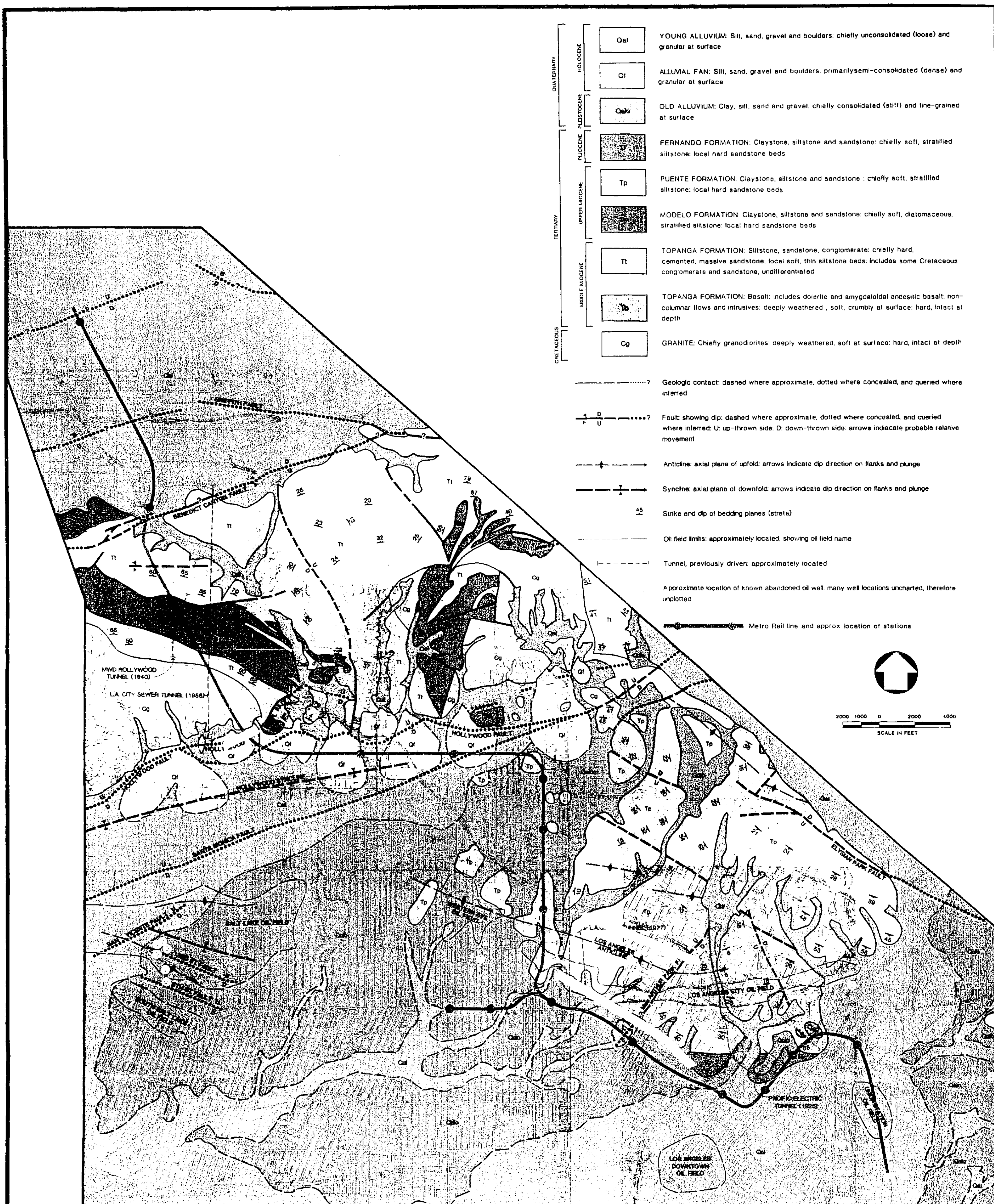


FIG. 2.1

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 2.1** **Regional Physiography of Los Angeles Region**	November 1995 W-7101-01



Prepared by:
METRO RAIL TRANSIT CONSULTANTS
DMJM, PBQD/KE/HWA
548 South Spring Street, Seventh Floor
Los Angeles, CA 90013

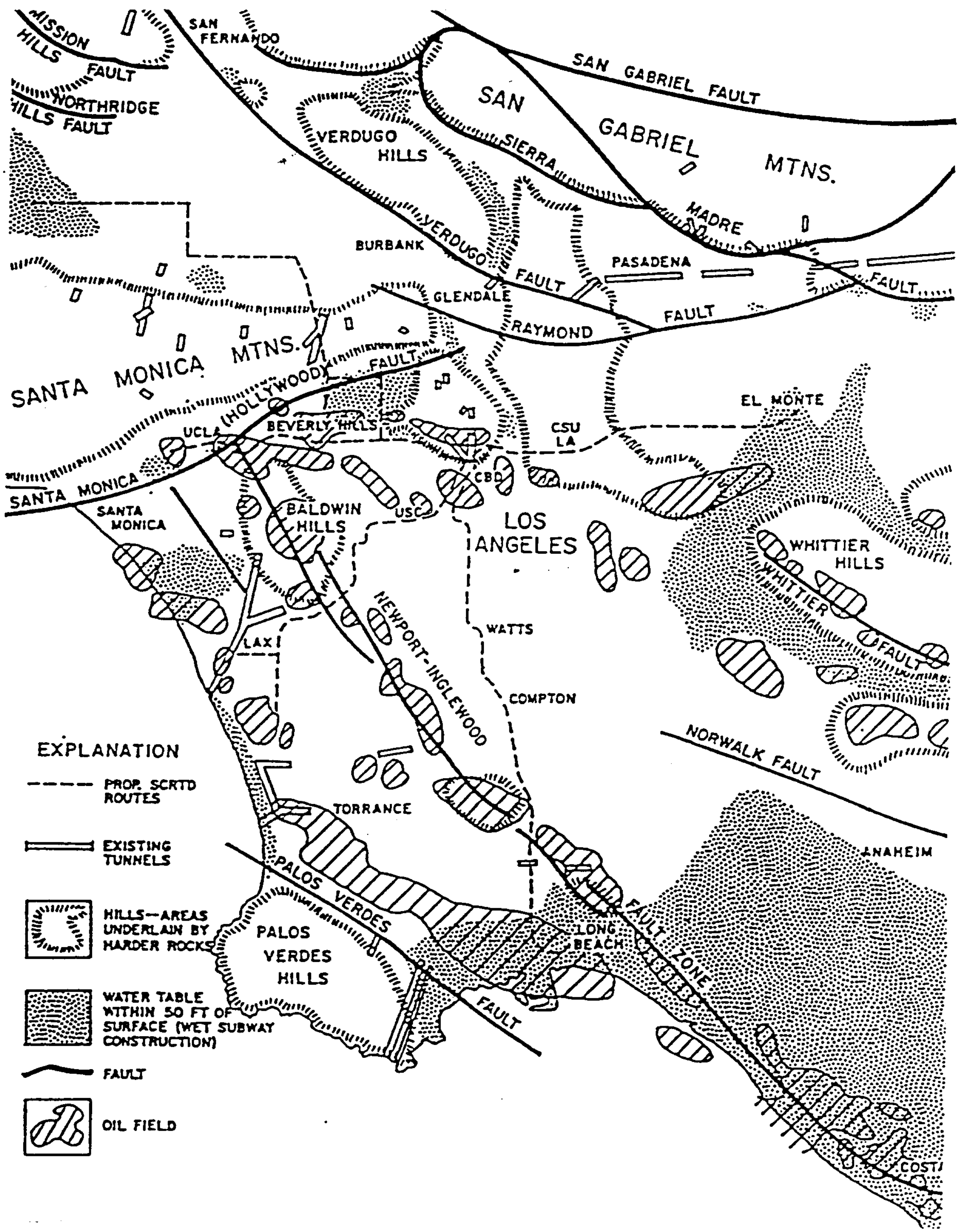
GEOLOGIC MAP

Southern California Rapid Transit District
METRO RAIL PROJECT

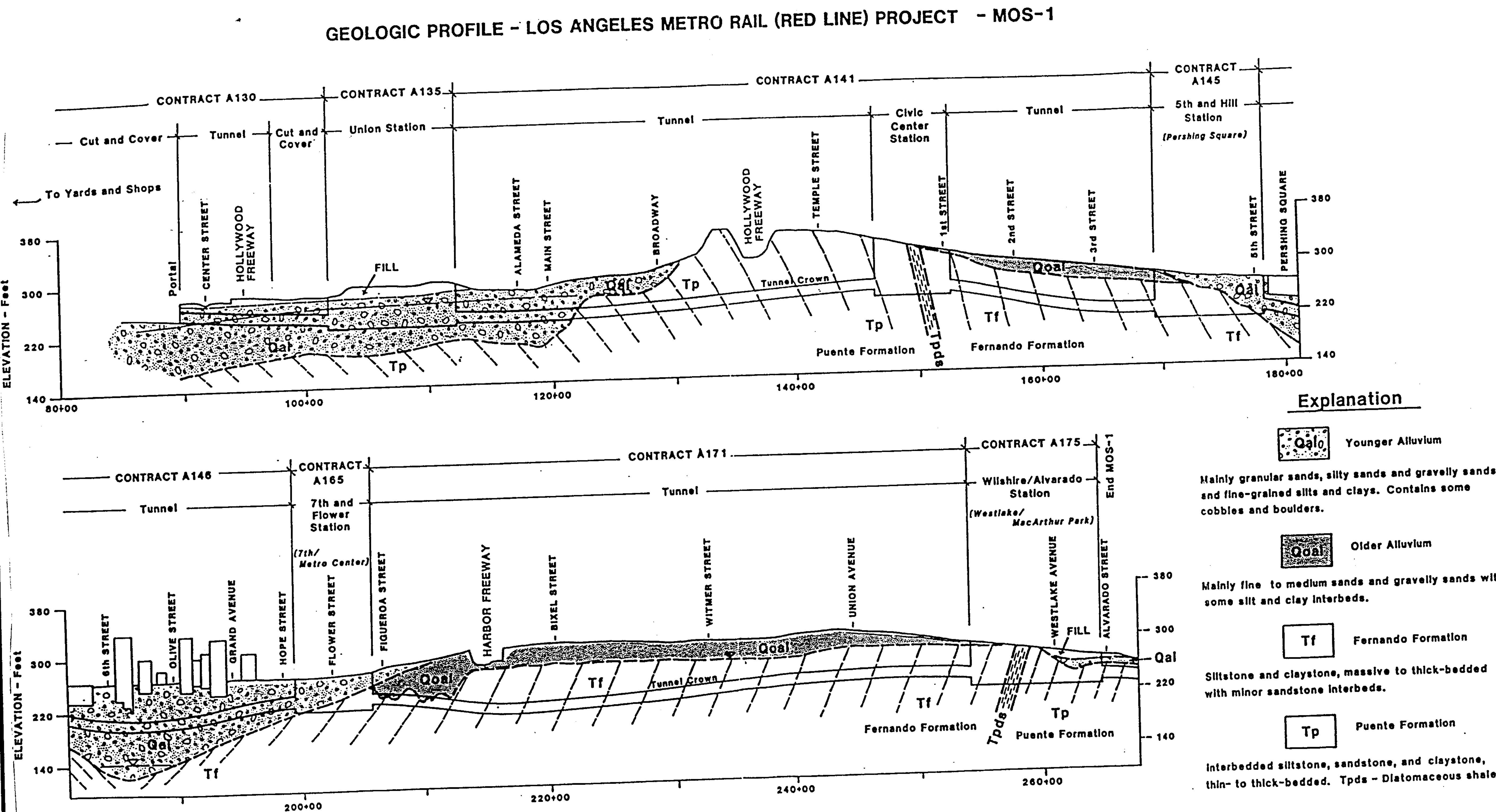
REFERENCES: Adapted from geology compiled from CWDD/ESA/GRC (1981)

