METRO RED LINE, SEGMENT 2, CONTRACT B251

Investigation of Tunnel Collapse and Sinkhole

Prepared for:

Los Angeles County Metropolitan Transportation Authority

Prepared by:

Wiss, Janney, Elstner Associates, Inc.

FINAL REPORT

October 17, 1995

WJE No. 951473

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Prepared for:

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EXECUTIVE SUMMARY

Introduction

During the early morning hours of June 22, 1995, large ground movements and seepage occurred as an 80-ft section of subway tunnel was being realigned along the south side of Hollywood Boulevard. At about 6:10 a.m., a portion of the tunnel collapsed, and a 15-ft deep sinkhole developed above the remined area between Barendo Avenue and Edgemont Street. Water from a broken 10-in. main accumulated in the sinkhole. At approximately 11:35 a.m., water and earth flushed eastward through the tunnel to the Barnsdall construction shaft.

At the request of the Los Angeles Country Metropolitan Transportation Authority (LACMTA), Wiss, Janney, Elstner Associates, Inc. (WJE) investigated the cause of the tunnel collapse and sinkhole.

Background

Contract Unit B251 includes construction of the Metro Red Line twin tunnels below Vermont Avenue and Hollywood Boulevard. The north and south tunnels along Hollywood Boulevard are referred to as HAR and HAL, respectively. Similarly, the east Vermont tunnel is called VAR and the west Vermont tunnel is called VAL.

The tunnels were designed by Engineering Management Consultants (EMC), an association of engineering companies including Parsons, Brinckerhoff, Quade & Douglas, Inc. and Daniel, Mann Johnson & Mendenhall. Contract B251 was awarded to the joint venture firm of Shea-Kiewit-Kenny (SKK). The Los Angeles County Metropolitan Transportation Authority oversees the construction. The joint venture firm of Parsons-Dillingham (P-D) is responsible for construction management.

As required by the contract specifications, the initial tunnel support consists of precast concrete segments. The inside diameter of the tunnel liner is 20 ft 5 in. After the initial support was completed, as-built surveys indicated that two sections of the liner were horizontally misaligned: an 80 ft section of the HAL tunnel between Barendo Avenue and Edgemont Street, and a 164 ft section of the VAL tunnel between Fountain and De Longpre Avenues. Remining operations were considered necessary to maintain the design track alignment.

The remining procedure used to correct the misalignment in the two locations consisted of a heading and bench operation. The heading operation included removing the upper quadrant (12:00 o'clock to 3:00 o'clock portion) of the segmental concrete liner, excavating the exposed earth to accommodate the realignment, installing the upper steel sets, and placing lagging between the steel sets and the exposed earth. The steel sets were temporarily supported by foot blocks resting on the unexcavated bench. A concrete wall beam was placed at the bottom of the steel sets after several steel sets had been placed.

After completion of the heading along the entire length of a remined area, the benching operation began. It consisted of removing the lower quadrant of the segmental concrete liner, excavating the exposed earth, installing the lower steel sets, and setting the lagging. Concrete was placed against the lagging around the steel sets to reestablish the support afforded by the concrete liner.

One hundred and sixty-four ft of the VAL segment were successfully remined and resupported during September 1994. Remining the HAL tunnel began on June 15, 1995. On June 22, as the heading was nearing completion, seepage was observed in the tunnel and the foot blocks for the upper steel sets settled 12 to 18 in. As described above, the tunnel subsequently collapsed and the sinkhole developed.

<u>Subsurface Conditions</u>

The conditions at the two remined areas were similar in that the material excavated was primarily siltstone and claystone of the Puente formation. The original tunneling through the two areas proceeded smoothly with no observations of water in the face. However, the Puente formation at the HAL site was more bedded and jointed than that at the VAL site, and included bedding planes that dipped 65 to 70 degrees as compared to the 10 degree dip at the VAL site. After tunneling had loosened the joints, the increased permeability along the joints would provide a more freely flowing pathway from the overlying perched water to the exposed rock in the crown during remining operations, than existed during the tunneling operations.

Groundwater conditions during tunnel construction and remining operations in this area can be inferred from water levels in nearby observation wells. After the water levels near the HAL site had dropped from the effects of a dewatering system at Edgemont Street, groundwater rise and subsequent gradual fall were observed. The two rapid rises lag behind heavy rainfalls in early 1994 and 1995. These water level changes are thus in response to recharge from precipitation, not a leak in the water main. In June 1995, the water level above the HAL remined area was computed to be 24 ft above the crown of the tunnel. When the VAL tunnel was remined, the water level was computed to be at most 20 ft above the crown of the tunnel.

With an estimated 24 ft of water head and more pervasive bedding at the HAL site, seepage forces were able to develop and load the steel sets at the HAL site. While the estimated levels of water at the VAL site may have been similar to those at the HAL site during remining (the water levels may have been lower), the less pervious and more horizontal bedding resulted in conditions where seepage did not develop at the exposed ground during remining.

<u>Design of Resupport System</u>

The alignment repair submittals were prepared by SKK and submitted in February 1994 for approval by EMC. The first submittal was rejected by EMC. Among other things, EMC requested the precast concrete segment design load be compatible with that used in sizing the steel sets. Also, SKK was asked to provide foot plate design and bearing capacity calculations. SKK resubmitted the alignment repair procedure and calculations in May 1994. The loads used for the temporary steel sets were the same in the resubmittal and corresponded to a load on each segment of 60 ft of overburden, or 7200 psf. This design pressure develops an axial load of 307 kips in each steel set. Based on the design load of 60 ft of soil, a bearing pressure of 4878 psi on the foot block was calculated. Bearing capacity calculations were not provided, however. The second submittal was approved.

The approach used to design the support system apparently was to size the temporary structural support elements for a load corresponding to 60 ft of overburden pressure, and to rely on the inherent strength of the Puente formation to temporarily support the steel sets as the heading was made. A scenario which resulted in small loads on the foot blocks was reasonable for remining in "dry" Puente formation based on the response of the Puente during the tunneling operations for the four Vermont and Hollywood tunnels. One can not design the supports for all stages of temporary construction without recognizing the benefits of arching within the Puente formation. However, when considering the arching, it is prudent to include a component of earth loading in the design of the foot blocks.

Evaluating the bearing capacity of the earth on which a foot block rests is a standard step in design of rib linings. Safe support of the foot blocks is essential during the heading operation. The results of the bearing capacity calculations indicate the ultimate load which each steel set can support varies from 9 to 23 kips, about equal to the load expected from seepage forces and nominal earth load, and much less than the 307 kip load corresponding to 60 ft of overburden pressure. The design of the resupport system was deficient in that the bearing capacity of the foot blocks apparently was never considered. While it is unrealistic to require the foot blocks to carry 60 ft of overburden pressure, it is also unrealistic to *design* the foot blocks without provisions for earth loading.

<u>Sequence of Events Leading to Development of the Sinkhole</u>

Work on the HAL remining area began on June 15, 1995. The exposed ground was dry by the end of work on June 21, when all but one precast segment was removed. As the last segment was being prepared to be removed on June 22, SKK personnel first observed water seeping at 12:30 a.m. between the lagging near the west end of the remined area. Soon after the appearance of water, nearby foot blocks sets were observed to have settled into the unexcavated bench. By approximately 2:30 a.m., the foot blocks for all steel sets had begun to settle and the concrete wall beam was settling at its west end. The area of seepage had grown to about 20 ft along the tunnel axis.

Personnel at the site believed the source of the seepage was a leak in the 10-in. main above the tunnel. At about 3:00 a.m., P-D personnel notified the LADWP Trouble Board that there was a possible water main break and requested that a crew be immediately dispatched. A crew from LADWP arrived on the site at 3:40 a.m., but not seeing water at the ground surface, left without shutting off the water. This investigation indicates that a preexisting leak is unlikely. Had a leak occurred, it would have made only a small contribution to the groundwater in the area. The ground water level was already well above the tunnel crown due to heavy rainfall in the preceding months. Therefore, the possible presence of a preexisting leak in the water main is not relevant.

As the foot blocks settled, the rock in the crown began to ravel. As the ravelling progressed, more load was transferred to the steel sets and remaining portions of the saw-cut concrete segments. These movements of the foot blocks resulted in a flattening of the crown of the tunnel, and would have resulted in settlements at the ground surface. The surface settlement induces stress in the water main, eventually causing the pipe to rupture. The water main probably broke before the collapse of the tunnel supports since the collapse reportedly occurred at 6:10 a.m. and the LADWP records indicate the water main ruptured 10 minutes earlier. Apparently, the sudden influx of water at a rate of about 14 cfs directly lead to the first collapse at 6:10 and development of the sinkhole at 6:15. Water and earth from this first collapse flowed about 260 ft west into the HAR tunnel. Had the water main been shut off before it ruptured, damage may have been limited to large ground movements in the tunnel and resulting surface subsidence.

After this first collapse, water from the broken main filled the resulting sinkhole for at least 45 minutes until the main was shut off. A gas leak was reported by the Gas Company at about 8:00 a.m. The Los Angeles Fire Department directed the Gas Company to shut down the main gas line, which was done at 2:45 p.m. Until the gas line was shut off, SKK was not allowed to pump water out of the hole. At about 11:35 a.m., a second collapse of the remined area occurred which resulted in water and earth flowing eastward from the remined area through the Barnsdall Shaft and into the Vermont tunnels.

Conclusions

The tunnel failure is due to a deficient design of the temporary support for the steel sets. This design deficiency became apparent when seepage within the Puente formation was encountered during remining. Minor variations from the approved remining procedure occurred which did not significantly affect the performance. Flow from the 10-in. water main did not contribute to the initial foot block failure, but damage would have been significantly reduced if the water main were shut off earlier.

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October 17, 1995

METRO RED LINE, SEGMENT 2, CONTRACT B251

INVESTIGATION OF TUNNEL COLLAPSE AND SINKHOLE

Prepared for

LOS ANGELES COUNTY METROPOLITAN TRANSPORTATION AUTHORITY

LOS ANGELES, CALIFORNIA

WJE No. 951473

CHAPTER 1 - INTRODUCTION

During the early morning hours of June 22, 1995, large ground movements and seepage occurred as an 80-ft section of subway tunnel was being realigned along the south side of Hollywood Boulevard. At about 6:10 a.m., a portion of the tunnel collapsed, and a 15-ft deep sinkhole developed above the remined area between Barendo Avenue and Edgemont Street. Water from a broken 10-in. main accumulated in the sinkhole. At approximately 11:35 a.m., water and earth flushed eastward through the tunnel to the Barnsdall construction shaft.

At the request of the Los Angeles Country Metropolitan Transportation Authority (LACMTA), Wiss, Janney, Elstner Associates, Inc. (WJE) investigated the cause of the tunnel collapse and sinkhole. WJE was asked to perform the following tasks:

- Review the tunnel remining plan developed by the Contractor and approved by the Engineer, to determine the adequacy of the plan given soil conditions at the site of the incident.
- Determine what soil information was available to the contractor when the remining plan was developed.
- Determine the groundwater conditions at the site both prior to and during the remining operation.
- Provide a chronology of events leading to the subsidence and subsequent release of water into the tunnel.

- Determine whether the Contractor was following the approved remining plan at the time of the incident.

These tasks were carried out under Contract LFA-477-96 dated July 13, 1995, between WJE and LACMTA. The work performed included review of relevant construction documents and geological records, interviews with design and construction personnel familiar with the incident, and geotechnical and structural analyses. This report describes the investigation, discusses the results and summarizes the findings. Background information is provided in Chapter 2. A review of the relevant construction documents is given in Chapter 3. Chapter 4 describes the sequence of events leading to the tunnel collapse and sinkhole development. Geotechnical and structural analyses are described in Chapter 5. Chapter 6 discusses the findings, and the conclusions are summarized in Chapter 7. Figures are provided at the end of each chapter.

CHAPTER 2 - BACKGROUND

Contract Unit B251 includes construction of the Metro Red Line twin tunnels below Vermont Avenue and Hollywood Boulevard (Fig. 2.1). Each of the twin tunnels along the Vermont Avenue segment are about 14,200 ft long and extend from the southeastern end of the Barnsdall access shaft (station 460+05) south along Vermont Avenue to the Wilshire/Vermont station (station 317+95). The twin tunnels along the Hollywood Boulevard segment are each about 17,600 ft long. Starting from the northwestern end of the Barnsdall shaft (station 461+50), they continue west along Hollywood Boulevard, turning north to the foothills of the Santa Monica mountains (station 599+83).

The north and south tunnels along Hollywood Boulevard are referred to as HAR and HAL, respectively. Similarly, the east Vermont tunnel is called VAR and the west Vermont tunnel is called VAL.

The tunnels were designed by Engineering Management Consultants (EMC), an association of engineering companies including Parsons, Brinckerhoff, Quade & Douglas, Inc. and Daniel, Mann Johnson & Mendenhall. Contract B251 was awarded to the joint venture firm of Shea-Kiewit-Kenny (SKK). The Los Angeles County Metropolitan Transportation Authority oversees the construction. The joint venture firm of Parsons-Dillingham (P-D) is responsible for construction management.

Tunnel profiles generally parallel the ground surface with local adjustment at underground utilities and other obstacles. The depth of the ground cover over the tunnel crown in the Vermont Avenue segment ranges from approximately 29 to 92 ft. Depth of the ground cover over the tunnel crown along the Hollywood Boulevard segment ranges from approximately 35 ft to over 100 ft at the far north end of the contract. Within the Vermont Avenue segment, the tunnel excavation is mostly within the Puente formation, a soft rock formation consisting primarily of beds of claystone, siltstone and sandstone. Subsurface materials along the Hollywood Boulevard vary considerably. Near the Barnsdall shaft, tunnels are located within the Puente formation. To the west under Hollywood Boulevard, subsurface conditions at the tunnel elevations consist mainly of sand, silt and clay soils derived from young alluvium and old alluvium deposits. Rock is encountered as the tunnels approach the Santa Monica mountains at the northern terminus of Contract B251.

As required by the contract specifications, the initial tunnel support consists of precast concrete segments. The inside diameter of the tunnel liner is 20 ft 5 in. The segments are nominally 8 in. thick by 4 ft wide and form an arc with an included angle of 88 degrees. The invert segment is erected first, followed by the two side segments and finally the crown segment. There are two expansion gaps, approximately 9 in. wide, located at the 10:30 and 1:30 clock positions.

After the initial support was completed, as-built surveys indicated that the liner was constructed out of alignment in the locations noted in Figure 2.1 to the extent that remining operations were needed to maintain the design track alignment. The sections in the HAL and VAL tunnel were offset horizontally, and the corrective measures needed at these two locations were the same.

The remining procedure consisted of a heading and bench operation (Fig. 2.3). The heading operation consisted of removing the upper quadrant (12:00 o'clock to 3:00 o'clock portion) of the segmental concrete liner, excavating the exposed earth to accommodate the realignment, installing the steel sets, and placing lagging between the steel sets and the exposed earth. A concrete wall beam was placed at the bottom of the steel sets after several steel sets had been placed. After completion of the heading along the entire length of a remined area, the benching operation began. It consisted of removing the lower quadrant of the segmental concrete liner, excavating the exposed earth, installing the steel sets, and setting the lagging.

One hundred and sixty-four ft of the VAL segment between stations 441+05 and 439+41 were successfully remined and resupported during September 1994. Remining the HAL tunnel between station 464+34 and 465+14 (AL stationing) began on June 15, 1995. During the early morning hours of June 22, large ground movements and seepage occurred near station 465 as the heading was nearing completion. A portion of the HAL tunnel collapsed at approximately 6:10 a.m. (Fig. 2.2). A sinkhole, approximately 15 ft deep, developed at the ground surface as earth and water ran westward through the HAL tunnel from the collapse site. After the collapse, water from a broken 10 in. diameter water main poured into the sinkhole until the water main was shut off (Fig. 2.4). At approximately 11:35 a.m. the same day, the ponded water caused an additional collapse, resulting in earth, debris and water to be pushed eastward through the Barnsdall Shaft and into the Vermont Tunnels.