FEBRUARY 1989

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	Figure 2.2 Geologic Map	November 1995 W-7101-01



For: Los Angeles County Metro Rail Project By: Geotechnical Panel	Figure 2.3 Generalized Geologic and Groundwater Conditions in the Los Angeles Basin (After Proctor, 1981)	November 1995 W-7101-01

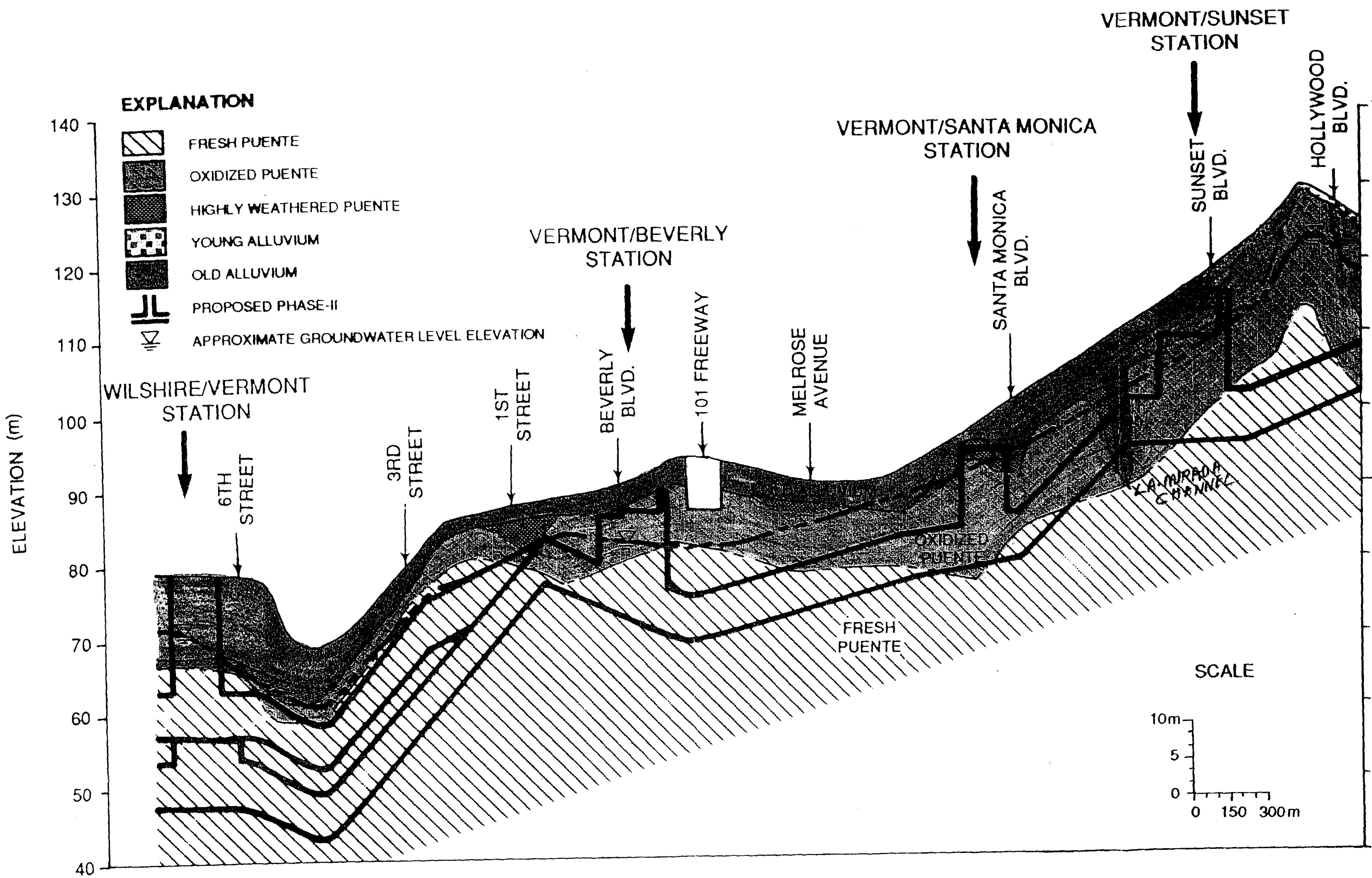


November 1995
W-7101-01

Figure 2.4
Geologic Profiles, Segment 1, Red Line Project (After Escandon et. al., 1992)

For: Los Angeles County Metro Rail Project
By: Geotechnical Panel

FIG. 2.4

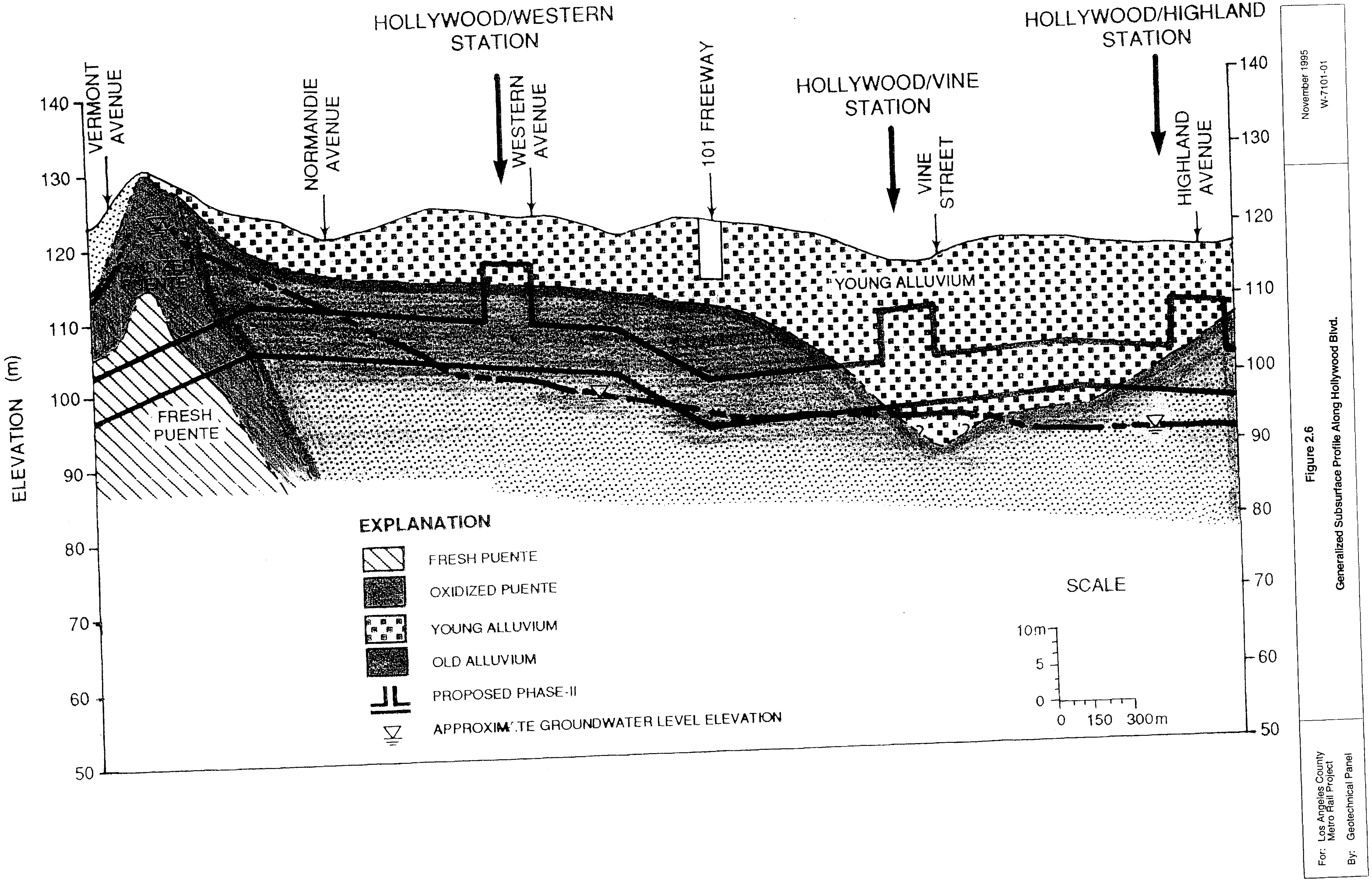


November 1995
W-7101-01

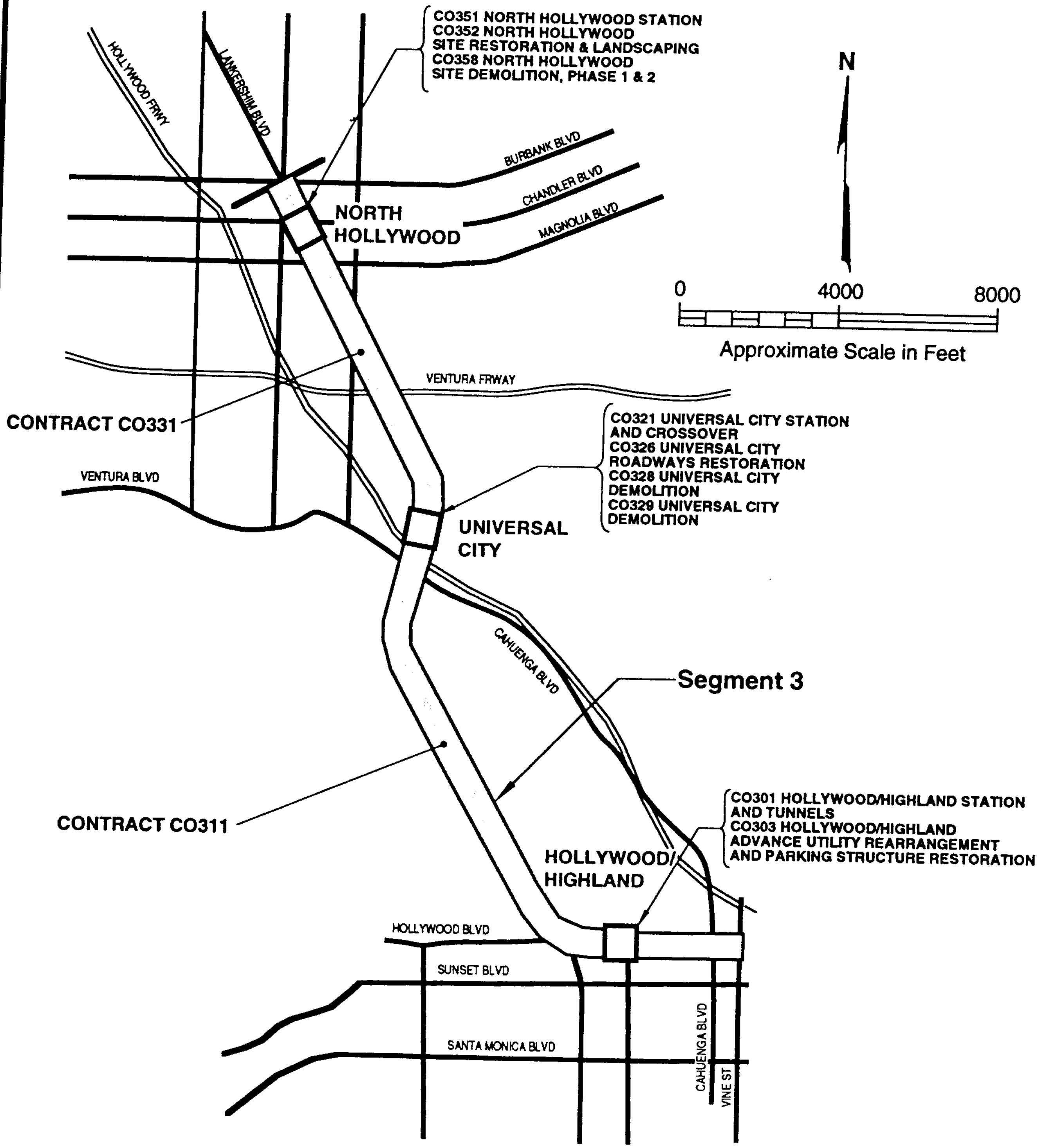
Figure 2.5
Generalized Subsurface Profile Along Vermont Avenue

For: Los Angeles County Metro Rail Project
By: Geotechnical Panel

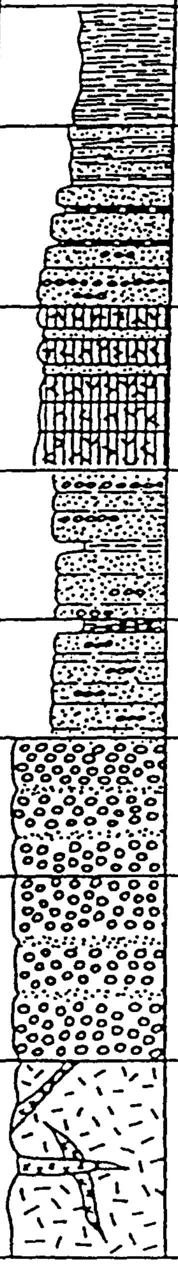
GENERALIZED SUBSURFACE PROFILE ALONG VERMONT AVENUE



GENERALIZED SUBSURFACE PROFILE ALONG HOLLYWOOD BLVD.