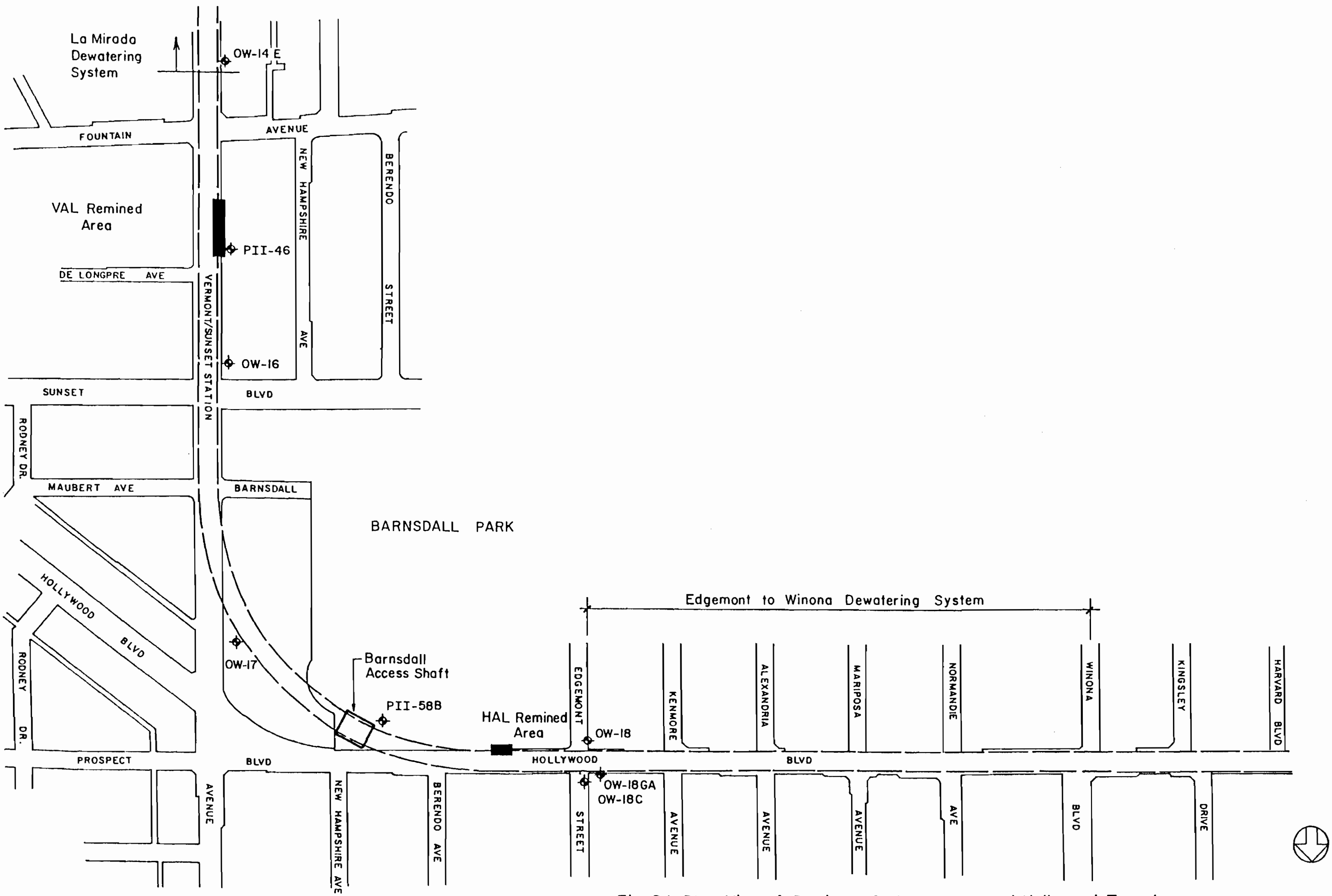

Fig. 2.1 Plan View of Portions of The Vermont and Hollywood Tunnels

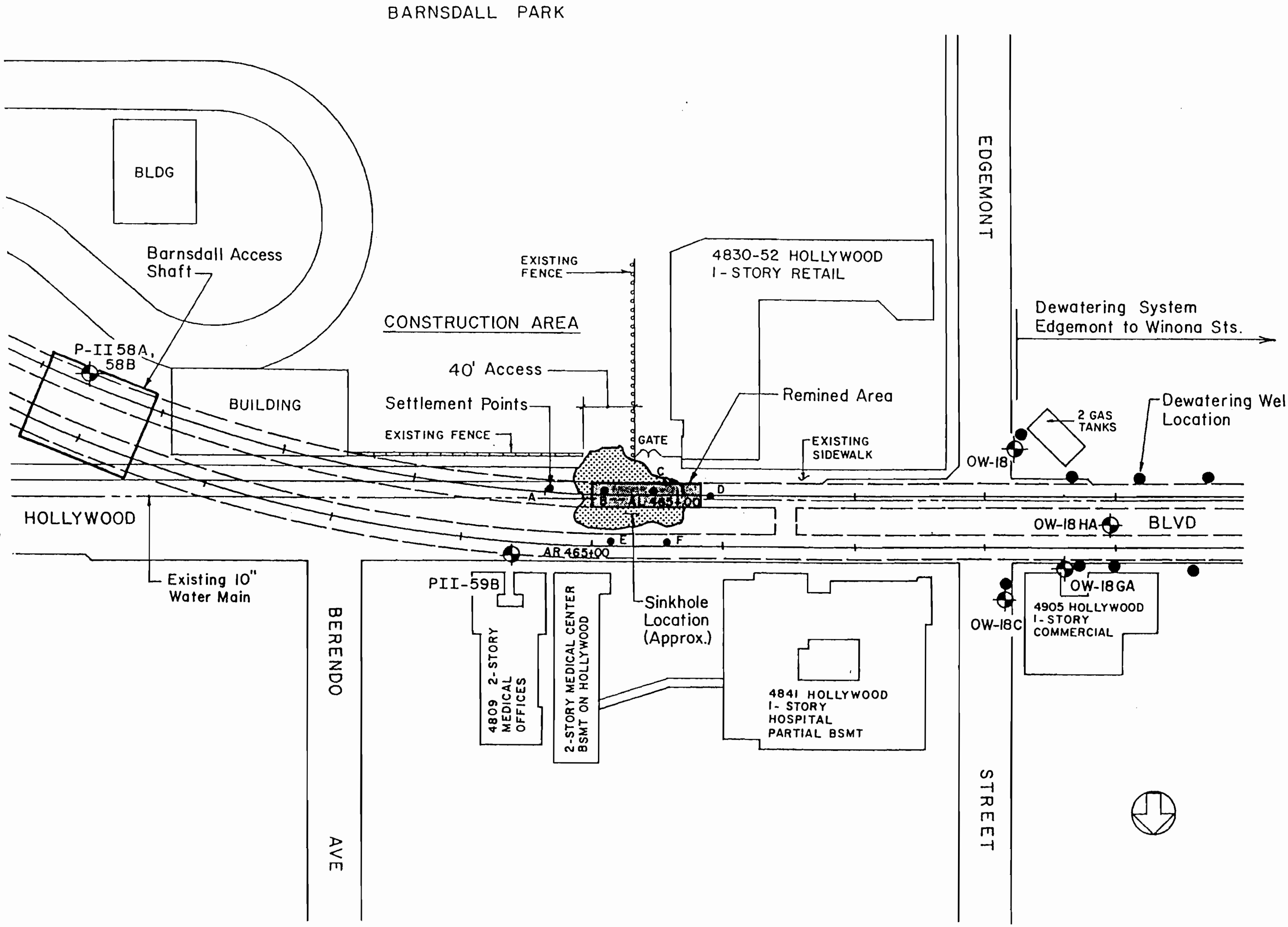

Fig. 2.2 Plan View of HAL Remined Area

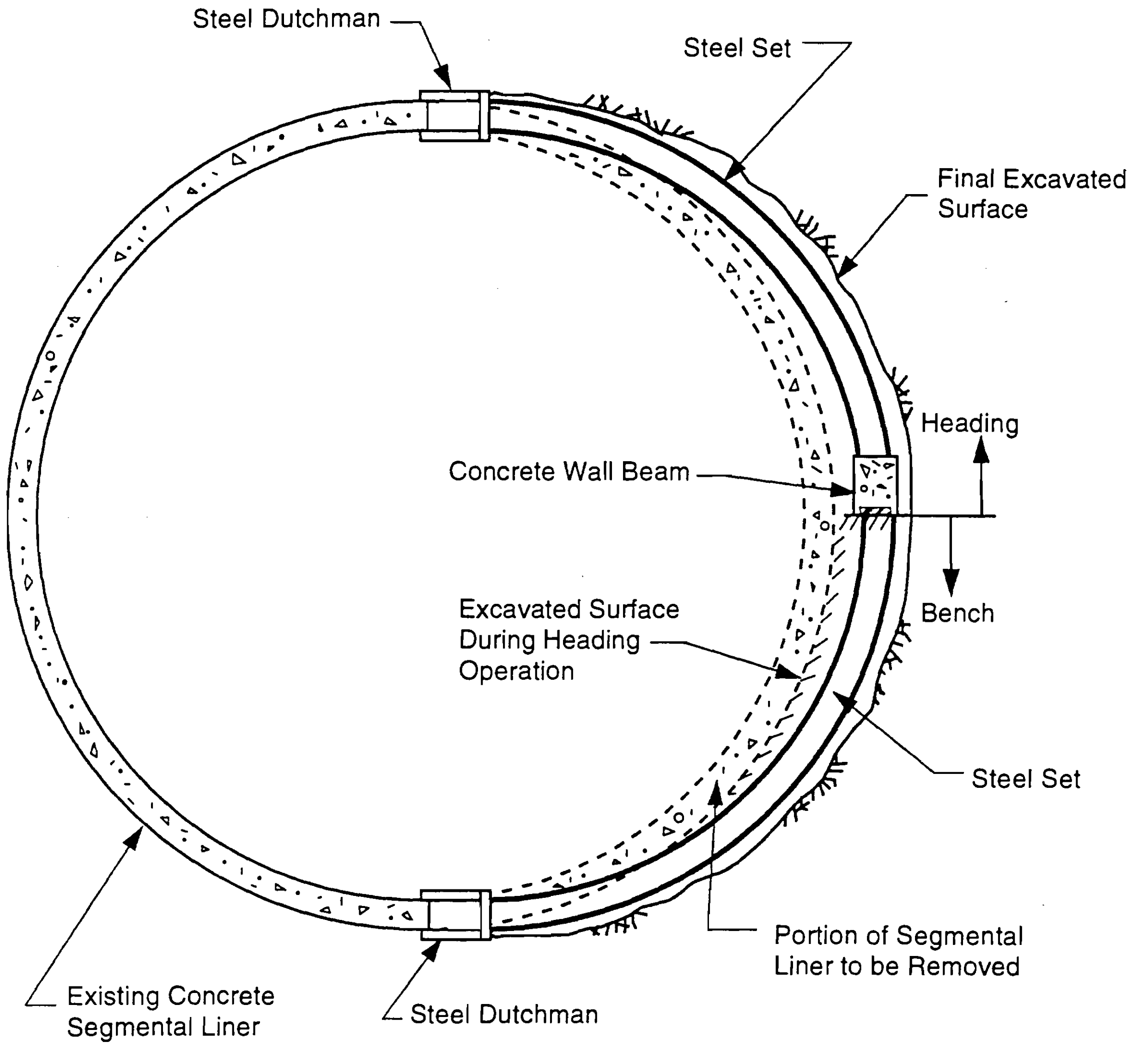

Figure 2.3 Temporary Support for Remined Area

Fig. 2.4 - Water main pouring into sinkhole above tunnel collapse

CHAPTER 3 - DOCUMENT REVIEW

WJE was provided with documents which pertained to the remining operations and tunnel collapse including (1) relevant submittals, (2) project correspondence, (3) daily reports of construction of the HAL and VAL tunnels at the remined sections (both when the tunnels were first constructed and during the remined period), (4) face sketches of the tunnels in the remined areas, (5) pertinent geotechnical reports, (6) groundwater data, and (7) water supply documentation from the Los Angeles Department of Water and Power (LADWP). The following sections summarize observations from review of these documents which are considered relevant to this investigation.

3.1 Alignment Repair Submittals

WJE reviewed the alignment repair submittals sent by SKK to P-D for approval by EMC. There were two such submittals: the initial submittal was received by P-D on February 24, 1994, and the revised was transmitted to P-D on May 16, 1994. These are included in Appendix A. The February submittal included six pages of shop drawings, a sketch of the support proposed to replace one-half the existing concrete lining (steel ribs and wood lagging), and four pages of calculations concerning anticipated loadings on steel ribs when the remining was located within the alluvium or the Puente formations, the allowable load on the steel sets based on structural considerations, wall beam calculations assuming a 48 in. maximum span, and the allowable load for wood lagging.

This submittal was rejected by EMC. Among other points, EMC requested (1) the precast concrete segment design load be compatible with that used in sizing the steel sets, (2) foot plate design and bearing capacity calculations, and (3) detailed descriptions of the excavation sequence, intermediate support details and face stability methods. EMC further noted that the details are only appropriate for sections wholly within the Puente formation. Groundwater conditions were not addressed, and no distinction was made among the weathered, oxidized and fresh portions of the Puente formation.

The May resubmittal included a cover letter from Robert B. Gordon dated May 16, 1994, a written description of the construction sequence, five pages of drawings and seven pages of calculations. The calculations included three of the original pages and one revised sheet for the wall beam calculations with a 6 ft span, a sheet with calculations showing the foot plate bearing pressure for 60 ft of overburden, a sheet with structural calculations for the concrete wall beam, and a sheet with sketches showing the temporary loading conditions. This submittal was approved by EMC.

According to the written description in this submittal, the proposed remining and resupport operations consisted of the following steps, as illustrated in Figure 3.1:

1. Install two rock anchors in each arch segment on the tunnel side that does not get removed.
2. Sawcut and remove upper quadrant of tunnel precast concrete segment.
3. Install steel dutchman, upper steel set and lagging. Install one split set stabilizer (rock bolt) through the steel dutchman. Only one segment was removed at any time without steel sets being installed: i.e. only 6 ft of tunnel was to be unsupported at any time - the 4 ft segment plus the 2 ft to the center of the steel set.
4. Repeat steps 2 and 3 until entire upper heading is complete.
5. Cast concrete wall beam with 5000 psi concrete.
6. Sawcut and remove lower quadrant of tunnel precast concrete ring.
7. Install lower steel quadrant post and lagging. Only one segment was removed at any time without steel sets being installed, i.e. only 6 ft of tunnel was to be unsupported at any time.
8. Repeat steps 6 and 7 until entire lower bench is complete.

The loads used for the precast segments and the temporary steel sets were the same in the resubmittal and corresponded to a load on each segment of 60 ft of overburden, or 7200 psf. This design pressure develops an axial load of 307 kips in each steel set. The calculations indicated that the anticipated load on each segment in the Puente formation was equivalent to 15.1 ft of overburden, or 1888 psf, with a corresponding axial load of 81 kips in each steel set. Based on the design load of 60 ft of soil, a bearing pressure of 4878 psi on the foot block apparently necessitated the use of 5000 psi concrete. This wall beam carried load temporarily, to distribute loads from the steel sets to the underlying ground after it was placed in step 5, and to span the bench when soil was being removed prior to placing a steel set in step 6. This latter requirement implies that the bench is rigid. This assumption appears to be unrealistic given the strength and stiffness of the weathered and oxidized Puente formation, as discussed subsequently.