For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 2.7** **Metro Red Line Project Segment 3**	November 1995 W-7101-01



AGE		FORMATION (Map Symbol)	APPROXIMATE STRATIGRAPHIC THICKNESS IN PROJECT AREA (Feet)	LITHOLOGY	DESCRIPTION
Period	Epoch				
TERTIARY	MIOCENE	PUENTE (Tp)	30+		Highly sheared dark gray to black claystone and shale. Thin bedded to laminated. Occurs as a fault sliver within the Hollywood fault zone. Marine
					FAULT
TERTIARY	MIOCENE	TOPANGA UPPER (Ttu)	3200+		Light gray to tan bedded sandstone and pebbly conglomerate. Grades upward into mostly gray micaceous siltstone and shale or claystone with interbeds of sandstone. Marine.
TERTIARY	MIOCENE	TOPANGA MIDDLE (Ttv/Tts)	1200-1500		Dark gray to black fine-grained basaltic volcanic rocks. Typically massive to brecciated. Locally vesicular with vesicles filled with white zeolite, calcite and chlorite. Typically hydrothermally altered to chlorite and serpentine group minerals. Includes interbeds of resistant sandstone. Marine.
TERTIARY	MIOCENE	TOPANGA LOWER (Ttl)	350-400		Light gray to tan massive conglomeratic sandstone and sandstone. Locally includes thin bedded sandstone, shale and cobble conglomerate. Clasts consist of quartzite, sandstone, slate, plutonics and volcanics. Marine.
					UNCONFORMITY
TERTIARY	PALEOCENE	LAS VIRGENES SANDSTONE (Tlv)	100-225		White arkosic sandstone with felsic sand pebble lenses grading into a greenish silty claystone. Thin green and red mudstone interbedded. White bentonite bed occurs near base. Nonmarine.
TERTIARY	PALEOCENE	SIMI CONGLOMERATE (Tsc)	200-300		Cobble and boulder conglomerate with coarse sand lenses. Lower half characterized by distinctive quartzite clasts. The upper half contains mostly volcanic clasts. Nonmarine.
					UNCONFORMITY
CRETACEOUS	LATE	"CHICO" (Kc)	700-800		Massive brown and gray cobble conglomerate with sandstone and dark gray shale interbedded. Clasts consist of metavolcanic and granitic rocks and quartzite in a sandy matrix. Locally includes reddish sandstone and claystone. Base sheared locally. Marine.
					UNCONFORMITY/FAULT
CRETACEOUS	(?)	UNNAMED PLUTONIC ROCKS (gd)	3500+		Medium to light gray granodiorite, quartz diorite and quartz monzonite that is massive to locally gneissic. Composed mostly of plagioclase feldspar, quartz, biotite and hornblende. Dikes and veins of basalt and quartz aplite occur. Mafic xenoliths locally abundant. Deeply weathered near surface. Nonmarine.

For: Los Angeles County Metro Rail Project

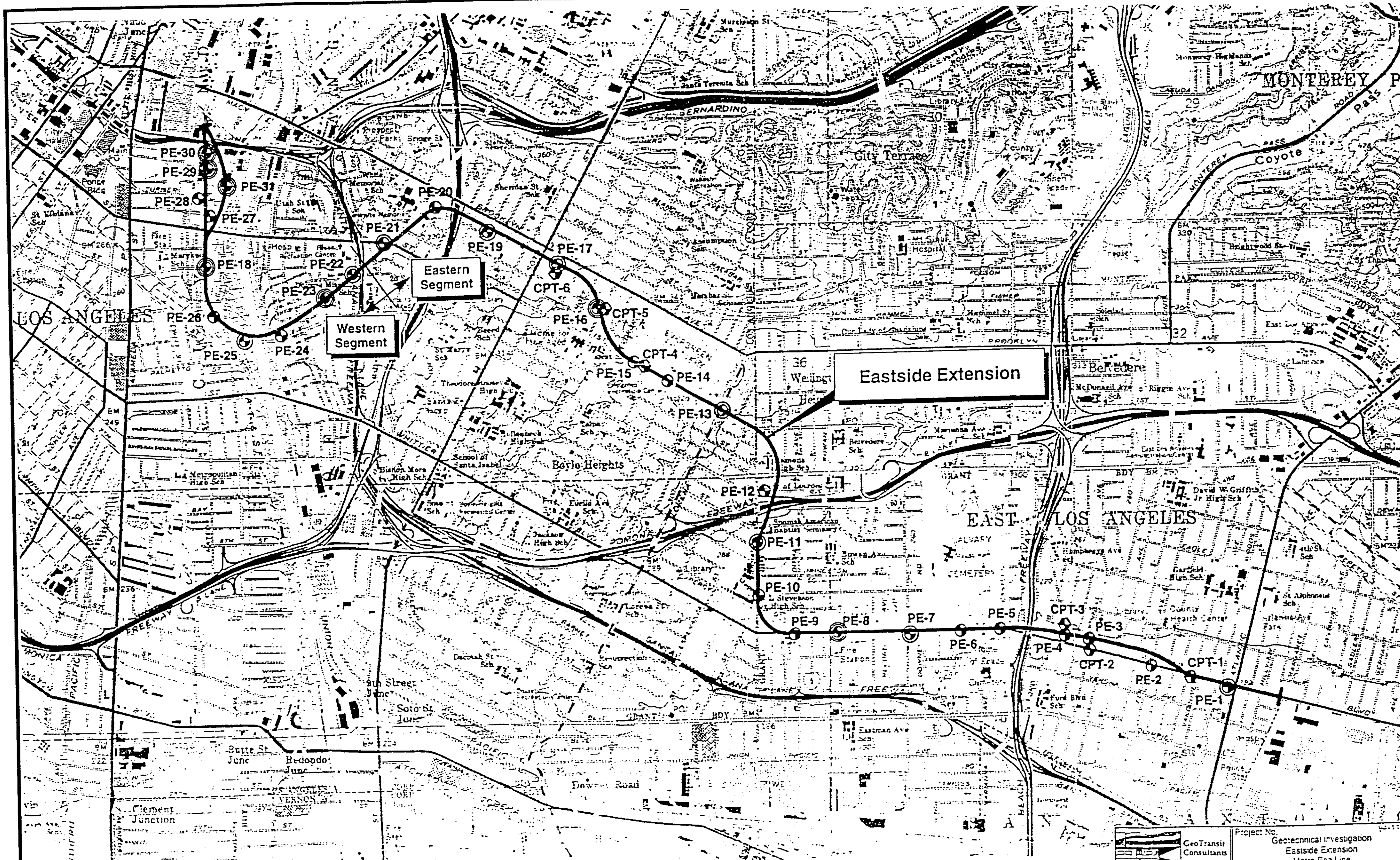
By: Geotechnical Panel

Figure 2.8

Stratigraphic Sequence of Bedrock Units

November 1995

W-7101-01



EXPLANATION

PE-31 Rotary Wash Boring Drilled for this Investigation

PE-1 Rotary Wash Boring with Piezometer Installed for this Investigation. Piezometers Converted to Monitoring Well Indicated by

CPT-6 Cone Penetration Test Conducted for this Investigation.

Map Location

NORTH

0 1000 2000 Feet

Scale

Base Map: U.S. Geological Survey Topographic Map, Los Angeles 7.5 Minute Quadrangle dated 1966; Photorevised 1981

GeoTransit Consultants

Project No.
Geotechnical Investigation
Eastside Extension
Metro Red Line

Location Map and Exploration Plan

2-94

Figure 2-1

FIG. 2.9

Figure 2.9
Metro Red Line Project
Eastside Extension

For: Los Angeles County Metro Rail Project

By: Geotechnical Panel

November 1995

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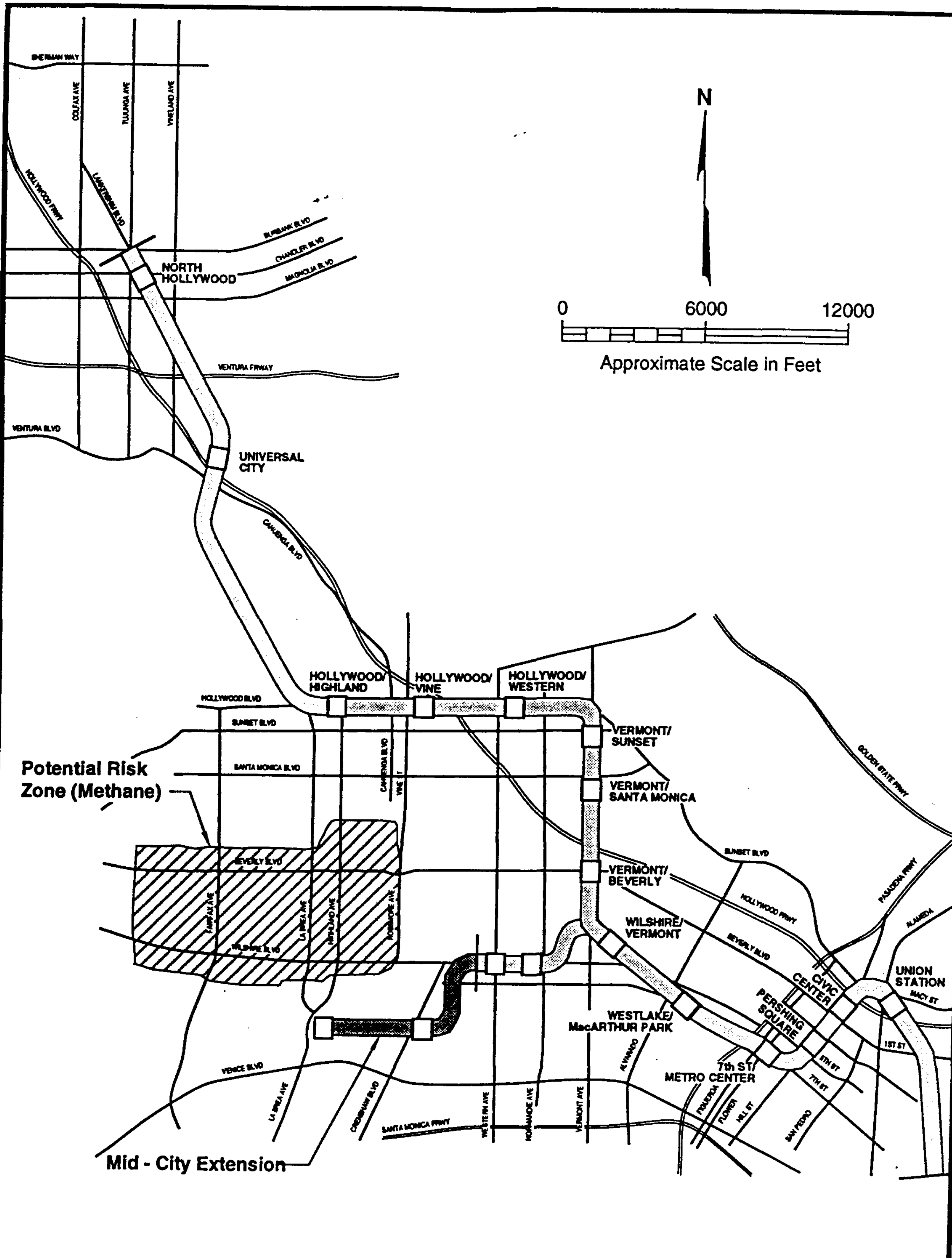