No calculations were available for review by WJE which evaluated either the bearing capacity of the concrete wall beam or the bearing capacity of the foot blocks before the concrete beam was placed, steps 3 through 6 in Fig. 3.1. These temporary conditions were not considered in the calculations submitted for the proposed alignment repair.

3.2 Subsurface Conditions at Remined Areas in HAL and VAL Tunnels

When SKK prepared their initial and modified plans for remining in February and May 1994, respectively, the following information was known concerning the conditions at the remined areas. The only exceptions are the ground water and settlement data obtained after this time.

3.2.1 Stratigraphy near HAL remined area - Based on borings presented in the Geotechnical Report for the Vermont/Sunset Station and Adjacent Tunnel Segments, May 1990, prepared by The Earth Technology Corporation, hereafter referred to as the GRV/S Report, the subsurface conditions at this location consisted of approximately 45 ft of Young and Old Alluvium overlying the Puente formation (Fig. 3.2). A P-D drawing which summarized the face records from construction of the HAL tunnel provided the detail shown in this figure at the tunnel elevation. The crown of the tunnel was about 65 ft below the ground surface. Prior to construction, perched groundwater levels were located in the Alluvium. The tunnel in the sinkhole area was located entirely within the Puente formation. The material was easily excavated and, according to the face records (see Appendix B), consisted of weathered Puente (T_{pw}), oxidized Puente (T_{po}) and fresh Puente (T_{pf}), with an increasing amount of T_{pw} encountered in the face as the excavation proceeded westward. The T_{pw} was described in the face records as consisting of friable, interbedded siltstone/claystone with fine-grained sand beds, less than 1 in. thick. The apparent dip of the bedding was as much as 65 to 70 degrees from the horizontal. The T_{pw} was jointed, highly to moderately sheared, and slickensided. The T_{pf} was distinguished from the T_{pw} primarily on the basis of color, olive gray rather than red or orange brown, and on strength, "weak" rather than "very weak."

3.2.2 Groundwater observations near HAL remined area - Groundwater conditions during tunnel construction and remining operations in this area can be inferred from water levels in nearby observation wells. Piezometer PII-58B and observation well OW-18, bound the HAL remined area, and were installed prior to tunneling (Fig. 3.2). Piezometer PII-58B is located about 400 ft from the east end of the remined area, whereas OW-18 is located about 260 ft from the west end of the remined area. Piezometer PII-58B provided data about levels of ground water prior to the start of tunneling near the remined section. No data was available from this piezometer during the tunneling or remining periods. OW-18C and 18 GA were installed near OW-18 as part of the dewatering system installed between July 1993 and January

1994. The dewatering system extending from Edgemont to Winona Streets was needed to control water levels during construction of the HAR and HAL tunnels. OW-18 is at the eastern edge of the Edgemont-Winona dewatering system.

These OW-18 series wells sense water levels at different elevations because of the details of their installation. OW-18 was screened from 11.5 to 90.5 ft and therefore sensed water within the alluvium as well as the Puente formation. OW-18C and 18GA were screened at depths from 75 to 90 ft below the ground surface; logs of these borings were not available for review by WJE, so the formations adjacent to the screened section are not known on this basis.

The temporal variations of groundwater levels near the HAL remined area are shown in Fig. 3.3. Data from observation wells OW-18, 18C and 18GA are presented. Also shown are the rainfall data recorded at the Hollywood Dam rain gage by the Los Angeles Department of Water and Power (LADWP). The OW-18 water levels show significant variations over time. After the water levels had dropped from the effects of the dewatering system at Edgemont Street, two episodes of rapid groundwater rise and subsequent gradual fall were observed. The other wells show no such variations. The two rapid rises in OW-18 lag behind the peak rainfall in both 1994 and 1995 by about 45 days. Barnsdall Park is the topographic high in the area. As noted in the GRV/S Report (1990), Barndall Park is the recharge area for the local ground water which tends to flow away from this area along the alluvium and Puente formation interface. OW-18 is screened throughout its entire depth and is therefore hydraulically connected to the alluvium. These water level changes are thus in response to recharge from precipitation. The other wells shown are all screened at greater depths and thus apparently are not hydraulically connected to the alluvium, or are screened in clay soils within the alluvium.

<u>3.2.3 Stratigraphy near VAL remined area</u> - Based on borings presented in the GRV/S Report (1990) and on the face records obtained during excavation of the VAL tunnel, the subsurface conditions at this location consisted of approximately 50 ft of predominantly fine grained Old Alluvium overlying the Puente formation (Fig. 3.4). A P-D drawing which summarized the face records from construction of the VAL tunnel provided the detail shown in this figure at the tunnel elevation. The crown of the tunnel was approximately 42 ft below ground surface. Prior to construction, perched groundwater levels were located in the Alluvium. The tunnel in the remined area was located within either the Puente formation or both the alluvium and Puente formations, depending on the interpretation of the materials

encountered in the heading. Based on logs of borings taken prior to construction, the tunnel was to have been excavated entirely within the Puente formation. Based on face records shown in Appendix B, the excavated material consisted of alluvial and colluvial fine grained soils and weathered Puente, T_{pw}. This latter interpretation is shown in Fig. 3.4. The alluvium was described as a reddish brown, medium stiff to stiff clay with low plasticity. The colluvium was described as the same soil as the alluvium, with inclusions of weathered Puente formation in quantities of 10 to 20%. The T_{pw} was described in the face records as consisting of very weak, highly weathered siltstone/claystone. The bedding was mostly horizontal, but dipped as much as 10 degrees. The bedding was difficult to distinguish because of the high degree of weathering. The difference in the two interpretations is primarily semantic in that the engineering properties are similar, i.e. low permeabilities except along discontinuities and strengths like very stiff to hard clays. In any case, the ground was observed ravelling from the crown as the shield advanced.

3.2.4 Groundwater observations near VAL remined area - Groundwater conditions during tunnel construction and remining can be inferred from water levels in several observation wells and piezometers in the area (Fig. 3.4). OW-16 is located about 430 ft from the north end of the remined area whereas OW-14E is located approximately 510 ft from the south end of the remined area. Piezometer PII-46 provided data about levels of ground water prior to the start of tunneling at the remined section. No data were available from this piezometer during the tunneling or remining operations. OW-14E and a number of other observation wells were installed as part of the dewatering system installed between Fountain and Lexington Avenues which was needed to control water levels in this area during construction of the VAR and VAL tunnels (Fig. 2.1).

OW-16 was screened from 83 to 104 ft and therefore sensed water within the Puente formation. Logs of the borings for the wells between Fountain and Lexington Avenues were not available for review by WJE, so the formations adjacent to the screened section are not known on this basis. According to P-D personnel, these wells, including OW-14E, are 90 ft deep with the last 15 ft consisting of the screened portion of the well. Given the depth of the granular nature of the alluvium in this area, these wells most likely sense the alluvium formation.

The temporal variations of groundwater levels in wells near the VAL remined area are shown in Fig. 3.5. Data from observation wells OW-14E and 16 are presented. The data from OW-14E is typical of the wells installed between Fountain and Lexington Avenues. Also shown are the data recorded at the Hollywood Dam rain gage by the LADWP. The OW-16 water levels vary over time, with two episodes of rapid groundwater rise after installation. The observation wells between Fountain and Lexington Avenues show no such variations and their responses are governed by the dewatering operation. The two rapid rises in OW-16 lag behind the peak rainfall in both 1994 and 1995 by about 60 days. As noted before, Barnsdall Park is the topographic high in the area, and is the recharge area for the local ground water which tends to flow away from the area along the alluvium and Puente formation interface. OW-16 is screened within the Puente, and it responds to the effects of recharge from precipitation. OW-16 is about 1200 ft away from the dewatering operation and is not affected to any great extent by the drawdown at that location, as indicated by the periods of slightly decreasing water levels observed as the dewatering system operated. It is somewhat surprising that OW-16 responded so quickly to the precipitation. The explanation may lie in the logs of the installation of OW-16 where it was noted that "intrusion problems and concern about pulling apart casing resulted in a 104 ft well depth." If the casing had slightly separated during installation, the well would be sensing water along its entire depth, because a gravel pack was placed above the bentonite seal. If that were the case, one would expect its response to precipitation to be similar to that of OW-18 since both wells are down gradient from the recharge area.

3.2.5 Permeability of the Puente formation - Based on data in the GRV/S Report (1990), the permeability of the Puente formation is anisotropic in that the coefficient of permeability in the vertical direction, k_v, is 20 to 100 times less than that in the horizontal direction, k_h. Slug tests conducted in piezometer PII-58A and B indicated that k_h is about 2×10^{-6} cm/s. Laboratory permeability tests indicated that k_v varied from 1×10^{-7} to 2×10^{-8} cm/s. This anisotropic behavior is attributed to the presence of bedding planes and sandstone beds which serve as preferred hydrological pathways. These pathways likely serve as hydrological connections between the Puente Formation and the overlying Alluvium.

3.2.6 Shear strength of Puente formation - Measures of strength of the Puente formation were found in the Geotechnical Design Summary Report (GDSR) for Contract B-251 Tunnels, the Vermont/ Hollywood Tunnel, Dec. 1991, and the GRV/S Report (1990). Data from samples obtained from borings located between stations 400+00 and 470+00 are summarized in this section. Within this reach, which contains the remined areas in the HAL and VAL tunnels, the strength data varied randomly with location along the alignment.

Table 3.1. Summary of Unconfined Compression Test Data

Material	Number of tests	Average S_u (ksf)	Range of S_u (ksf)
Weathered Puente, T_{pw}	1	0.95	–
Oxidized Puente, T_{po}	13	2.6	0.8 to 4.3
Fresh Puente, T_{pf}	6	6.5	1.1 to 8.8

The results of the 20 unconfined compression (U) tests on specimens obtained from the Puente formation are summarized in Table 3.1. The value of undrained shear strength, S_u, reported in the table is equal to one-half the unconfined compressive strength determined in the test. While there is a general trend of increasing strength with decreasing degrees of weathering, there is a good deal of overlap of the strength values based on geologic classification. The values of undrained strength based on the U tests generally are lower than the values in situ as a result of sample disturbance and lack of confinement. In soft rocks with secondary structure like the Puente, the lack of confinement tends to open joints and bedding planes which results in strengths being mobilized in the test which are lower than would be mobilized in the field.

The results of 10 isotropically consolidated, undrained triaxial compression tests on specimens obtained from the Puente formation are summarized on Figure 3.6. No tests were conducted on T_{pw} specimens. The data indicates that the laboratory values of S_u increase slightly with effective consolidation pressure, σ'_c, and are generally larger than those found in the U tests (Table 3.1), most likely as a consequence of the increased confinement in the triaxial tests. Again, no real distinction can be made between the strengths of the T_{po} and the T_{pf} specimens. To obtain a value of S_u for a given depth of cover, a value of S_u/σ'_c can be found, based on a slope through the data, and that ratio is multiplied by the corresponding vertical effective stress. Using the minimum value of S_u/σ'_c of 1.2, the corresponding value of S_u for the conditions at the HAL remining area is 7.4 ksf.

In the GDSR, the Earth Technology Corporation recommended S_u values of 1.5 ksf for T_{pw} and 5.0 ksf for both T_{po} and T_{pf}.

3.3 Nonconformance Reports Related to Grouting Through the Precast Concrete Liner

Several nonconformance reports (NCR) related to contact grouting of the initial support in the Hollywood and Vermont tunnels were filed by P-D prior to the sinkhole incident. These reports, filed in the October 1993 and September 1994, indicated that contact grout had not been placed between the precast concrete liner and the surrounding ground in accordance with specification 02311.1.,B.4. In particular, no contact grout was placed through any of the grout holes until well after the initial support had been placed at the locations of the remining in the HAL and VAL tunnels (eg. 11 months for the VAL tunnel). Therefore, the excavated rock would collapse onto the completed liner since the space between the excavated surface and segmental liner was not grouted. The outside diameter of the shield was 21 ft 10 in. and the theoretical diameter of the liner was 21 ft 9 in. The tail void gap thus is nominally 1 in., but in reality would be larger because of pitching of the shield, and the fact that the wood wedges were not installed such that the expansion gaps were 9 in. (See Structural Investigation of Wood Wedge Expansion Gap System, Metro Red Line, Segment 2 Report, Contract B251, prepared for LACMTA by WJE). Grout cannot be placed in a timely manner using this expansion gap system as-constructed since the grout, as it is pumped under pressure, would invade the tunnel through the space in the expansion gap between the wood wedges. This gap was in many places not constructed with dry pack grout. (WJE ref.) The NCRs were closed when the segments were eventually grouted.

3.4 Settlement Records

WJE reviewed ground surface settlement records provided by P-D for locations near the sinkhole. Ground surface settlements within several hundred ft of the sinkhole area were 1 to 1.5 in. prior to the development of the sinkhole on June 22, as illustrated by the data from survey points 48100004, 48100007, 48090014 and 48090021 on Fig. 3.7. Locations of these points are shown in Fig. 2.2, and are labeled A, D, E and F, respectively. These survey points are typical of those in the vicinity. The majority of these settlements occurred as the HAR and HAL tunnels were driven in June of 1993.

Two exceptions were observed at ground point 48100005 and 48100006, points B and C in Fig. 2.2, located above the HAL tunnel at the location of the sinkhole. As indicated in Fig. 3.7, surface settlements were similar to the typical settlements immediately after shield passage, but subsequent incremental drops of about 1.5 in. were observed prior to the sinkhole collapse. These movements did not occur at the same time, but developed at least 5, and as many as 10, months apart. More accurate definition is precluded by the paucity of readings taken at these points. Total settlements prior to the start of the remining operations were 2.7 and 2.6 in., respectively. These two anomalies may have been the result of very localized effects of water infiltration resulting in hydrocompression settlements, or heavy construction traffic, more likely in the case of point 48100005 since it was located in front of a construction gate at the Barnsdall shaft. The increased settlement at point 48100006 could also have been a bust in the survey data, because no additional data was obtained after the large incremental settlement was observed.

3.5 Documentation from LADWP

WJE was provided with water service map 148-198 covering the Barnsdall park area and records from the water supply monitoring station at Franklin and Kenmore Avenues. Also, Fred Barker and Jim Campbell of the LADWP were interviewed on July 21.

West of Vermont Avenue, water supply along Hollywood Boulevard is provided by a 10-in. cast-iron water main located about 20 ft north to the south curb. According to the LADWP water service map, the water supply line was installed in 1916 and lined with cement in 1992.

The 10-in. line along Hollywood Boulevard is interconnected with a network of supply lines at the intersecting streets. There are typically one or more shutoffs at each intersection; four valves would have to be closed to shut off water at the sinkhole site.

There are pressure and flow recorders at the Franklin and Kenmore monitoring station. During the low demand period from midnight to 5:00 a.m., flow is typically 6 to 8 cu ft per second (cfs) at this station. During peak demand periods from about 6:00 a.m. to 10:00 a.m., water usage usually increases to about 11 to 15 cfs. Service pressure (low-side pressure) is maintained at about 78 psi. Pressure in the supply lines to the regulator station (high-side pressure) varies from about 110 to 140 psi. There were no discernible changes in this pattern in the four months preceding the tunnel collapse. However, leaks are not detectable by the pressure and flow recorders. For example, a leak rate of 44 gallons per minute (gpm) changes the flow by only 0.1 cfs. According to LADWP personnel, water loss at leak sites typically ranges 5 to 30 gpm, and leaks are usually detected by the appearance of water on the street surface. Ground subsidence has been known to cause leaks.

The tunnel collapse and water main break were detected at the Franklin/Kenmore monitoring station. Pressure and flow chart records are provided in Figs. 3.8 and 3.9, respectively. At approximately 6:00 a.m. on June 22, flow increased from 10 to 24 cfs, a 14 cfs increase. The exact time is difficult to determine from the photocopy of the flow record. As can be seen in Fig. 3.8, low-side pressure dropped from 78 to 72 psi at 6:00 a.m. A sudden drop in the high-side pressure was recorded at about 5:45 a.m.