For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 2.10** **Metro Red Line Project Mid-City Extension**	November 1995 W-7101-01

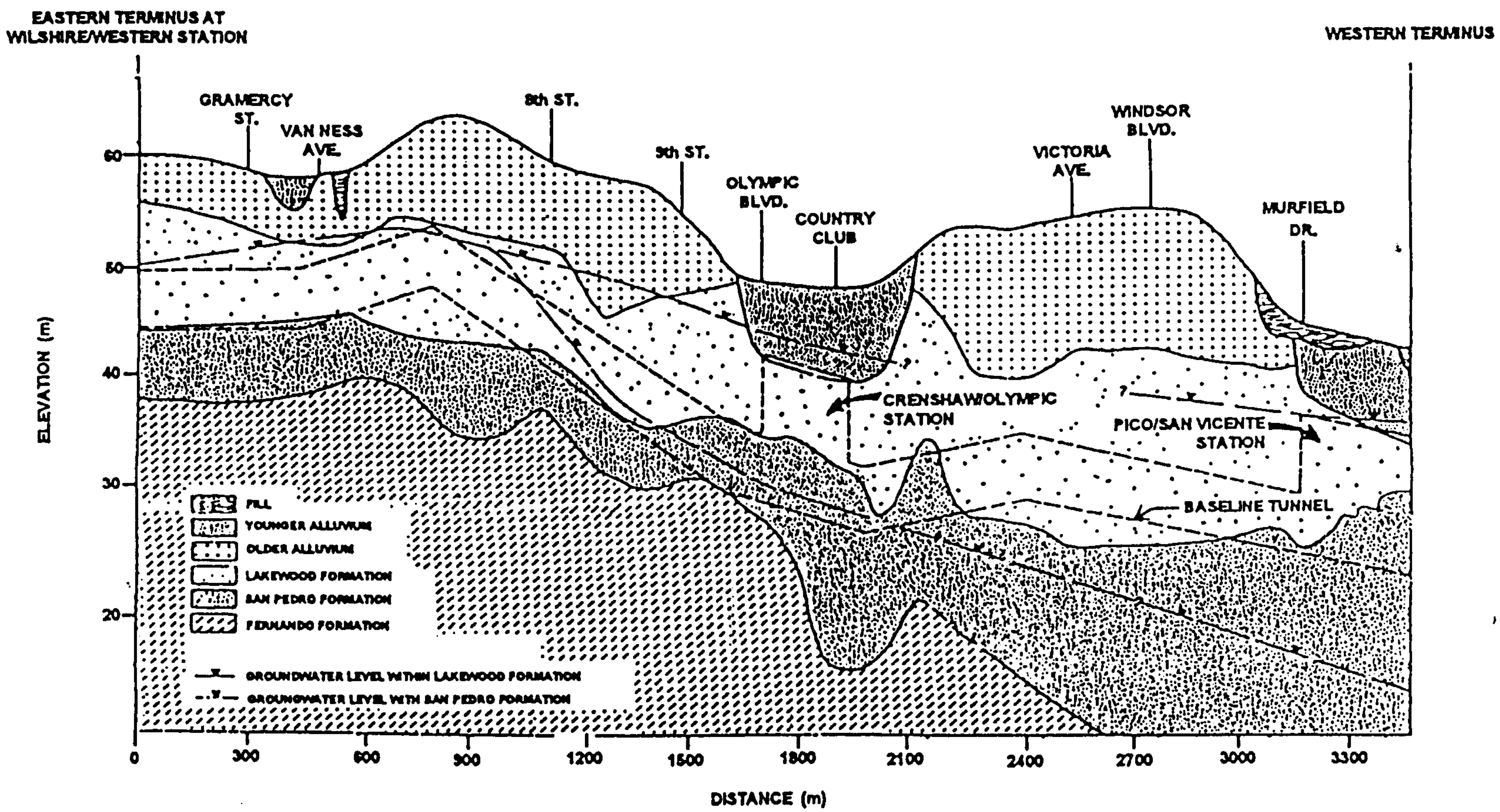


FIG. 2.11

For: Los Angeles County
Metro Rail Project

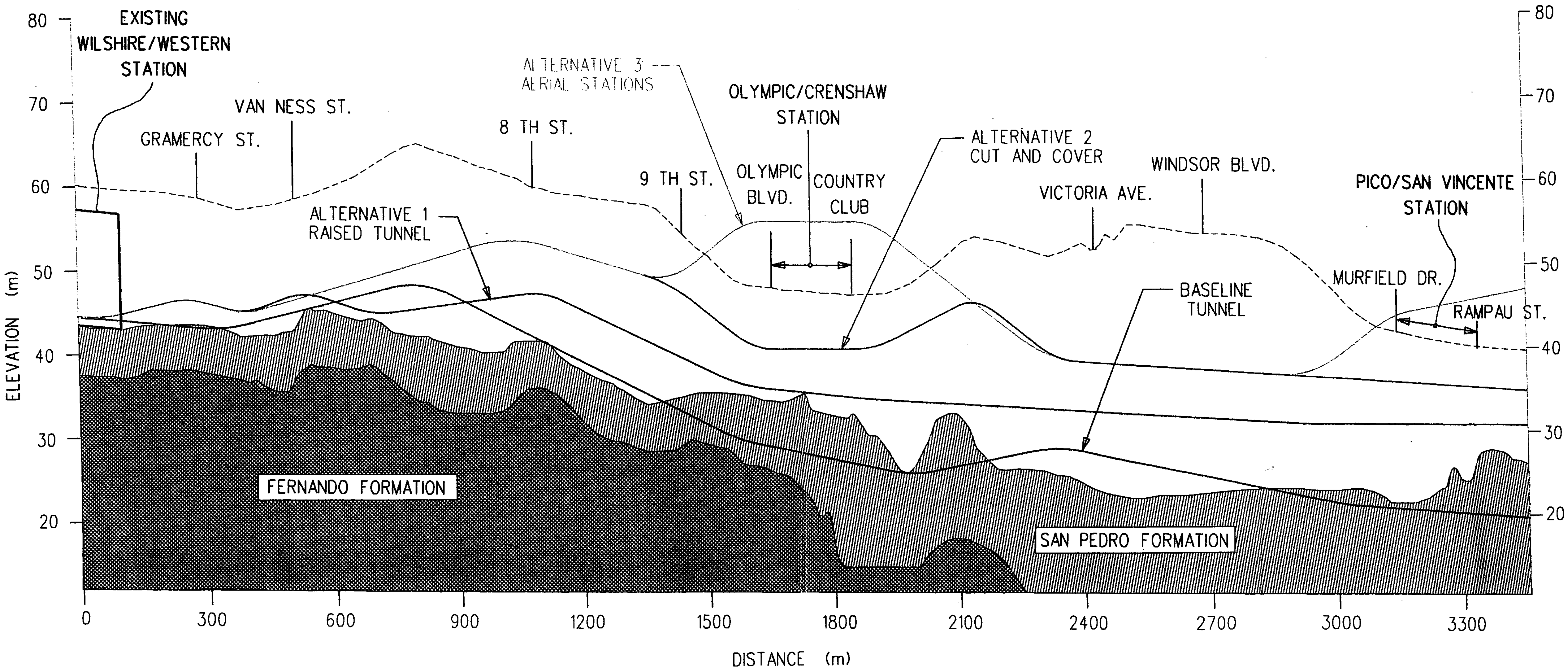
By: Geotechnical Panel

Figure 2.11

General Geologic Profile
Along Mid - City Extension

November 1995

W-7101-01



PROPOSED ALTERNATIVE ALIGNMENTS MIDCITY EXTENSION

(top of rail elevations shown)

DESIGNED BY J. TODOROV DRAWN BY J.TODOROV DATE DEC. 29 1994	LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY METRO RED LINE ENGINEERING MANAGEMENT CONSULTANT SUBMITTED APPROVED	LA CBD TO MID-CITY GEOLOGIC PROFILE

November 1995

Figure 2.12
Proposed Alternative Alignments

For: Los Angeles County Metro Rail Project

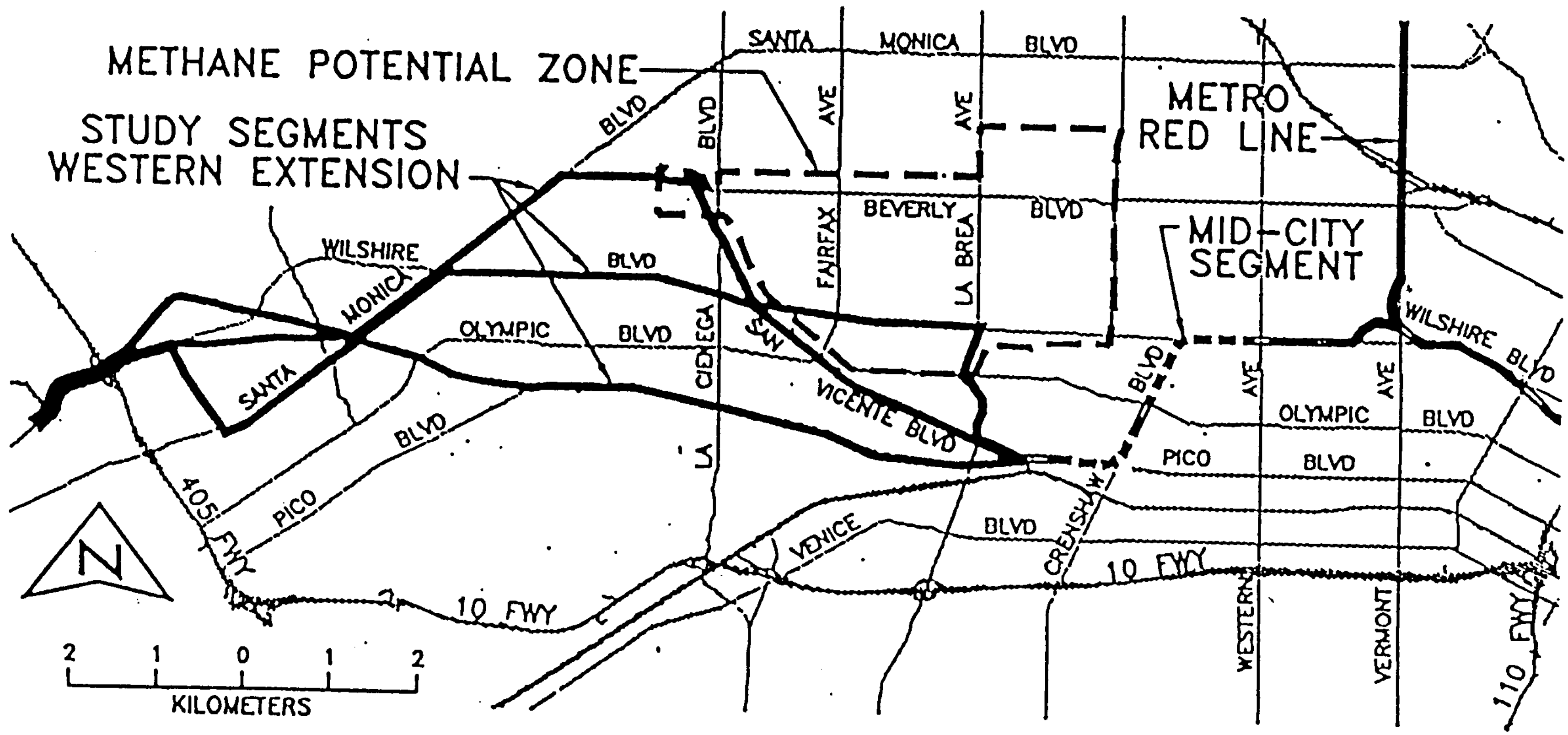


FIG. 2.13

For: Los Angeles County Metro Rail Project

By: Geotechnical Panel

Figure 2.13
Metro Red Line
Western Extension - Alternative Routes

November 1995

W-7101-01

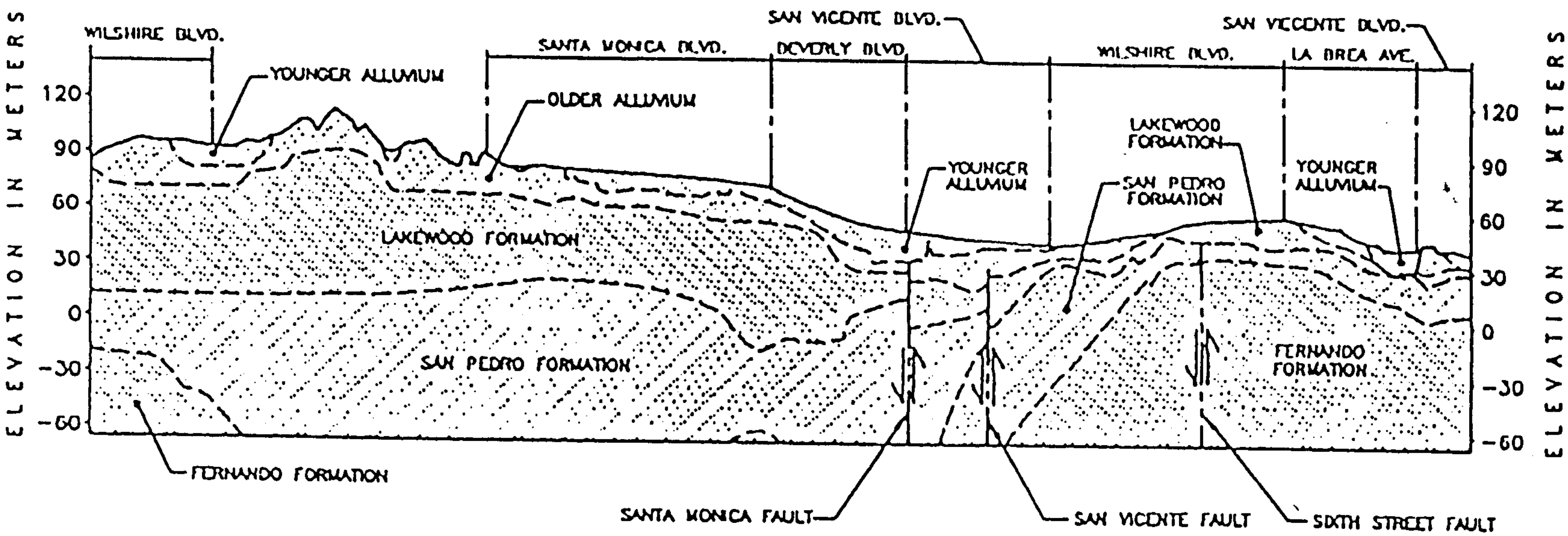


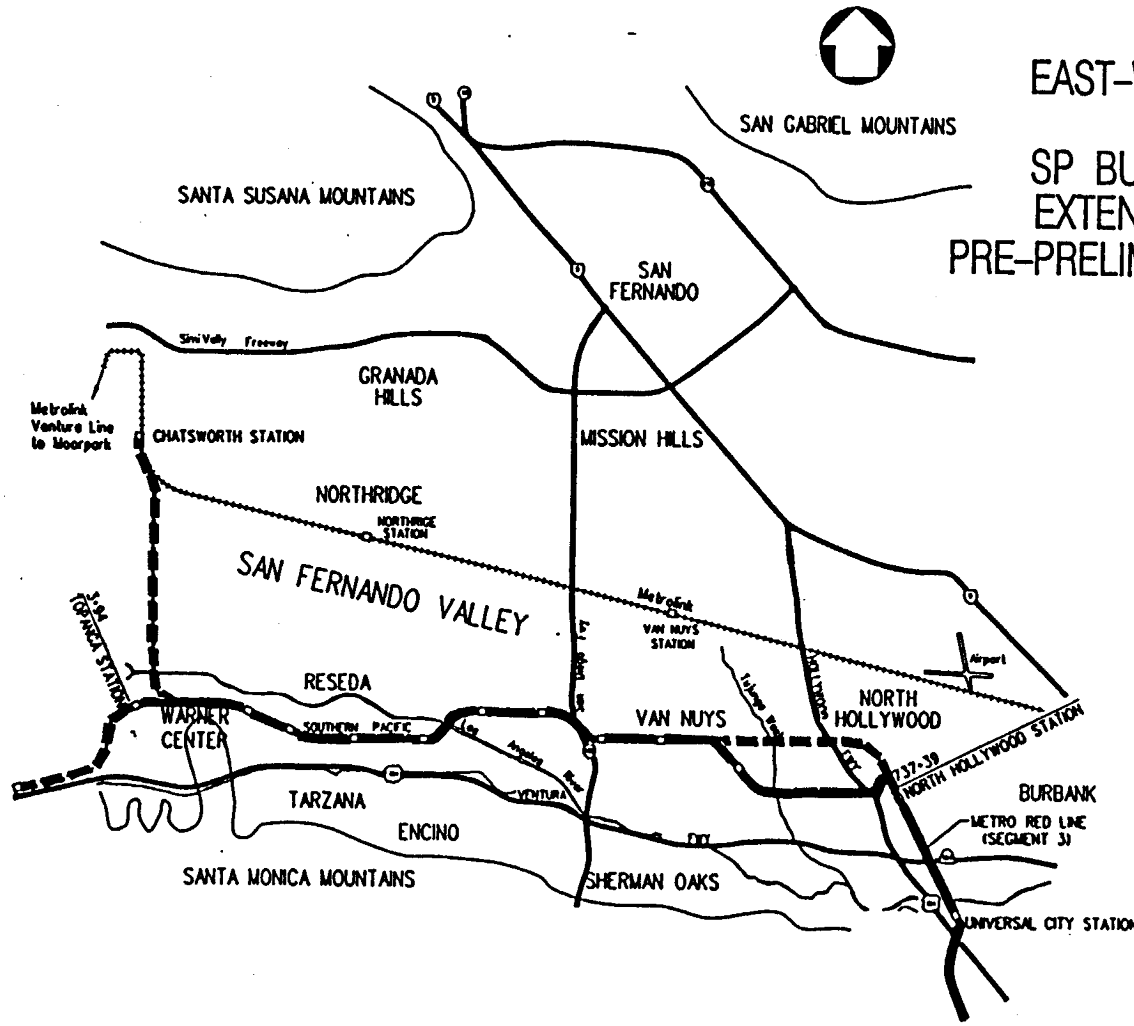
FIG. 2.14

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 2.14** **Western Extension Study Segment**	November 1995 W-7101-01

SAN FERNANDO VALLEY
EAST–WEST RAIL TRANSIT PROJECT

SP BURBANK BRANCH ALIGNMENT
EXTENDED METRO RAIL SOLUTION
PRE–PRELIMINARY ENGINEERING STUDY

AUGUST 1994



VOLUME II
FINAL REPORT

TECHNICAL DRAWINGS
BASE ALIGNMENT & OPTIONS

FIG. 2.15

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 2.15** **Proposed San Fernando Valley Extension**	November 1995 W-7101-01