This 15 minute difference is apparently due to a required offset in multiple pen chart recorders. According to Fred Barker of the LADWP, the low-side pressure recorder is normally set to the correct time, and the original flow chart record shows the sudden flow increase occurring at exactly 6:00 a.m. Therefore, 6:00 a.m. is considered the best estimate of the time when the water main ruptured. However, data is recorded for one week on a 360° chart with 168 one-hour divisions. Therefore, the accuracy of the chart recorder is probably at best ± 5 minutes. At about 6:45 a.m., pressure and flow returned to normal. Apparently, the water to the sinkhole site was shut off at this time, although flow may have continued as water from the cutoff section of the main emptied into the sinkhole.

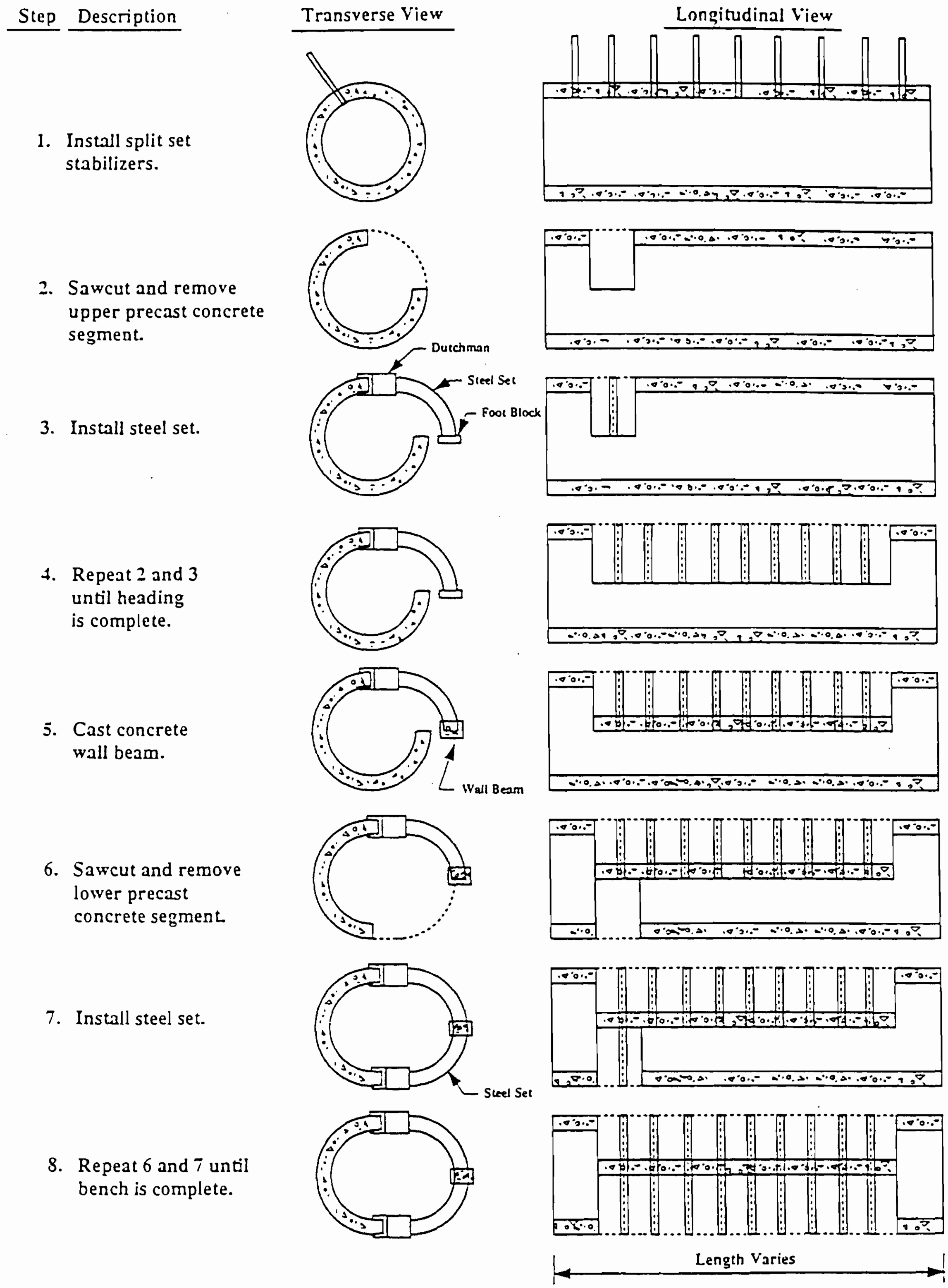

Fig. 3.1 Remining and Resupport Operations

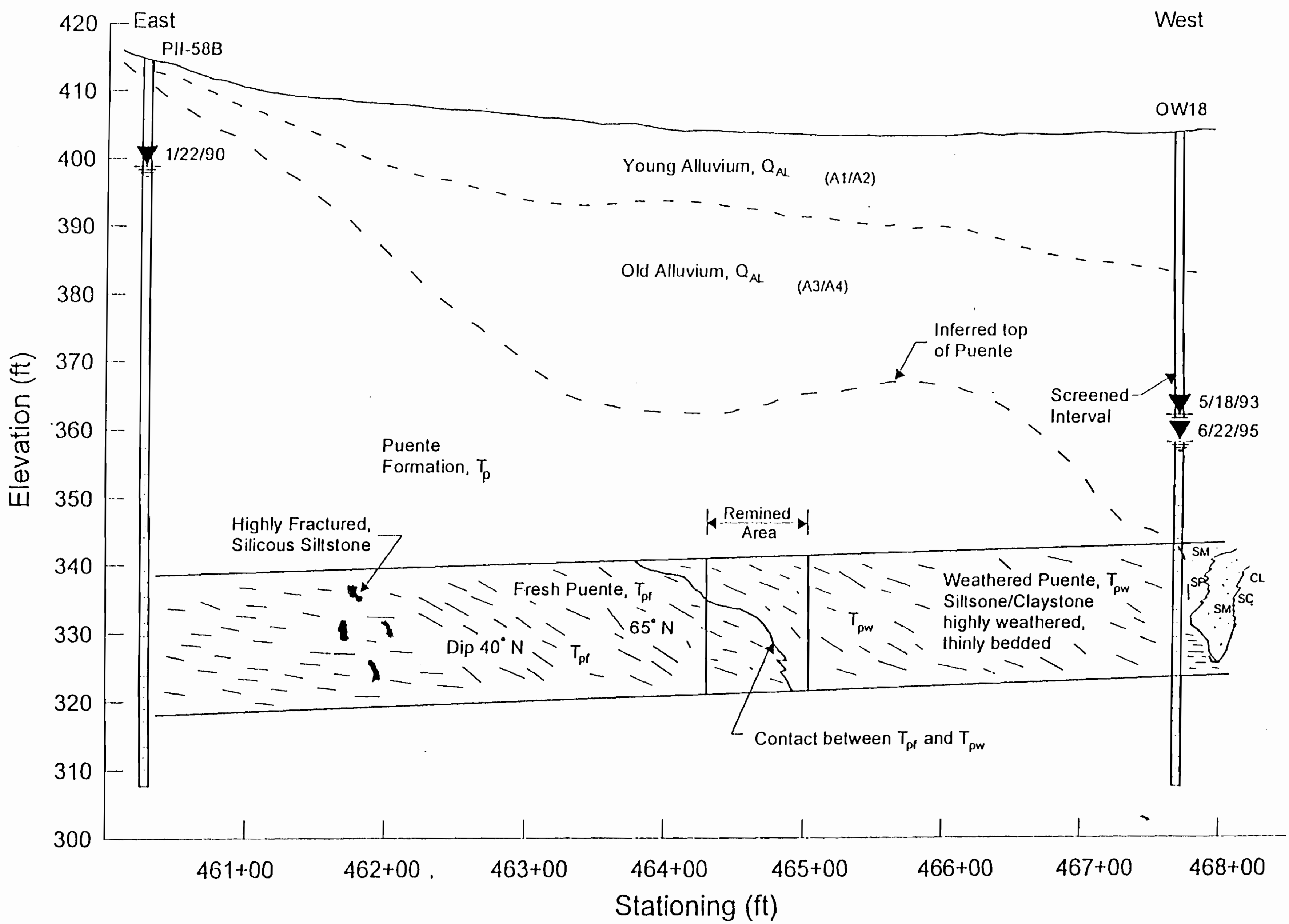

Fig 3.2 Subsurface Conditions Near HAL Remined Area

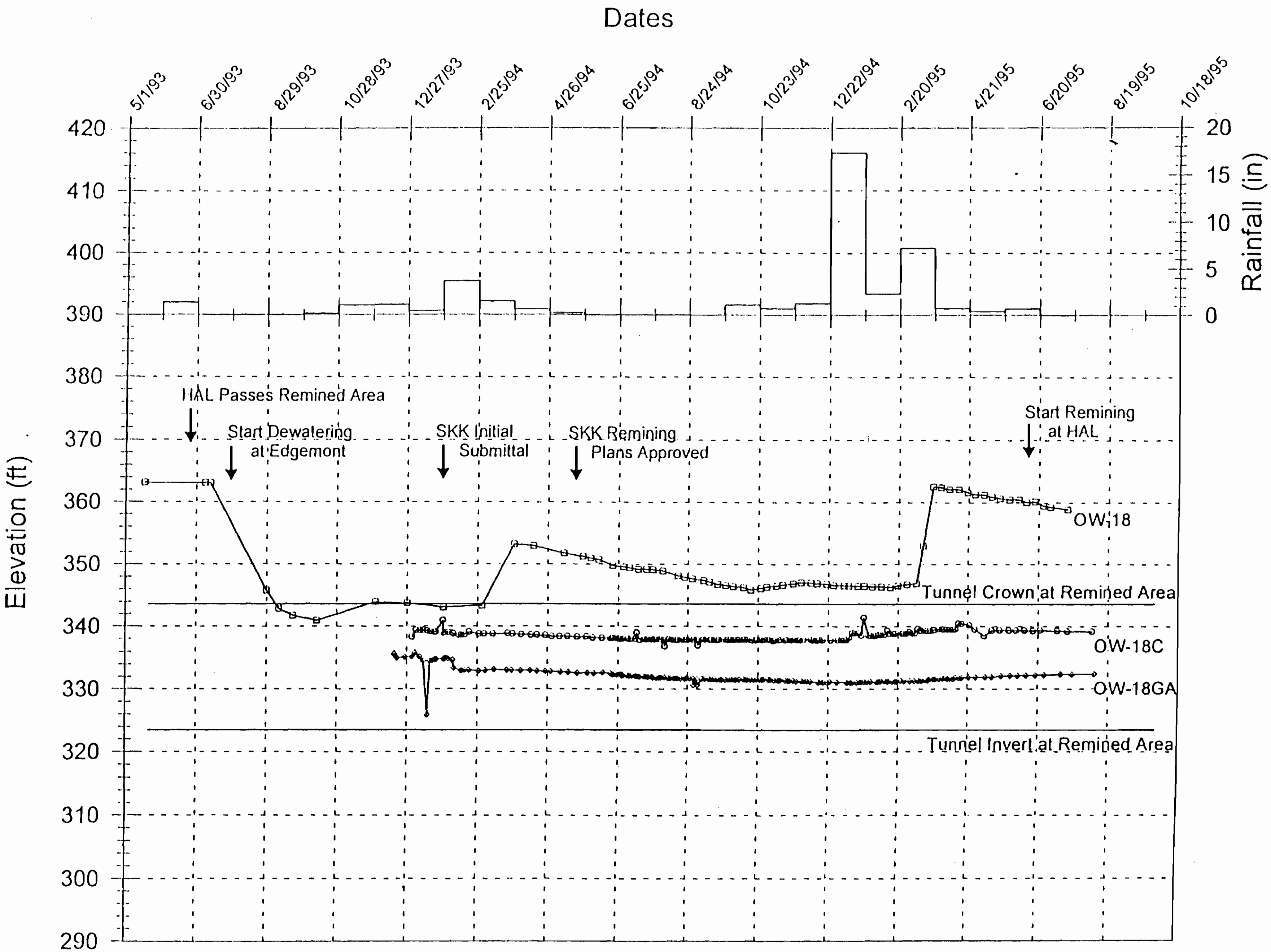

Fig. 3.3 Ground Water Levels Near **HAL** Remined Area

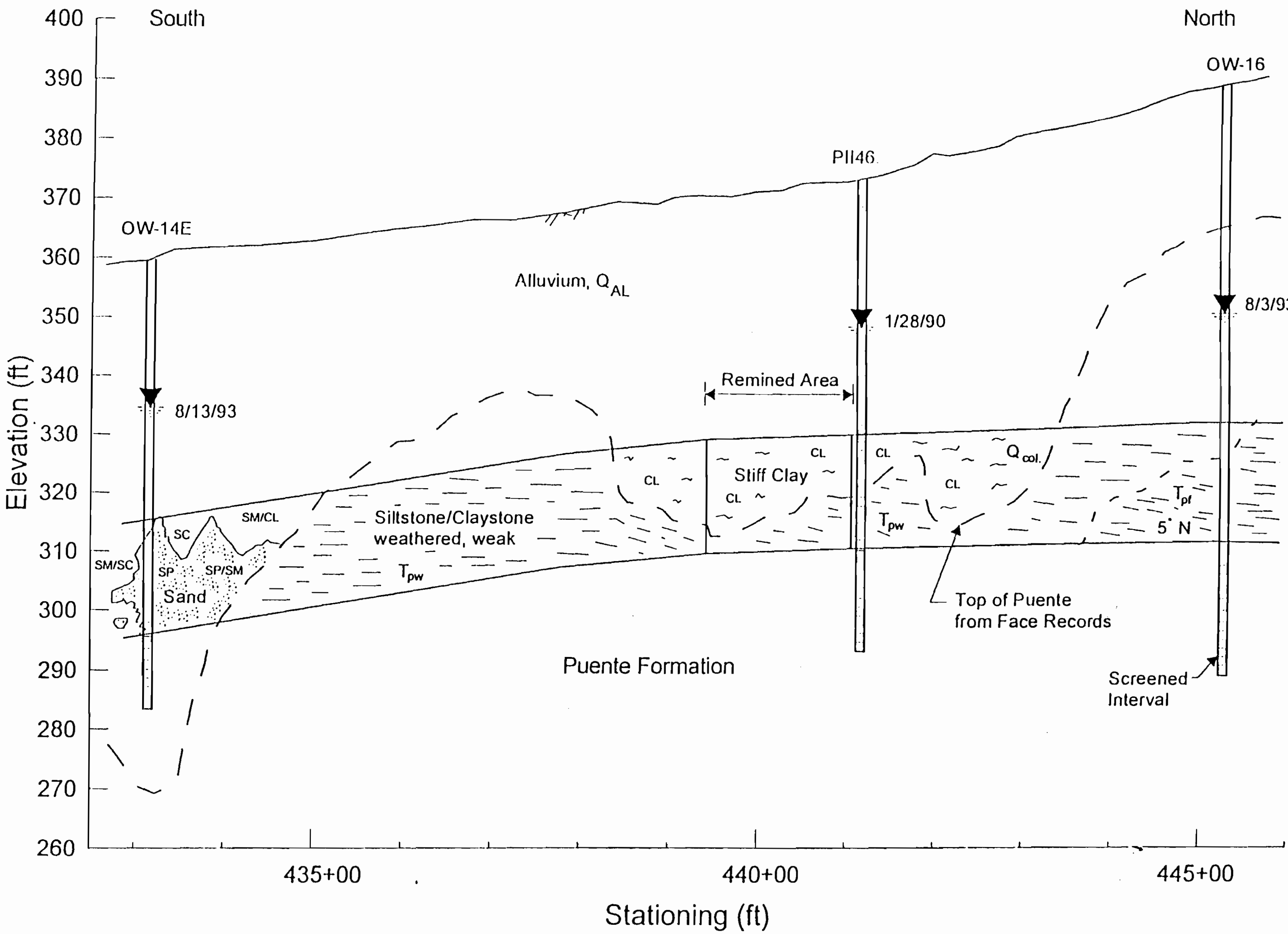

Fig. 3.4 Subsurface Conditions Near VAL Remined Area

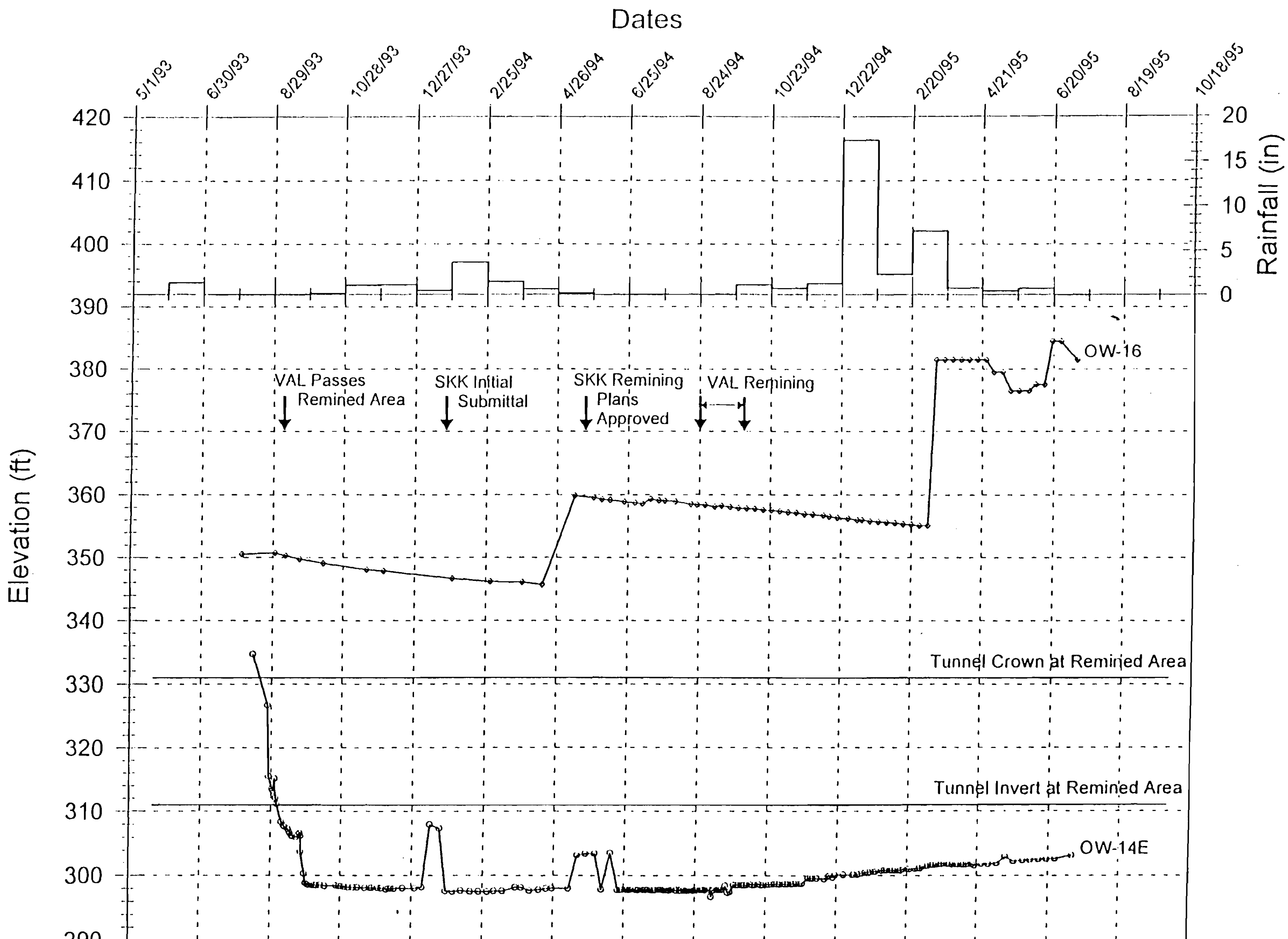

Fig. 3.5 Ground Water Levels Near VAL Remined Area

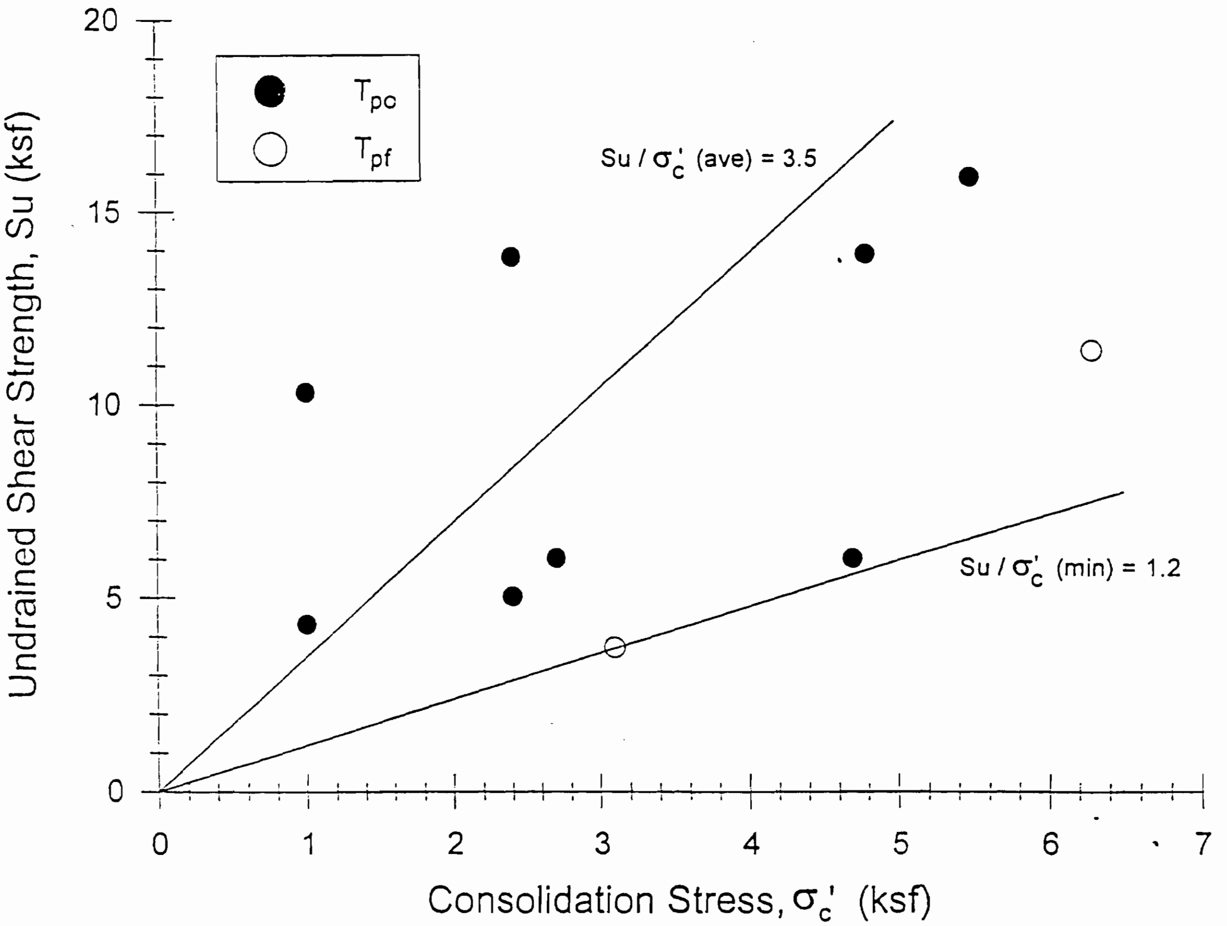

Fig. 3.6 Summary of Triaxial Test Data on Puente Specimens

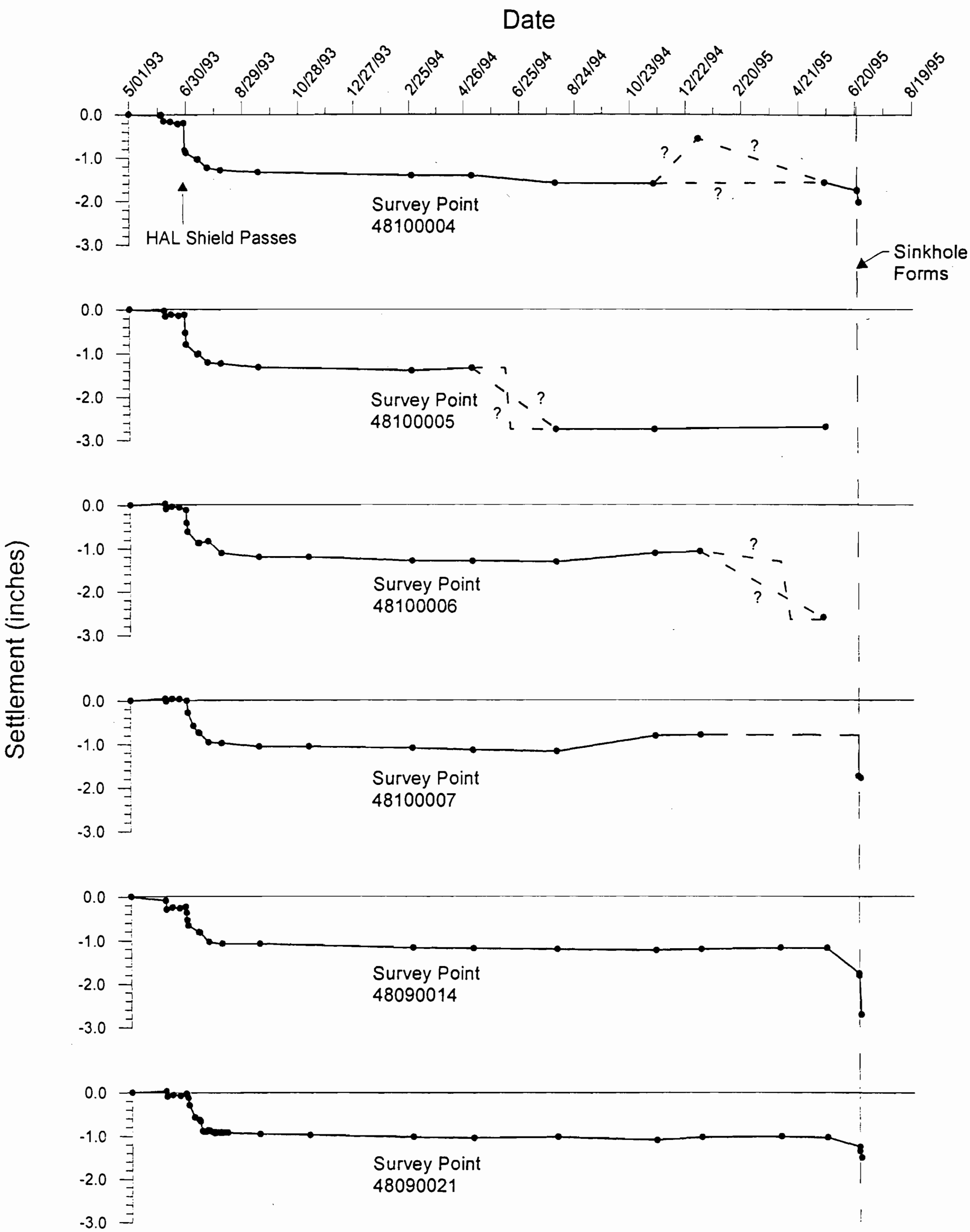

Fig 3.7 Ground Surface Settlements Near HAR Remined Area

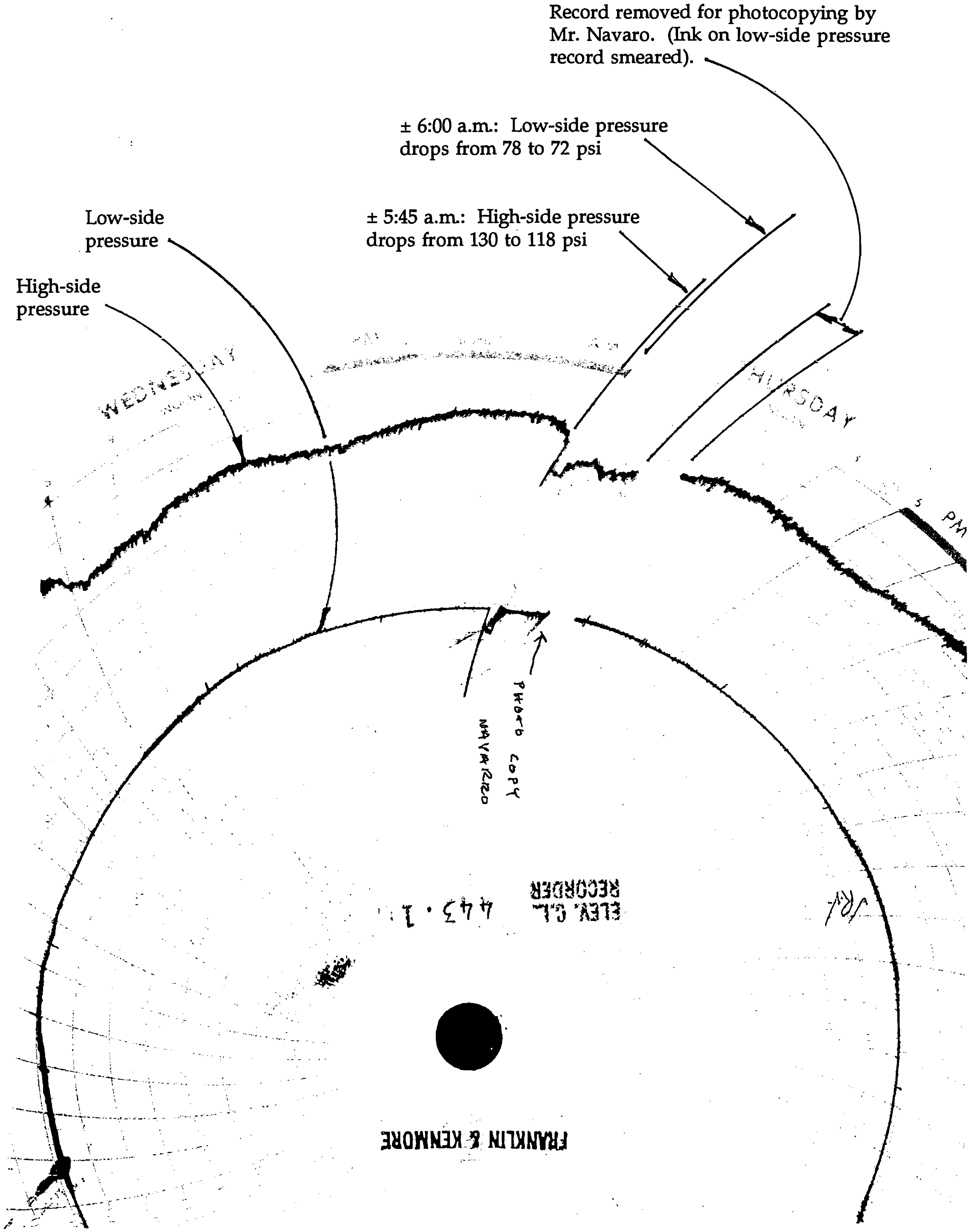

Fig. 3.8 - Franklin/Kenmore pressure record

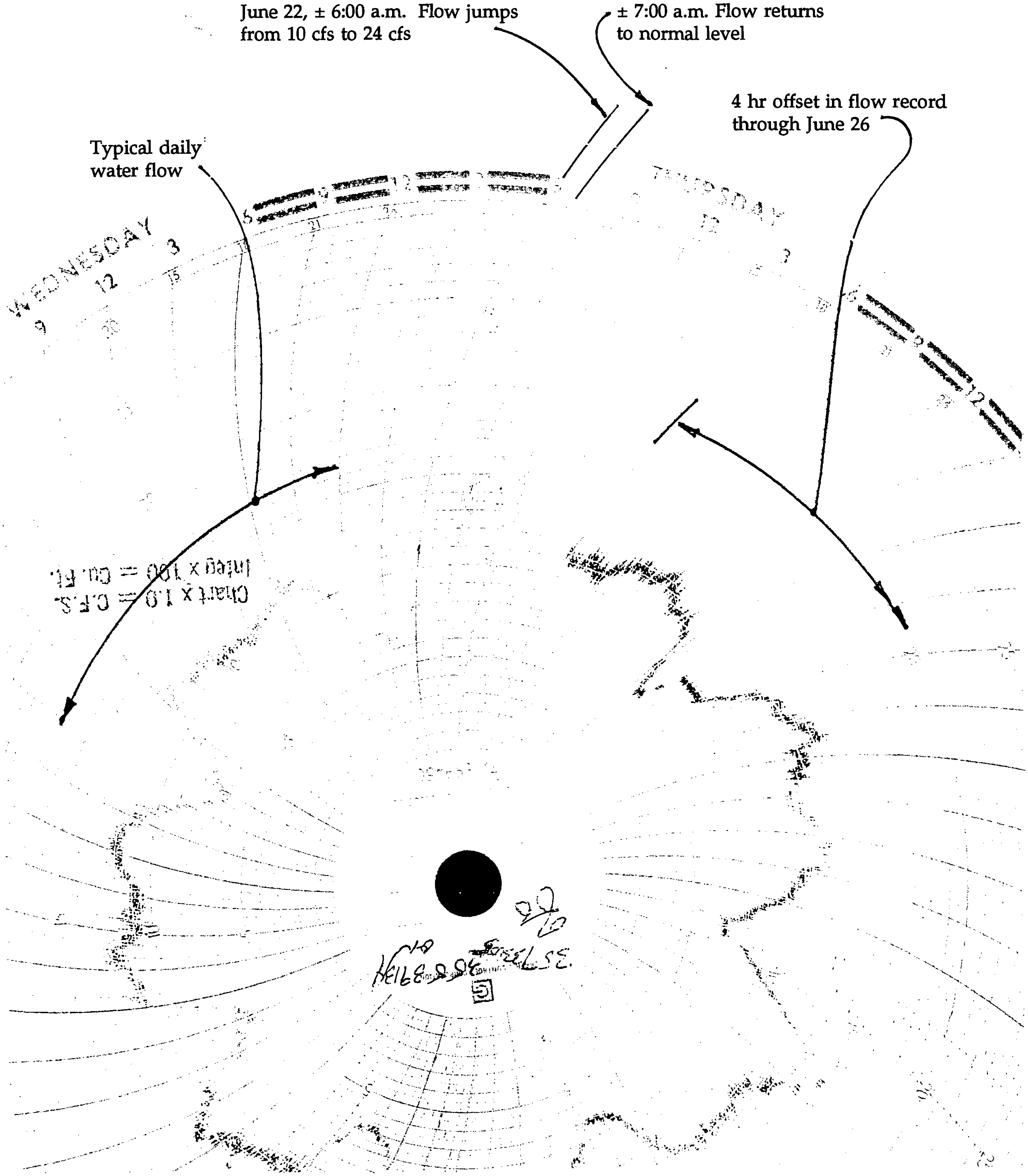

Fig. 3.9 - Franklin/Kenmore flow record, June 19-26, 1995

CHAPTER 4 - SEQUENCE OF EVENTS LEADING TO THE DEVELOPMENT OF THE SINKHOLE

The following description of events is based on the information in the P-D Daily Reports, the "Chronology of Hollywood Boulevard Sinkhole Occurrence and Mitigation Activities" prepared by P-D, LADWP records, and interviews with personnel in the tunnel on the morning of June 22, including Messrs. T. Hogan, M. Graber, S. Toney and J. Veatch of P-D, and Messrs. N. Hutchins and D. Sayer of SKK. A section through the remined area with segment and steel set numbers given for reference is shown in Fig. 4.1.

Work on the HAL remining area began by saw cutting segments 82 through 101 and then installing the split set anchors in the crown. Removal of segment 82 began on June 15, 1995. By June 16, segments 82 through 88 had been removed and steel sets 1 through 7 had been placed. Concrete segment removal is illustrated in Fig. 4.2 in a sequence of photographs taken by P-D personnel. The detail of the connection between the steel set and the concrete segment at the tunnel crown and the nature of the lagging behind the steel set are seen in these photographs. The lagging is set against the exposed Puente formation and blocks are placed between the lagging and the steel set to transfer load from the ground to the steel set. The exposed ground is seen to be dry when these photographs were taken. The concrete wall beam was placed on June 19 at the location shown in Fig. 4.1, and by the end of June 21, all segments through 100 had been remined and steel sets through 19 had been placed. Work reportedly had proceeded without significant problems through this juncture.

Table 4.1 summarizes the significant events of the morning of June 22. As Segment 101 was being prepared to be removed on June 22, SKK personnel first observed water seeping at 12:30 a.m. between the lagging near steel set 19. Soon after the appearance of water, foot blocks beneath steel sets 15, 16 and 17 were observed to have settled approximately 8 to 12 in. By approximately 2:30 a.m., the foot blocks for all steel sets had begun to settle and the concrete wall beam was settling at its west end. The area of seepage had grown to about 20 ft along the tunnel axis. The seeps were described as concentrated leaks which looked like water flowing from a garden hose. Estimates of the flow from one of these seeps was about 5 gpm. At about 3:00 a.m., P-D personnel notified the LADWP Trouble Board that there was a possible water main break and requested that a crew be immediately dispatched to shut down the water line. A crew from LADWP arrived on the site at 3:40 a.m., but not seeing water at the ground surface, left without shutting off the water.

The situation continued to deteriorate as the line of seepage moved eastward, with more seeps appearing randomly behind the seepage front. Load was being transferred to the lagging and steel sets as the water line reached a particular area. The crown was beginning to flatten out at the west end of the remined area. The concrete segments left in place at the crown had vertically displaced downward, giving the appearance of a flat arch. The concrete segments in the crown were breaking at about the 11:00 o'clock position, midway between the wood wedge expansion gap and the crown. By about 4:00 a.m., water was leaking through the split set bolts, and the entire remined area was taking load. P-D personnel estimated that the foot blocks for the steel sets ultimately had settled approximately 12 to 18 in. SKK supported the tunnel by placing steel posts at segments 87 and 88, i.e. at the west end of the concrete grade beam, then placed a steel post near segment 96 (this segment number was estimated by SKK personnel). LADWP records indicated a water main located above the tunnel ruptured between 5:45 and 6:15. The best estimate of the time of the rupture is 6:00, as noted by the low-side pressure data. At approximately 6:10, as SKK was setting up under the next segment to the east (95), the crown of the tunnel collapsed.

A photograph taken minutes after the collapse by P-D is shown in Fig. 4.3. The two steel posts at segments 86 and 87 can be seen. One can also see several concrete segments at the crown which had flattened. The difference in the visible height of the concrete wall beam along the right side of the tunnel at the springline gives an indication of the settlements of the beam which had occurred up to the point when the photograph was taken. The photograph also shows that the fallen soil was moist and not full of water, in agreement with reports by personnel in the tunnel. However, in the opposite direction, the collapsed soil was apparently much wetter because it filled the tunnel to just below the springline at Cross Passage (CP) 28 and extended approximately 140 ft to the west. As earth and water flowed to the west, portions of it flowed through CP 28 into the HAR tunnel.