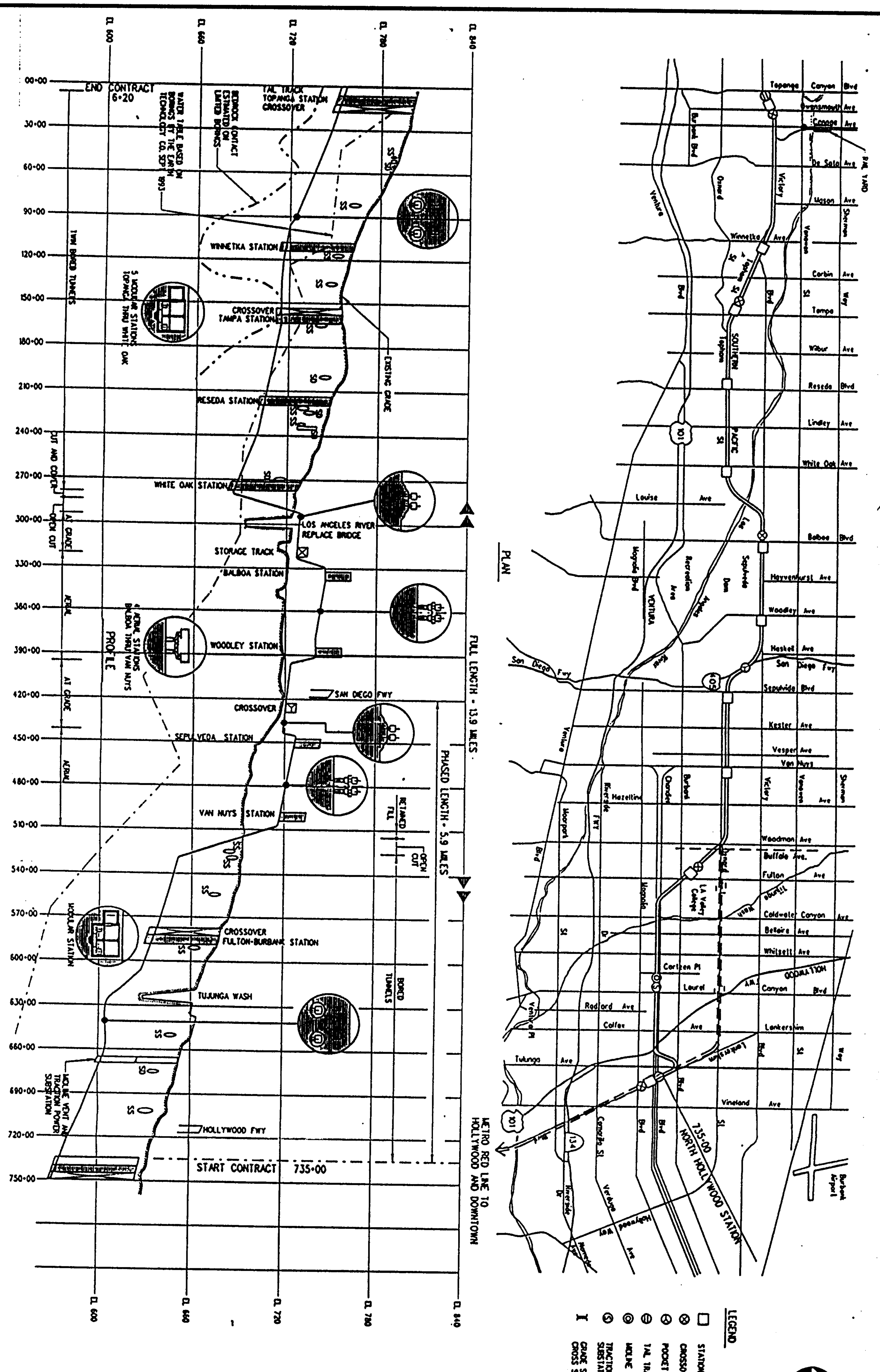


FIG. 2.16

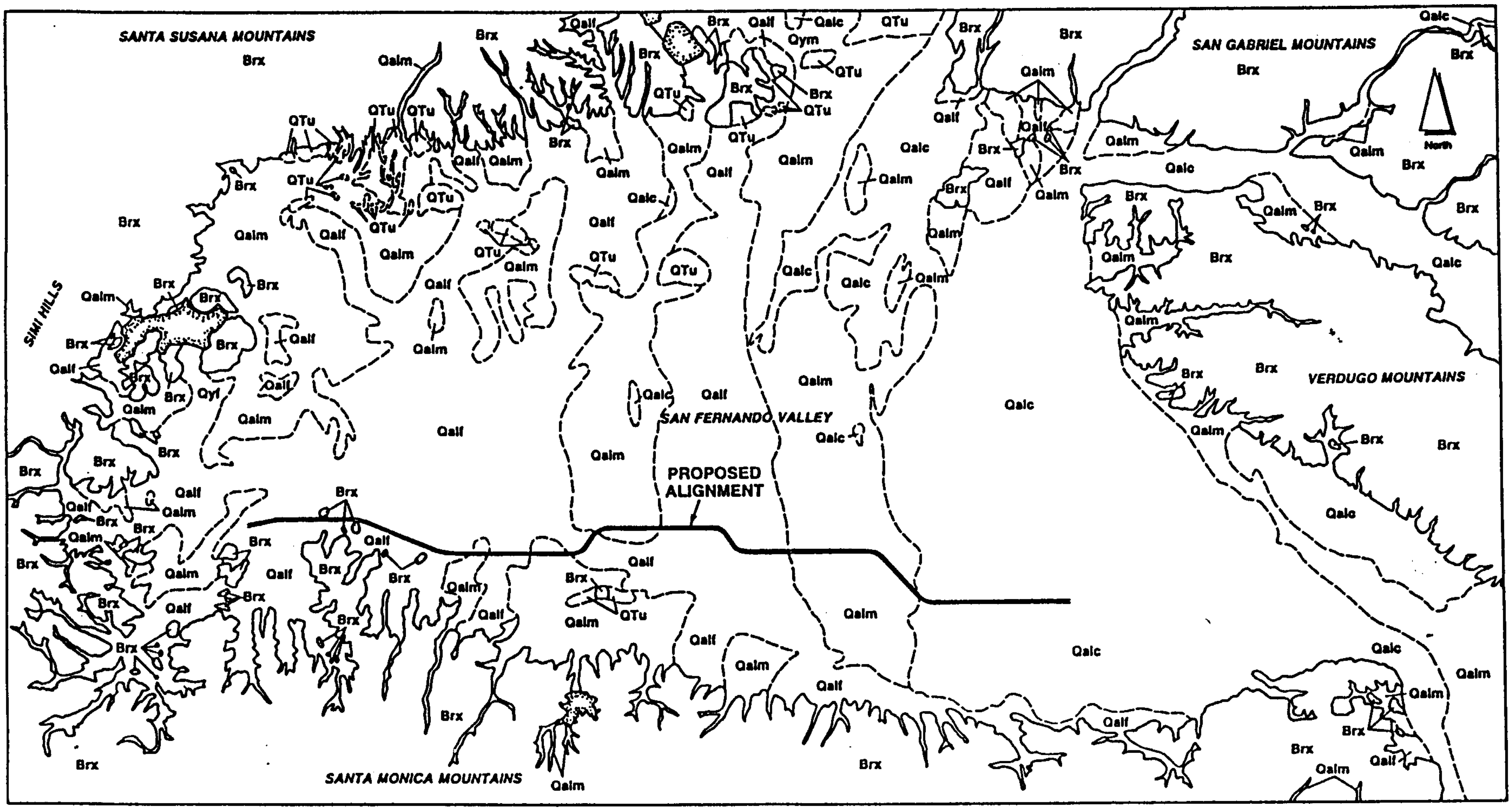
For: Los Angeles County Metro Rail Project

By: Geotechnical Panel

Figure 2.16
Representative Alternate Rail Profile

November 1995

W-7101-01



Explanation:

UNITS

Qalf	FINE-GRAINED (MAINLY SILT AND CLAY)	YOUNG VALLEY FILL ALLUVIUM
Qalm	MEDIUM-GRAINED (MAINLY SAND)	YOUNG VALLEY FILL ALLUVIUM
Qalc	COARSE-GRAINED (MAINLY SAND AND GRAVEL)	YOUNG VALLEY FILL ALLUVIUM
QTu	QUATERNARY AND LATEST TERTIARY OLDER ALLUVIUM DEPOSITS, UNDIFFERENTIATED	
Brx	UNDIFFERENTIATED BEDROCK (SEE FIGURE 4-2 FOR GENERALIZED SUBDIVISIONS)	

SYMBOLS

- CONTACT BETWEEN SUBUNITS OF YOUNG ALLUVIUM AND OLDER ALLUVIAL DEPOSITS, DASHED WHERE APPROXIMATELY LOCATED
- CONTACT BETWEEN ALLUVIUM AND BEDROCK
- PROPOSED RED LINE ALIGNMENT
- RESERVOIR

Sources:
1. Adapted From Tinsley and Fumal (1985) and State Water Rights Board Referee (1961).
2. Limits of Young Valley Fill Alluvium from Dibblee (1991 A,B,C,D; 1992 A,B,C,D).

0 2 4 Miles

Scale 1:100,000

FIG. 2.17

For: Los Angeles County Metro Rail Project

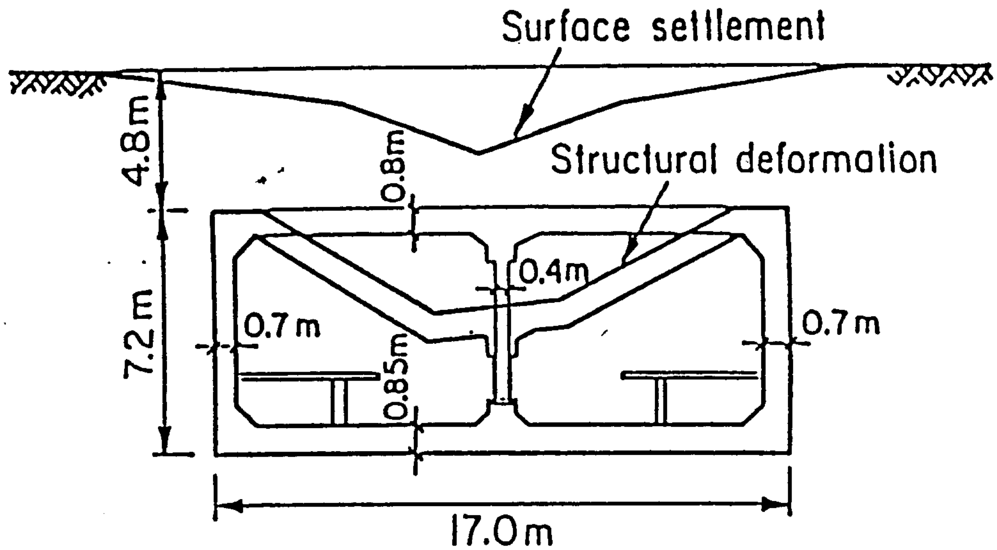
By: Geotechnical Panel

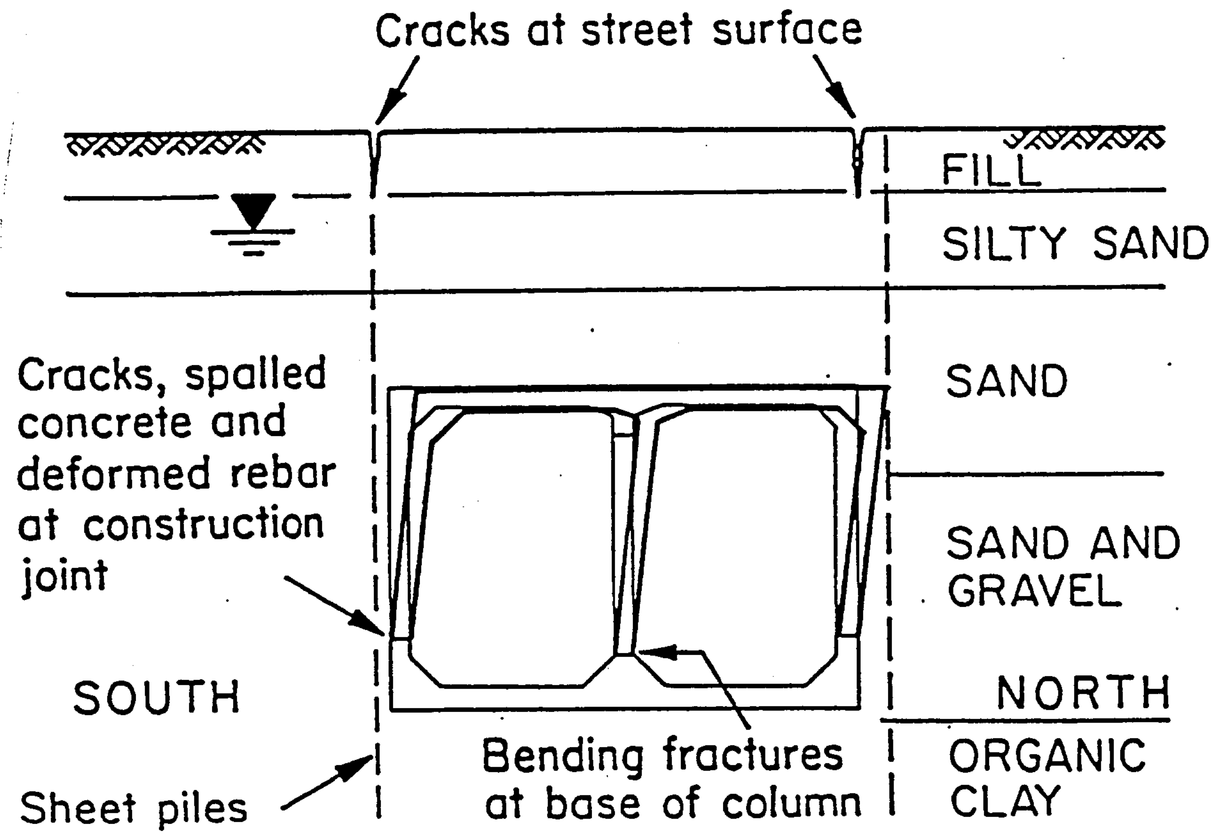
Figure 2.17

Generalized Geologic Map of Alluvial Deposits

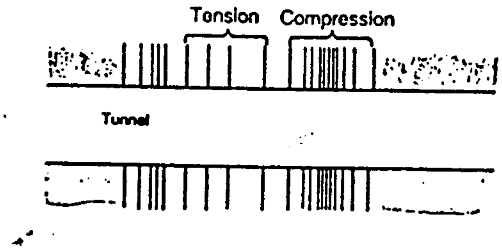
November 1995

W-7101-01

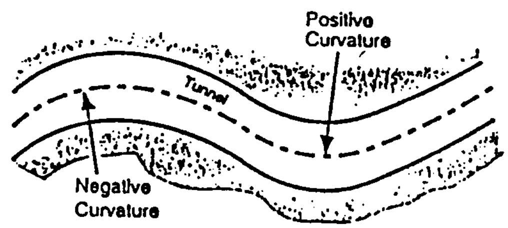




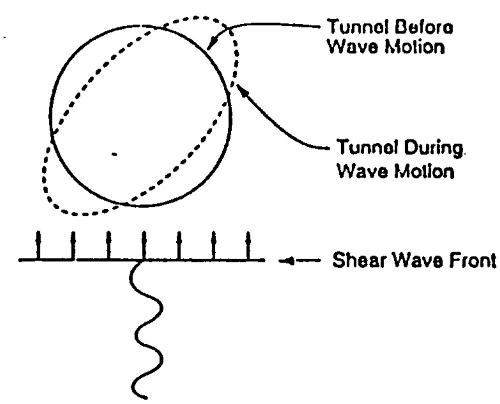
For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 2.18** **Damaged Station and Tunnel Sections, Kobe Rapid Transit Railway (After O'Rourke, 1995)**	November 1995 W-7101-01