The collapse resulted in the development of a sinkhole at the ground surface at 6:15 a.m. according to P-D personnel. The ruptured 10-in. diameter water main discharged water into the sinkhole until 7:30 a.m. (according to P-D). LADWP data indicate the water was shut off at 6:30 to 7:00 a.m. This discrepancy may be explained by continuing flow from the pipes. A gas leak was reported by the Gas Company at about 8:00 a.m. and the Los Angeles Fire Department (LAFD) directed the gas company to shut down the main gas line and prevented any work from being done in the tunnel until the gas line had been secured and shut off, which was done at 2:45 p.m. Until the gas line was shut off, SKK was not allowed to pump water out of the hole. At about 11:35 a.m., a second collapse of the remined area occurred which resulted in water and earth flowing eastward from the remined area through the Barnsdall Shaft and into the Vermont tunnels. The muck left by this flow was about 4 ft high at the portal to the HAL tunnel at the Barnsdall shaft. Because of the amount of ponded water released in this second event, the character of the earth flow was more fluid than the initial collapse. It flowed to the east presumably as a result of the collapse of the remaining temporary support system at the remined area and the restraining effect of the earth from the initial collapse which filled the west portion of the remined area and extended several hundred ft to the west.

TABLE 4.1 - MAIN EVENTS OF JUNE 22

12:30 a.m.	Water seeping into tunnel near set 98 to 100 Foot blocks 15, 16 and 17 settling (10 to 12 inches) Earth in crown area "paying off" (Danny Sayer)
2:00 a.m.	All steel sets have begun to settle Steady drips over a 20 ft area as water lines begins to move east towards Barnsdall shaft (Danny Sayer)
2:30 a.m.	S. Toney into tunnel at 2:30 a.m. and observes water seepage in crown area
3:00 a.m.	Crown begins to flatten out Timbers begin to break Water seeps moving east randomly behind water line Start place vertical supports in tunnel as SKK "prepares to lose ground" (Danny Sayer and Norm Hutchins)
4:00 a.m.	Split sets leaking at crown (S. Toney)
?	Foot blocks settled 12 to 18 in. prior to collapse (S. Toney)
5:45 a.m.	LADWP high-side pressure data indicates water main has ruptured
6:00 a.m.	LADWP low side pressure data indicates water main has ruptured
5:45 to 6:15 a.m.	LADWP flow data indicates water main has ruptured
6:10 a.m.	Tunnel collapse (Danny Sayer)
6:15 a.m.	Sinkhole reported at ground surface (P-D)
6:30 to 7:00 a.m.	LADWP pressure and flow data back to normal range
7:30 a.m.	Water main flow stops (P-D)
11:35 a.m.	Second collapse: water and earth flush eastward to Barnsdall shaft

Times at or before 4:00 a.m. were times given to WJE in interviews with personnel in tunnel at time of collapse and therefore are only estimates.

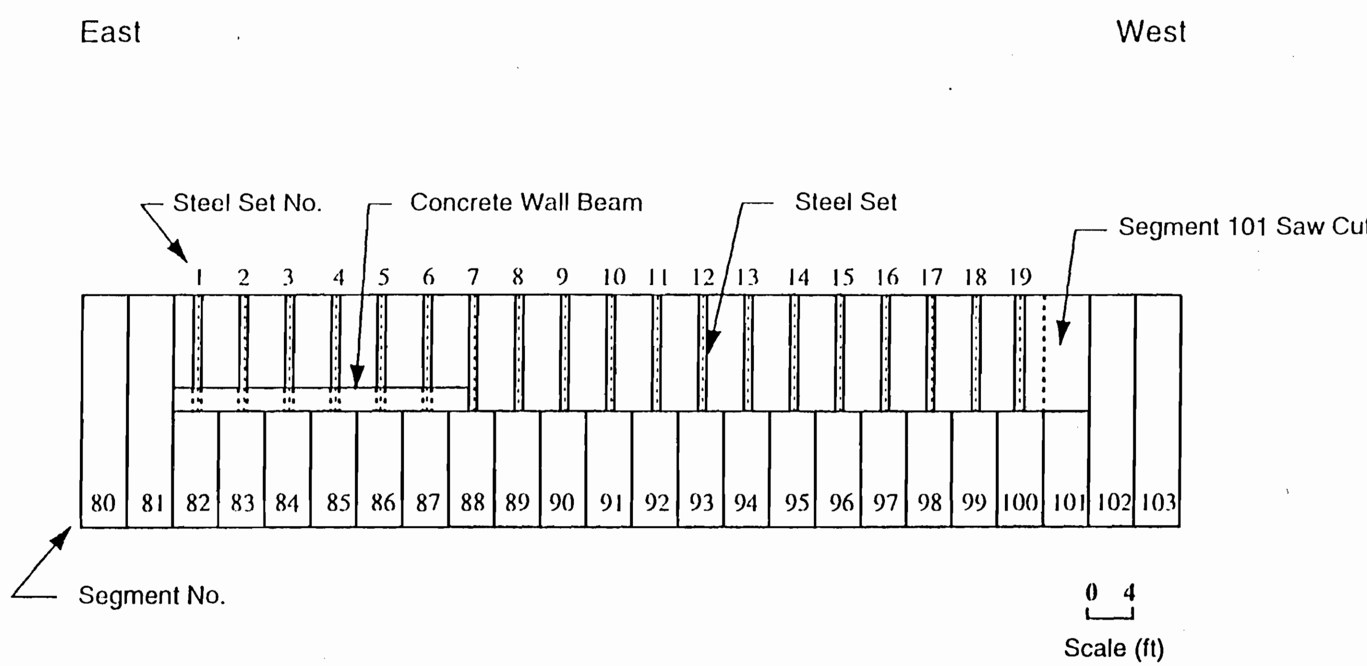

Fig. 4.1 Longitudinal Cross–Section of HAL Remined Area Just Before Initial Collapse

Fig. 4.2 - Removing concrete segments: HAL Tunnel looking east

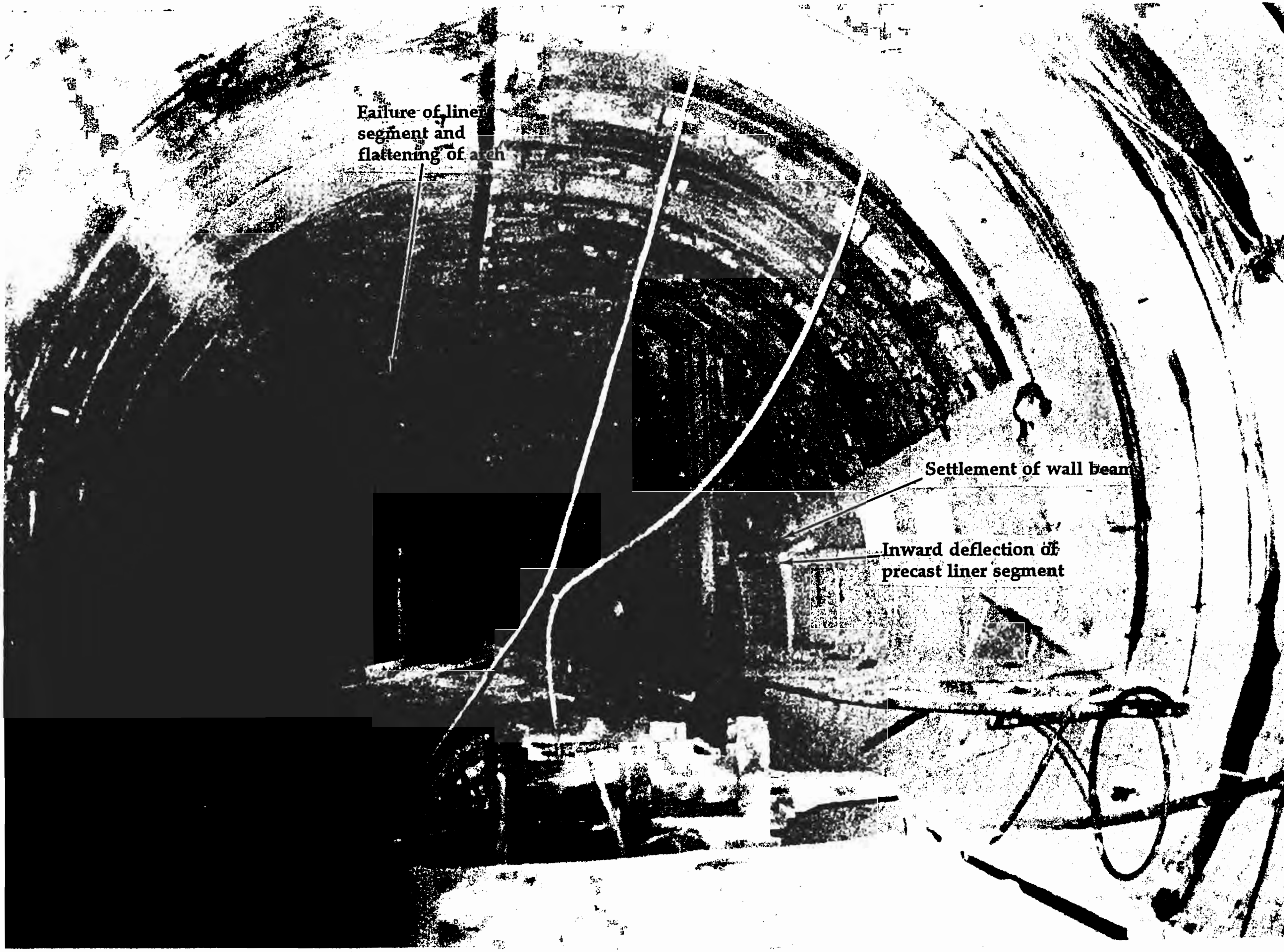

Fig. 4.3 - Photograph of collapse of HAL remined area: looking west

CHAPTER 5 - ANALYSES

5.1 Bearing Capacity of Foot Blocks for Steel Sets

A sketch of a typical foot block for a steel set is shown in Fig. 5.1 . The steel set bore upon a 7 in. by 9 in. by 5/8 in. thick steel plate which in turn rested upon a 12 in. by 12. by 3 in. or 12 in. by 8 in. by 3 in. timber pad. Observations of this foot block as it settled prior to the collapse indicated that the steel plate did not penetrate into the timber block. Thus the area of the bearing surface for the foot block was that of the timber.

The bearing capacity of the foot block was determined by the Brinch Hansen formula[1] which explicitly accounts for the effects of an inclined ground surface near a footing. For a level footing with no depth of embedment which is loaded vertically, the short-term, ultimate bearing capacity, q_u, can be found from:

$$q_u = 5.14\, S_u \left(1 + 0.2\frac{B}{L} - \frac{\beta}{147} \right) \qquad (5.1)$$

where B by L are the width and length (B<L) of the footing, respectively, and β is the angle, in degrees, that the slope makes with the horizontal.

The variable with the greatest uncertainty in equation 5.1 is S_u. Given the variations in shear strength found in laboratory tests discussed in Section 3.2.5, a value of S_u of 1 to 1.5 ksf may be applicable for the weathered Puente formation and a value of 5 to 7.4 ksf may be applicable for the oxidized and fresh Puente formations. Since the foot blocks for the steel sets rested on the weathered Puente at the western end of the remined area and the oxidized or fresh Puente at the eastern end, the S_u values based on the latter are more applicable for the bearing capacity calculation. This conclusion is based on the performance of the HAL face as the tunnel was excavated in the area. According to notes contained in the face sketches in the P-D reports, the face was stable as it was excavated.

The overload factor, OF, relates stability of tunnels in cohesive materials to S_u, and is defined as:

[1] Brinch Hansen, (1961), "A General Formula for Bearing Capacity," *Bulletin No. 11*, Danish Geotechnical Institute, Copenhagen.

$$OF = \frac{\gamma H}{S_u} \qquad (5.2)$$

where γH is the total vertical stress at the springline of the tunnel. For the depth of 76 ft to the springline in the vicinity of the HAL remined area, the OFs for S_u of 1 to 1.5 ksf vary from 6.3 to 9.5, while the OFs for S_u of 5 to 7.4 ksf vary from 1.9 to 1.3. An OF greater than 6 would indicate that the shear stresses induced by excavating an opening at the tunnel face would result in a failure of the soil and attendant large movements into the face and at the ground surface. An OF of about 1 would indicate that the induced shear stresses are low relative to the shear strength of the soil and the corresponding (essentially) elastic behavior results in a stable face and small movements at the ground surface. This latter OF corresponds to the observed conditions in the vicinity of the remined area, i.e., relatively stable face conditions during mining and ground surface settlements of 1 to 1.5 in. Therefore shear strengths of 5 to 7.4 ksf will be used to estimate the bearing capacity of the foot blocks.

Table 5.1 Summary of Bearing Capacity Analyses

Assumed S_u (ksf)	Timber Block Size L (in.) x B (in.)	Bearing Capacity (ksf)	Ultimate Load (kip)
5.0	12 x 8	13.9	9
	12 x 12	15.6	16
7.4	12 x 8	20.6	14
	12 x 12	23.1	23

The results of the bearing capacity calculations are shown in Table 5.1. The ultimate bearing capacity, q_u, of the foot blocks corresponding to these values of S_u ranges from 13.9 to 23.1 ksf. For the different sized timber blocks and for the range of S_u values, the ultimate load which each steel set can support varies from 9 to 23 kips. These values are significantly smaller than either the 307-kip load corresponding to the load from the design load of 60 ft of overburden, or the 81-kip load corresponding to the expected load for tunnels in the Puente formation, according to the initial SKK remining submittal.

5.2 Groundwater Conditions at HAL and VAL during Remining

Groundwater conditions at both the HAL and VAL remined areas during the remining operations are not known on the basis of observed data since no observation wells were located within either area.

The closest well was located 260 ft from the HAL site and 430 ft from the VAL site. Piezometer PII-46 was very near the VAL site (Fig. 3.4), but no data was collected after the initial site investigation, which occurred well before the tunneling began. Therefore the water conditions at both sites must be interpreted from wells located at some distance from each.

The basic model of groundwater flow has been described in Section 3.2.2, and consists of flow in the alluvium in an outward direction from Barnsdall Park. This area is the topographic high as well as the piezometric high in the vicinity. It serves as a recharge area for the ground water. The thin sand beds and the bedding, joints and secondary structure of the siltstone and claystone form the hydrologic pathways from the Alluvium to the Puente Formations. Thus, flow through the Puente is primarily through the secondary structure of the soft rocks; the more massive the rock, the less permeable it would be.

5.2.1 HAL remining - This remined area is located between the recharge area at Barnsdall Park and the closest observation well to the site, OW-18 (Fig. 3.2). Therefore it is up gradient from OW-18 and the water levels at the remined area would be higher than those measured at OW-18. In June 1995 when remining took place, the water level in OW-18 was at elevation 360 ft. Assuming a gradient of 0.022, based on water levels given in the GDSR between the Barnsdall shaft and OW-18, the water level above the remined area would be computed to be elevation 366 ft, or 24 ft above the crown of the tunnel.

5.2.2 VAL remining - This remined area is located between OW-16 and OW-14E (Fig. 3.4), with the water levels decreasing from OW-16 southward. Therefore it is down gradient from OW-16 and the water levels at the remined area would be lower than those measured at OW-16. In September 1994 when remining took place, the water level in OW-16 was at elevation 358 ft. Assuming a gradient of 0.013, based on water levels between OW-16 and OW-14 before the dewatering system near OW-14E was pumped, the water level above the remined area would be computed to be elevation 351 ft, or 20 ft above the crown of the tunnel. This calculation assumes that the dewatering near OW-14E had no effect on the water levels at the remined area.

If it is assumed that the piezometric variation between OW-16 and 14E was linear at the time of the VAL remining, then the water level at the VAL remined area would be computed to be elevation 332, or near the crown of the tunnel in the area.

The most likely scenario would place the water level somewhere between these limits, probably somewhat closer to el. 351 rather than el. 332.

5.3 Effects of Ground Water

The remining operation removes a portion of the segmental liner and results in the exposed Puente becoming an unobstructed seepage face until concrete has again been placed against it. During exposure, these openings in the segmented liner act as a drain to which ground water will flow. Note that while the segmental concrete liner is not impervious, it is relatively so compared to an exposed face of ground 80 ft long and extending over a quarter of the tunnel circumference. The amount of flow will depend on the head of water above the crown and the flow characteristics of the rock after initial excavation of the tunnel.

The following analysis is based on observation well data and does not consider the flow from the ruptured water main. As such, it is representative of conditions before the rupture at approximately 6:00 a.m.

The horizontal coefficient of permeability of the Puente formation was estimated to be 2×10^{-6} cm/s, roughly 20 to 100 times larger than that in the vertical direction (Section 3.2.5). The pathways for seepage in this formation are along the bedding planes and sandstone beds. This k_h value is based on a slug test conducted in Puente formation with beds that dipped 5 to 10 degrees, based on information in boring logs. Presumably the permeability in the vertical direction in the field depends on the orientation of the bedding planes, and thus higher vertical permeabilities would exist if the beds were more steeply dipping. Furthermore, during tunneling operations, the Puente moved into the face and the tail void since there was no timely grouting of the tail void gap (Section 3.3). This movement caused a reduction in stress in the rock, attendant opening the of the joints, and increased permeability of the Puente Formation in the zones affected by the tunneling.

The face records at the HAL remined area indicated that presence of friable, interbedded siltstone/claystone with thin, fine grained sand beds which dipped at 65 to 70 degrees. The weathered Puente was jointed, highly to moderately sheared and slickensided. This type of material would exhibit an increase in permeability as a result of the tunneling operations, particularly in the direction of the bedding. The face records at the VAL remined area indicated that the bedding was obscure and the material in the upper portion of the tunnel was clay and not rock. Therefore very little flow would occur through secondary structure and the permeability would be controlled by the matrix clay. In this case, permeability would be quite low in both the horizontal and vertical directions, and little increase in permeability would result from the tunneling operations. Even under a relatively large water head, very little flow would occur in this type of material. This is what was indicated by the daily reports during remining at the VAL site.

The general pattern of flow which developed during the remining operation can be visualized by means of a flow net. A flow net[2] is the graphical representation of the solution to the Laplace equation which describes the steady state flow of water through a porous medium. It consists of flow lines and equipotential lines along with the pertinent boundary conditions. The ground water flows in a complex three-dimensional pattern toward the openings in the liner at the remined area. This pattern is further complicated by the fact that water flows through secondary structure of the Puente formation. These factors make a detailed flow net analysis quite difficult and not particularly useful. To visualize the flow and to make estimates of hydraulic gradients acting at the openings, simplified models are more appropriate to use in this case.

Two, two-dimensional boundary element models are used herein to evaluate the flow towards the openings (Fig. 5.2). Boundary element flow models allow one to specify conditions at the boundary of a porous medium and compute the flow characteristics within the mass. Boundary conditions can be either no flow or piezometric head-specified. A seepage face is specified by setting the head equal to zero at such a boundary. Material properties are taken as uniform within the mass, and hence only the Puente formation is modeled in the analysis. From the solutions, flow nets can be drawn which indicate the general pattern of flow, hydraulic gradients -and thus seepage forces - can be computed, and flow rates can be calculated.

The two different models shown in Fig. 5.2 are used to evaluate two possible pathways for the water. The longitudinal section assumes that the alluvial channel near OW-18 can affect the conditions at the remined area, and implies that the bedding *between* the remined area and the alluvial channel was nearly horizontal. Consequently, the piezometric heads are specified at the boundaries adjacent to the alluvial channel and above the tunnel. The east end of the upper boundary of the Puente is modeled as impervious to represent the presence of fresh Puente formation at higher elevations in this location. The

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[2] Cedergren, H.R., (1967), *Seepage, Drainage and Flow Nets*, John Wiley & Sons, Inc., New York.

impervious boundary on the left hand side of the model represents the groundwater divide near Barnsdall Park. The transverse section assumes that the presence of the alluvial channel has little effect on the flow regime and considers the effect of the steeply dipping beds at the remined area. Not enough geologic information was available to consider the possible recharge from the vertical boundaries in this case. The impervious boundaries were set far enough away from the tunnels such that they did not significantly affect the equipotential lines near the tunnel. This model also considers the effects of the adjacent tunnel on the flow path.

In both models, the piezometric head above the crown of the tunnel was 24 ft of water (section 5.2.1). The tunnels are considered impervious, except for the exposed Puente at the remined area. Use of equal permeabilities represents the effects of increased vertical permeabilities from the steeply-dipping (65 to 70 degrees) bedding planes opening during initial tunneling and remining. Two sets of coefficients of permeability were considered in both models, $k_h = 100k_v$ and $k_h = k_v$, to account for the possible variation in the anisotropy. In all cases, k_h was selected 2×10^{-6} cm/s based on the results of the slug tests (Section 3.2.5).

The flow net based on the longitudinal analysis with the anisotropic permeabilities is presented in Fig. 5.3a. Because the k_h is 100 times larger than k_v, the horizontal dimension is shortened by a factor of 10 $\{=(k_h/k_v)^{1/2}\}$ to account for the differences in flow rate in the two directions. The quantity of flow through each flow channel, the area bounded by two adjacent flow lines, is the same for all channels. As can be seen by examining the flow lines, about 30% of the total flow comes from the alluvial channel under the conditions assumed in this analysis, but a significant portion comes from above the tunnel. When this analysis was conducted with equal vertical and horizontal permeabilities, the flow into the remined area was essentially all from above the tunnel. Depending on the in situ ratios of permeability after tunneling, it is possible that recharge from the alluvial channel would have affected the remedial area.

The flow net based on the transverse analysis with equal permeabilities is presented in Fig. 5.3b and illustrates the flow pattern associated with these conditions. This case would be representative of the situation with bedding dipping 65 to 70 degrees, as observed near the remined area. In this case, all flow in the remined area came from above the tunnel.

Table 5.2 Summary of Flow Net Analyses

Section	Permeability Ratio	k_h (cm/s)	Exit Gradient	Ave. Flow Rate (gpm)/ft^2
Longitudinal	$k_h = 100\ k_v$	2×10^{-6}	2.4	0.0088
	$k_h = k_v$	2×10^{-6}	2.0	0.034
	$k_h = k_v$	2×10^{-8}	2.0	0.00034
Transverse	$k_h = 100k_v$	2×10^{-6}	2.6	0.0028
	$k_h = k_v$	2×10^{-6}	2.3	0.092
	$k_h = k_v$	2×10^{-8}	2.3	0.00092

The average flow rates over the entire exposed remined area and the exit gradients[3] for the analyses are given in Table 5.2. Because the flow rates are directly proportional to the permeability, the assumed value of permeability is the most important parameter in the analysis when determining quantities of flow. When one compares the transverse and longitudinal results for the same permeability assumption, the flows are similar. Also note that for the transverse section, with $k_h = 100k_v$, the flows are small relative to the others; this suggests that either the alluvial channel affected the remined area or k_v after tunneling is not equal to $k_h/100$. This latter situation is reasonable for steeply dipping beds (as much as 70 degrees) after they have been loosened during tunnel excavation. Either scenario is possible. It is important to observe that the exit gradients are similar in all cases.