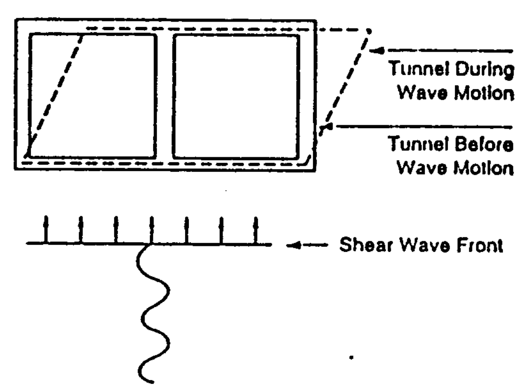
Axial Deformation Along Tunnel



Curvature Deformation Along Tunnel



Ovaling Deformation of a Circular Cross Section



Racking Deformation of a Rectangular Cross Section

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 2.19** **Tunnel Deformation Mechanisms Under Earthquake Loading (After Wang, 1991)**	November 1995 W-7101-01

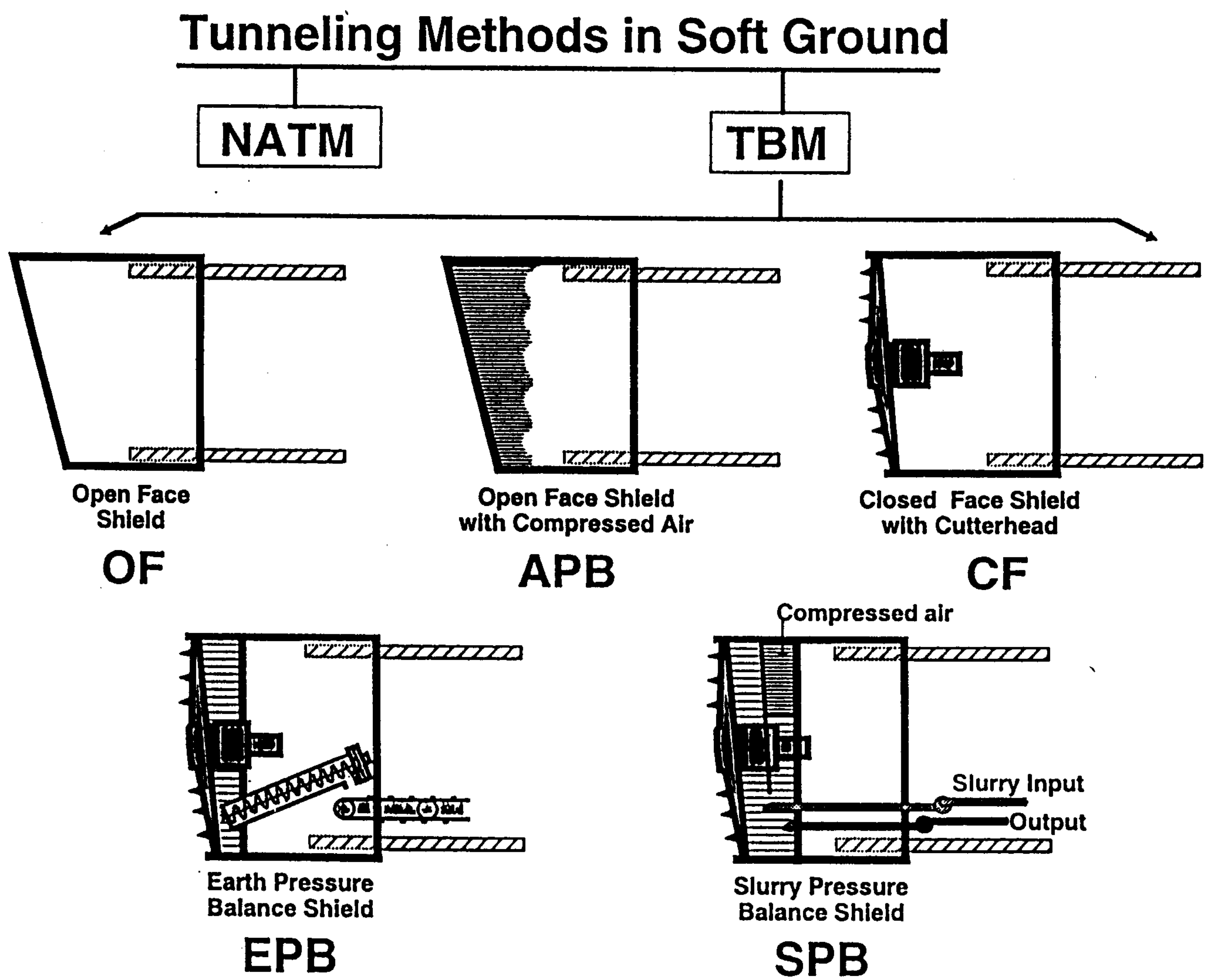


FIG. 3.1

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	Figure 3.1 Tunneling Methods in Soft Ground	November 1995 W-7101-01

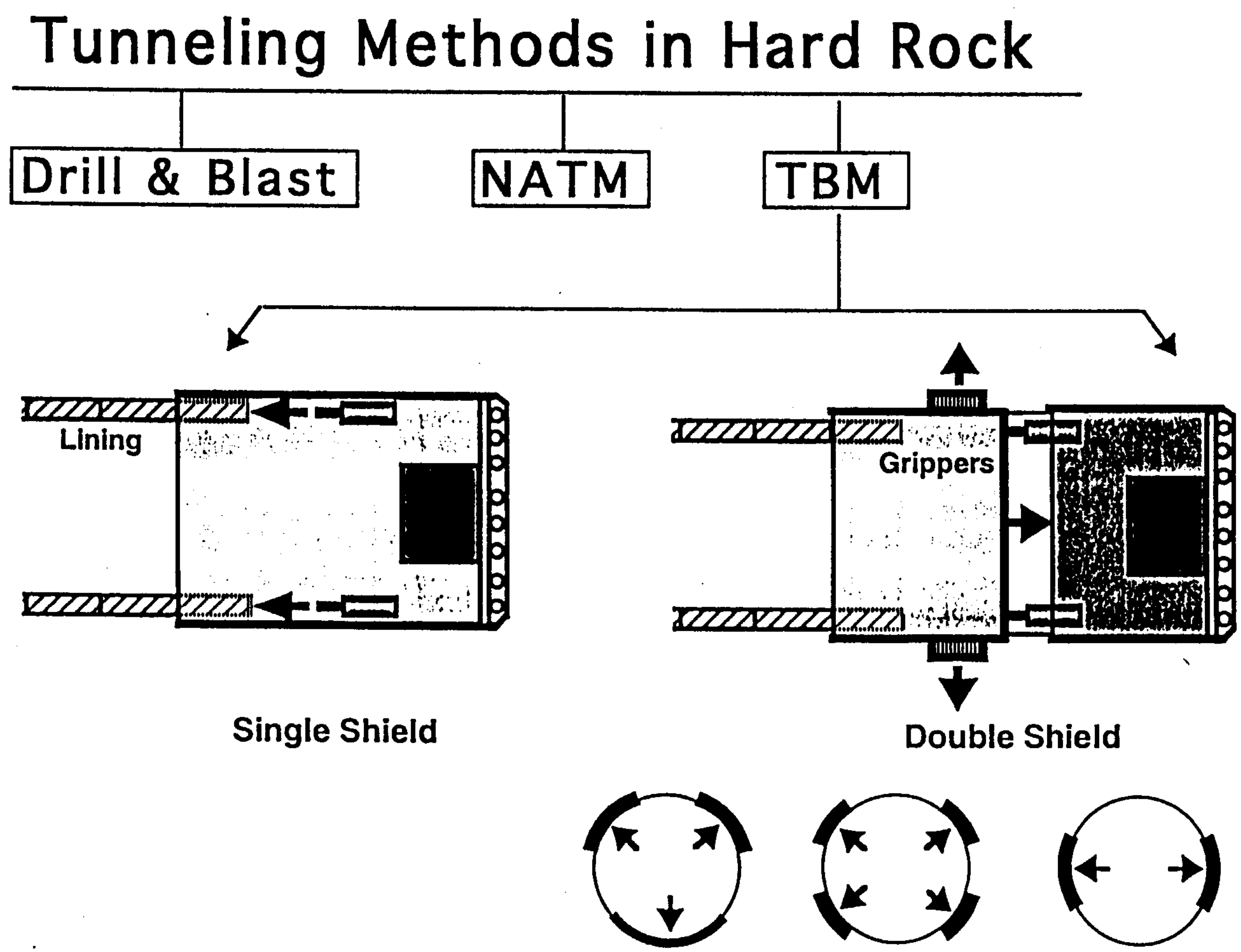


FIG. 3.2

For: Los Angeles County Metro Rail Project

By: Geotechnical Panel

Figure 3.2

Tunneling Methods in Hard Rock

November 1995

W-7101-01

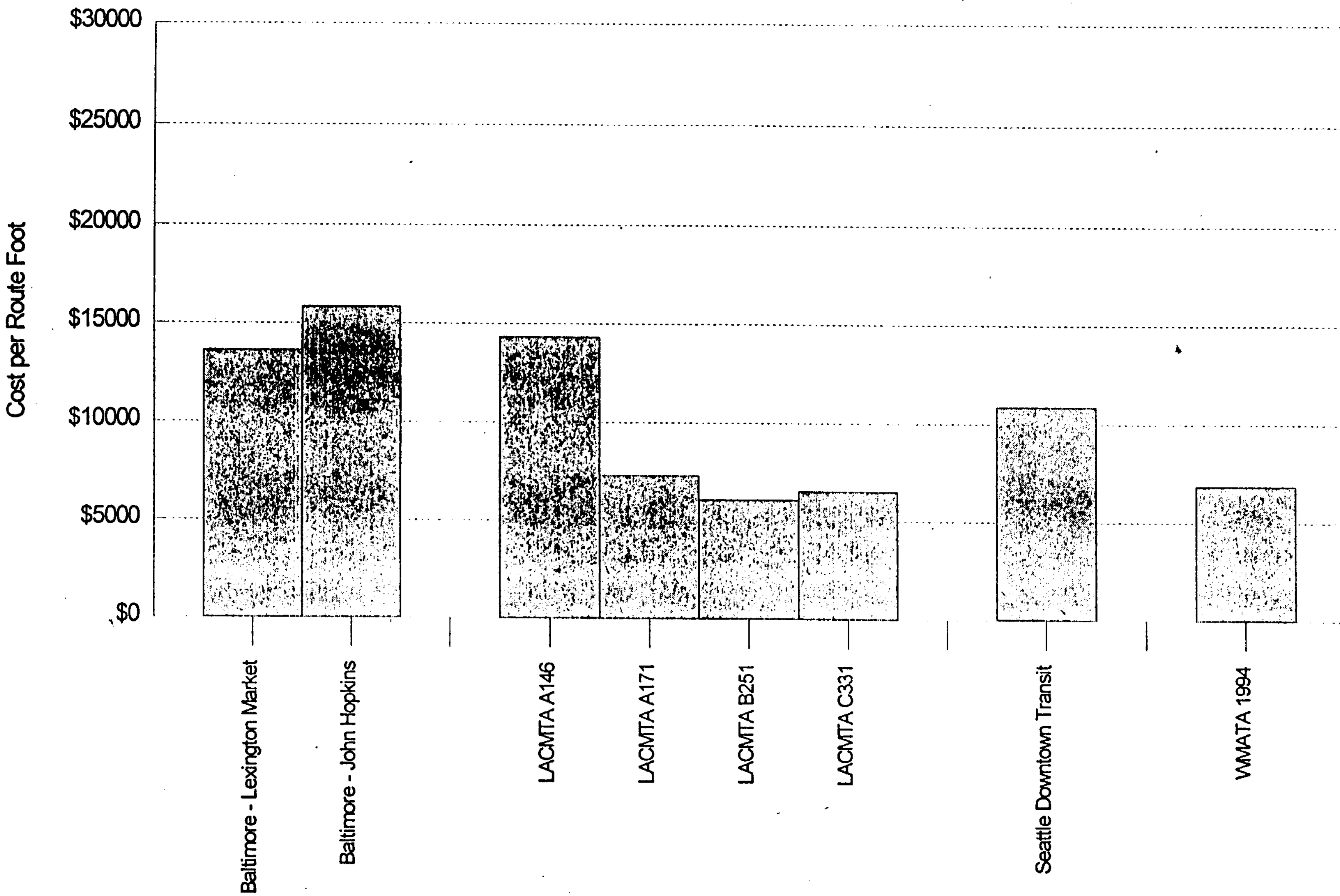


FIG. 6.1

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 6.1** **Unit Costs for Selected Soft Ground Tunnels**	November 1995 W-7701-01

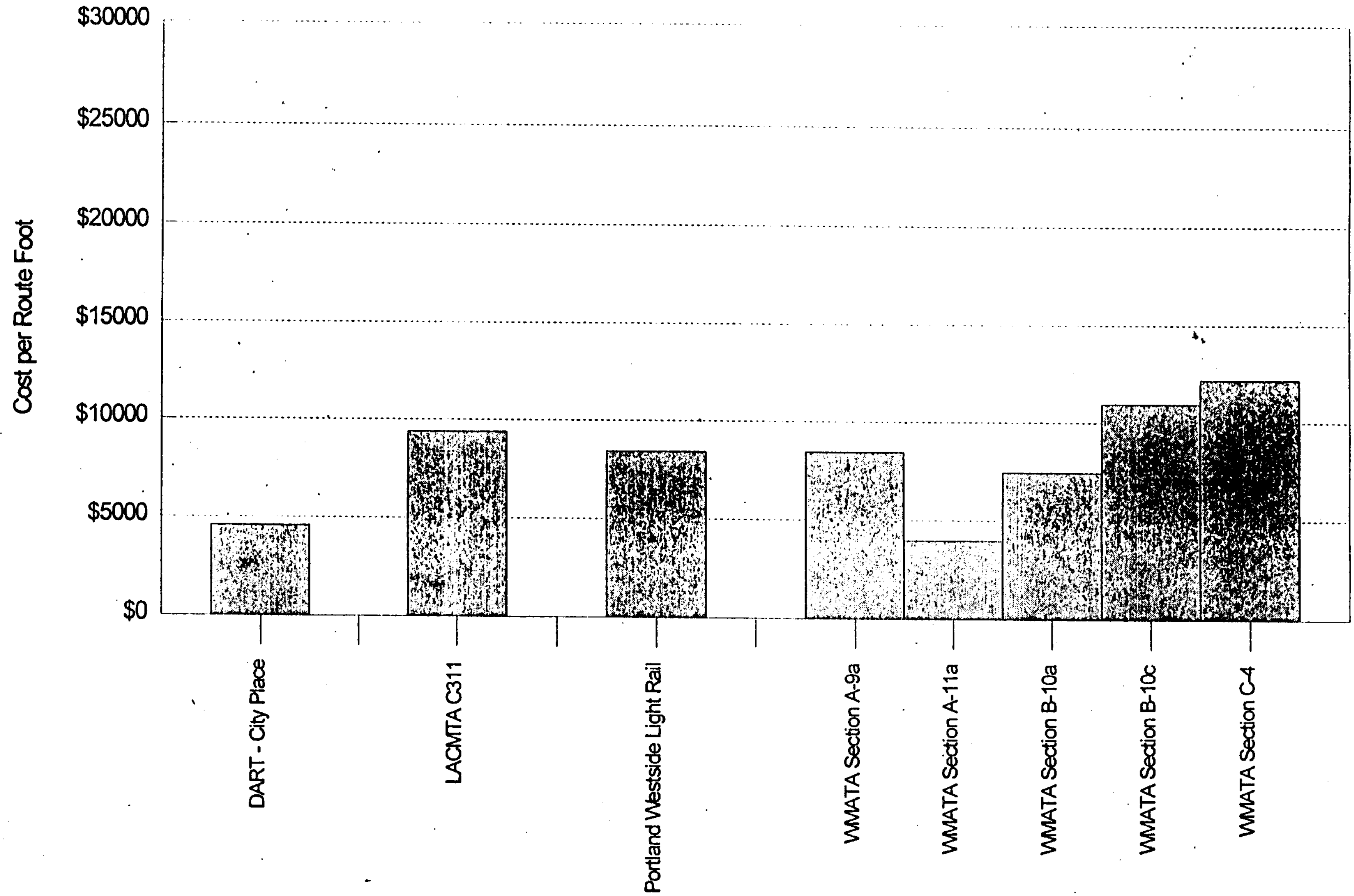
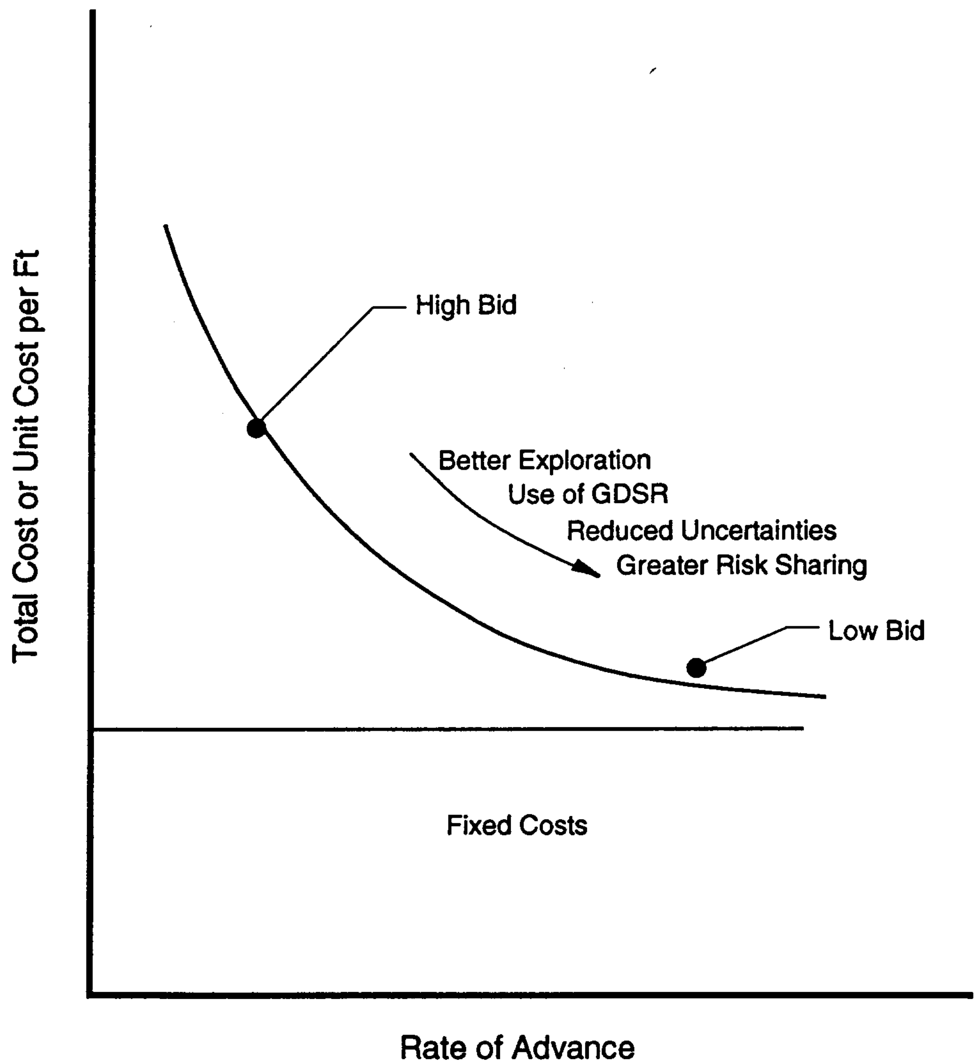
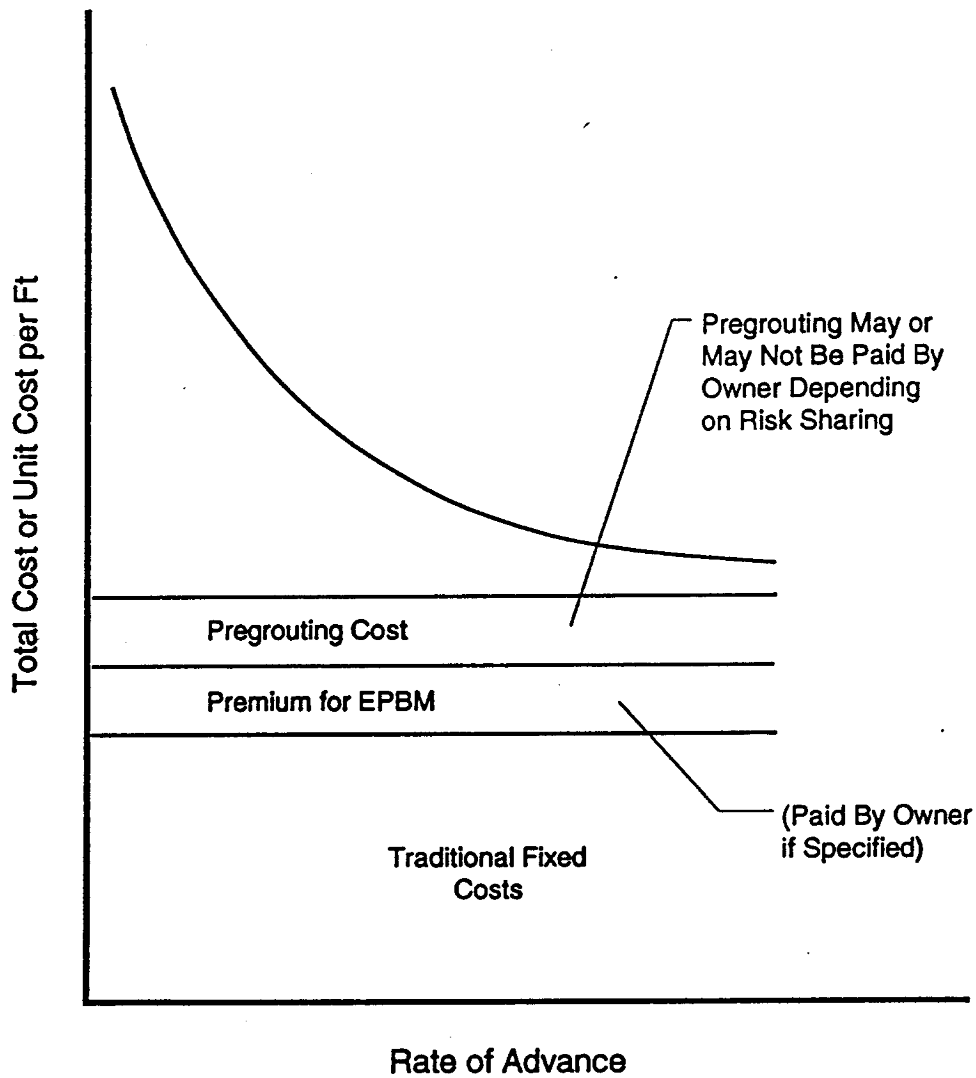


FIG. 6.2

For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 6.2** **Unit Costs for Selected Rock Tunnels**	November 1995 W-7101-01



For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 6.3** **Effect of Rate of Advance on High and Low Bids**	November 1995 W-7101-01



For: Los Angeles County Metro Rail Project By: Geotechnical Panel	**Figure 6.4** **Effect of Improved Technology for Ground Control**	November 1995 W-7101-01