The flow quantities appear small, but it must be noted that these represent the average flow over the entire seepage face. In the case of a bedded and jointed rock, the flow concentrates through the secondary structure (recall that the seepage was observed as a flow from a garden hose at an estimated rate of 5 gpm). When this average flow is concentrated in smaller zones, the rate over that smaller area becomes much higher. For example, the total flow rate of 0.0088 gpm was computed for the longitudinal section over a projected area of 80 ft by 10.75 ft. If that flow was concentrated within an area of 1 ft^2, the rate of flow would be 7.6 gpm. This 1 ft^2 flow area is large enough to contain a number of seeps which could account for the observed flows in the tunnel just prior to collapse. This estimate is based on the scenario with the lowest flow rate when it is assumed that k_h equals 2×10^{-6} cm/s. Only for cases with $k_h \leq 2 \times 10^{-8}$ cm/s would there be insufficient flow to explain the observed seeps. Thus for any of the scenarios in Table 5.2 with k_h equal to 2×10^{-6} cm/s, the naturally-occurring ground water could have

[3] The exit gradient is defined as the change in head divided by the flow length for the elements closest to the seepage face, in this case the elements closest to the remined area.

provided the amount of water which was observed prior to collapse.

The hydraulic exit gradients vary within a narrow range and average 2.3. The value of hydraulic gradient is most sensitive to the total head and the flow length. The flow length is primarily affected by the geometry of the openings relative to the stratigraphy, which is known with a reasonable degree of confidence. The head is based on the observed water levels in OW-18 and the assumed hydraulic gradient. Reasonable changes in the gradient used to make these calculations will not greatly affect the total head.

In summary, both models of flow yield the same basic information. The quantities of flow and hydraulic gradients are similar. Thus the source of the water could be either above the tunnel, with the flow path along steeply dipping beds and joints or a combination of flow from above the tunnel and from the alluvial channel near Edgemont St.

These analyses were not conducted for the VAL tunnel since no flow was observed at the VAL site. This difference in behavior is likely due to: (1) the Puente formation was more massive at the VAL site - the bedding was indistinct - and the bedding that did exist, dipped only 10 degrees, and (2) the horizontal distance to the down-gradient alluvial channel at the VAL remined area was twice as great as that at the HAL remined area.

5.4 Loads on Steel Sets

5.4.1 Design loads - The steel sets were sized for a pressure of 7200 psf corresponding to 60 ft of overburden, or three times the nominal tunnel diameter, the same design load as the segmental concrete liner. While it is accepted in tunneling practice that stress redistribution through arching will result in stresses acting against a liner that are less than the overburden stresses, relatively shallow tunnels are commonly design for the thrust corresponding to full overburden stress. When a liner is sized for this load, the stresses induced by transporting the liner into the tunnel, assembling it and pushing against the liner to advance the shield can be adequately resisted. Thus it is not expected that loads corresponding to full overburden act on the liner after it has been installed[4].

[4] The load-deformation response of a rock-liner system can be envisioned by means of a ground reaction curve (Deere, D.U., Peck, R.B., Monsees, J.E. and Schmidt, B., "Design of Tunnel Liners and Support Systems," Final Report, U.S. Department of Transportation, Contract No. 3-0152, University of Illinois, 1962). When an excavation is made and the rock at the excavated surface moves toward the opening, the load from a rock mass decreases with increasing movements as a result of mobilization of the rock's shear strength. However, after a certain amount of deformation, which is a function of the rock type,

5.4.2 Expected loads - It is standard practice to estimate expected loads on liner systems using methods based on arching theory and practical experience. These methods generally express the vertical pressure on a tunnel to be equal to a certain height of soil or rock times the unit weight of the soil or rock. This height depends on the soil or rock type and the local groundwater conditions. The Puente formation encountered by the tunnels in the vicinity of the HAL remined area is a soft rock (based on a geological classification) which is expected to act similar to a jointed, very stiff to hard clay. For moderately blocky and seamy rock, the load on the concrete segments as given by Terzaghi's rock load classification[5] corresponds to a height of 0.5 times the tunnel width, or in this case 0.5(21.5) or 10.75 ft. This height corresponds to a pressure of 1340 psf, much less than the design pressure of 7200 psf.

The loads on the steel sets in the remined area would be smaller than those corresponding to 1340 psf, at least initially. When the concrete segments are removed, the stresses at the exposed rock surface are equal to zero and additional unloading of the rock would occur. Assuming that the movements associated with this additional unloading are small, as was the case in the remining operations until segment 101 was reached, the loads on the steel sets would have to be smaller than those corresponding to 1340 psf because the small inward movements which occurred between the time when the segment was removed and the lagging was set. Once contact between the rock and the steel set was established, the resulting stress against the steel rib would be smaller than 1340 psf, assuming that the inward displacements were not large enough to cause loosening of the jointed rock.

When there is seepage towards the exposed section, a force will be induced in the direction of flow. Because the exposed Puente in the remined area acts as a seepage face, once a steel set was been blocked against the lagging, load from the seepage forces would be transmitted to it. These seepage forces can be computed by:

$$F_s = i\,\gamma_w \quad (\,force\ /\ volume\,) \tag{5.4}$$

where F_s is the seepage force per unit volume of soil, i is the hydraulic gradient which can be found from

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the load on the support system begins to increase. In the case of a jointed and bedded rock, this increase is due to the effects of opening the secondary structure. The load that eventually is taken by the support system depends on when the support system is installed and its stiffness.

[5] Proctor, R.V. and White, T.L., (1946), *Rock Tunneling with Steel Supports*, The Commercial Shearing and Stamping Co., Youngstown, Ohio.

a flow net, and γ_w is the unit weight of water. F_s represents the force that develops at steady state flow conditions. It would take some time for this condition to develop after water began to flow towards the exposed surface during remining operations.

Based on the analyses summarized on Table 5.2, the average hydraulic gradient for the earth adjacent to the exposed face is 2.3. The steel sets must resist both the seepage forces which develop in the rock and the submerged weight of the loosened rock. If one assumes that just 2 ft of rock has loosened (this is one half the distance between the steel sets), then the pressure acting on the steel sets due to the seepage forces is 290 psf. Adding this value to the pressure found from the submerged weight of two ft of earth, results in a total pressure of 410 psf. For a 4 ft span between steel sets, the load in each steel set is 18 kips. As loosening progresses upward into the rock mass above the tunnel, the pressure on the steel sets would increase at a rate of about 200 psf/ft.

5.5 Stability of Foot Blocks

Based on consideration of the bearing capacity of the Puente formation at the remined area, the ultimate load the foot blocks can sustain varies from 9 to 23 kips, depending on the assumptions concerning the shear strength of the Puente. This capacity is far less than the 307 kip load corresponding to the design pressure of 7200 psf. It is also significantly less than the 81 kip load corresponding to the expected pressure of 1888 psf given in the SKK alignment repair submittal for tunnels in Puente formation. The ultimate load of 9 to 23 kips is approximately equal to the 18 kip load based on loadings from the weight of the rock and seepage force described in Section 5.4.2. Thus one would expect a bearing capacity failure of the foot blocks, and attendant large settlements, once seepage forces began to develop. This was the case at the remined section. This also is consistent with the observations throughout the remining operation that the tunnel was stable until the seepage was noted. Given that the allowable bearing capacity of a footing usually includes a factor of safety of 3, or sometimes 2 for a temporary loading, the foot blocks were not adequately sized to resist even the 18 kip load, much less the 307 kip design load. However, it was adequate to resist loading from the dry Puente formation, as indicated by the observed performance. But had calculations been made, the foot blocks would have been shown to be inadequately designed (i.e. factor of safety < 2 for temporary loads) for any loading greater than approximately 200 psf.

5.6 Water Main Analysis

5.6.1 Metallurgical testing - Engineering Systems Inc. (ESI) examined the recovered sections of the water main and tested samples of the cast iron. The text of their report is provided in Appendix C. Nine longitudinal strips (Talbot strips), about 12 in. long by 1/2 in. wide, were tested in bending. The modulus of rupture (MOR) and modulus of elasticity (MOE) were determined from the Talbot strip tests. MOR is the bending stress at failure; MOE is a measure of the material stiffness. Parallel samples were tested in direct tension to determine tensile strength. Results are summarized below.

Test	Property	Range	Average	AWWA Spec
Talbot strip	MOR (psi)	27,621 to 35,194	31,460	30,000 min.
	MOE (psi)	5,686,733 to 7,488,468	6,678,000	10,000,000 max
Tensile test	Tensile strength (psi)	13,559 to 20,436	18,620	–

The pipe material is considered to be in substantial compliance with both the original (1908) and current American Water Work Association (AWWA) specifications. Although there were indications of corrosion, the pipe maintained its mechanical integrity.

Examination of the fracture surfaces by ESI indicates that the water main failed in bending, but there is no indication of preexisting cracks.

5.6.2 Settlement-induced stress - As described in Section 3.4, review of ground surface settlement records indicates localized settlement of Hollywood Boulevard near the sinkhole site prior to the start of remining. Ground point 48100005 (point B in Fig. 5) is in front of an access gate to the Barnsdall construction site. Between May and September 1994, settlement at this point increased 1.5 in. (from about 1.2 to 2.7 in.). Several months later, survey point 48100006 (point C) settled about the same amount. As illustrated in Fig. 5.5, ground movements that result in settlement can also cause significant bending stress in the water main. The water main will rupture when the bending stress exceeds the tensile strength for the cast-iron pipe.

The bending stress depends on the size of the water main, the amount of settlement, and length over which the settlement occurs. Since the survey points are 40 ft apart, the exact nature of localized settlements cannot be determined. The point of maximum settlement may be missed, and there is no way to distinguish between localized settlements occurring over lengths of, for example, 20 ft and 60 ft. Also, subsidence of the water main may be much less than surface settlement if the surface settlement is due to construction traffic.

A parametric study was undertaken to evaluate the relationship between settlement amount and length of the settlement zone. Average strength and stiffness values from the ESI tests were considered in the analysis. The average tensile strength of the cast iron was used rather than the MOR. Tensile strength is believed to be a more accurate indicator of the failure stress of the pipe cross section. Pipe joints were considered to be rigid. Also, the pipe was assumed to curve uniformly downward to the quarter points of the settlement zone and uniformly upward through the middle section. The results are shown in Fig. 5.5 as a plot of critical pipe deflection vs length of settlement zone.

As can be seen in Fig. 5.5, a 1-1/2 in. settlement would not be expected to rupture the water main unless the settlement occurred over a very short distance; that is, less than about 18 ft.

Figure 5.5 is also useful for evaluating the possibility of rupture of the water main 10 or 15 minutes prior to development of the first sinkhole, as suggested by the chronology discussed in Section 4. Foot block settlements of approximately 12 to 18 in. were reported prior to the initial tunnel collapse. Empirical relations have been developed between ground movement at the crown of a tunnel and observed ground surface settlements.[6] Given the geometry at the HAL site, the ground surface settlements can be estimated from

$$S_g = \left[1 - \propto\left(\frac{C}{D}\right)\right] S_c \tag{5.5}$$

where S_g is the settlement at the ground surface, S_c is the movement at the crown, C is the depth of cover above the crown (65 ft for the HAL site), D is the tunnel diameter (21.5 ft), and ∝ is an empirical factor which accounts for dilation of the earth above the crown. This value varies from 0.40 for dense sands (very dilative soil) to 0.13 for clays (incompressible soil).[6] Given 12 to 18 in. of movement at the crown (equal to the maximum settlement of the foot plate) and an ∝ value of 0.21 (corresponding to a slightly dilative soil), the surface settlement would be 4.4 to 6.6 in. Assuming a 5 in. surface settlement, Fig. 5.5

[6] Atkinson, J.H., and Potts, D.M., (1977), "Subsidence above Shallow Tunnels in Soft Ground," Journal of the Geotechnical Engineering Division, ASCE, Vol. 103(GT4), pp. 307-325.

indicates the length of the settlement zone corresponding to rupture in the pipe is about 32 ft. If the prior 1.5 in. settlement is added, the length of the settlement zone corresponding to rupture of the water main is about 37 ft. These lengths are somewhat less than the size of the original sinkhole, but rupture of the water main due to larger ground movements in the tunnel is conceivable. Therefore, this analysis does not rule out the chronology indicated by site observations and LADWP records; that is, the water main ruptured before the tunnel completely collapsed.

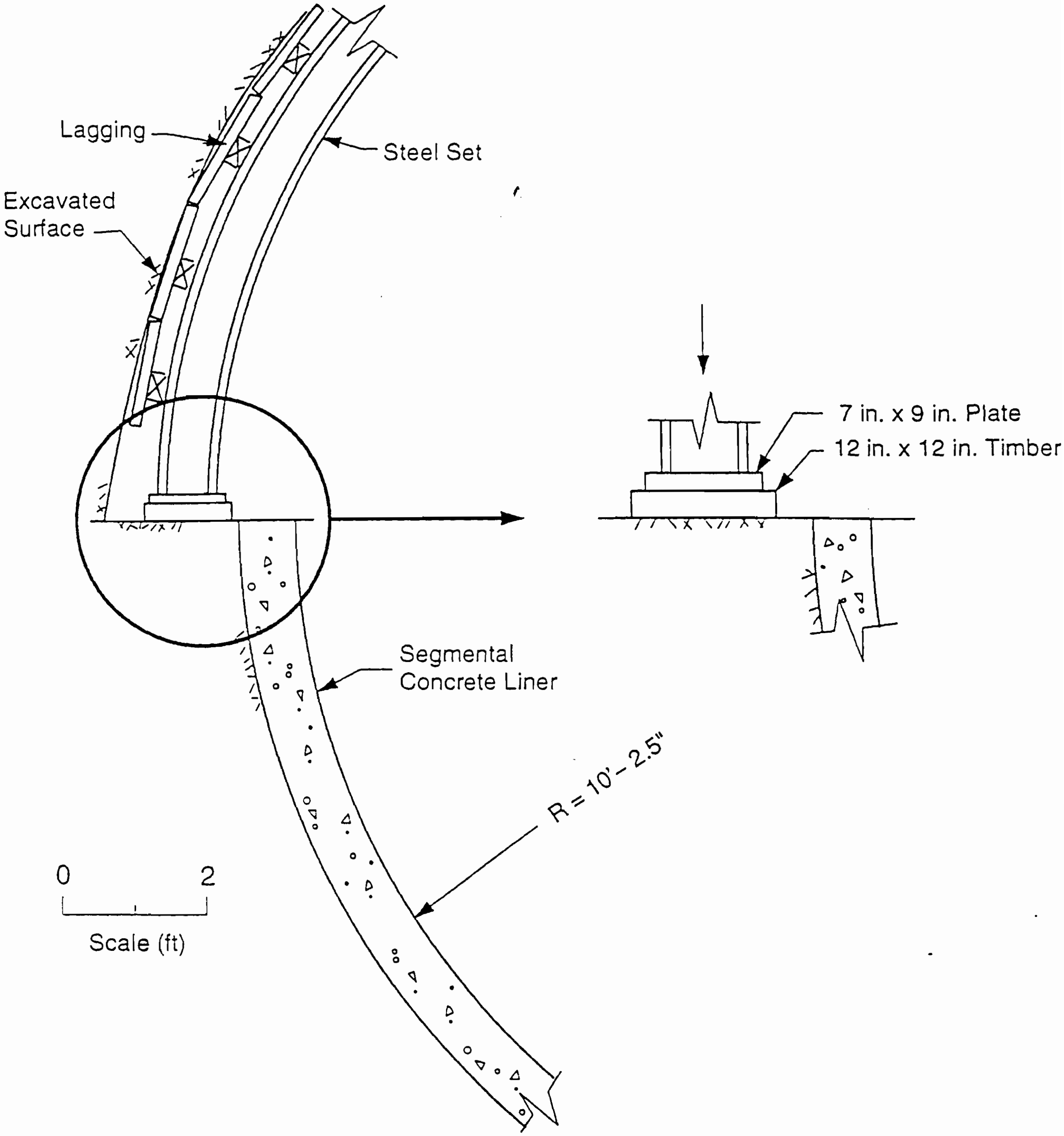


Fig. 5.1 Typical Foot Block For Steel Sets

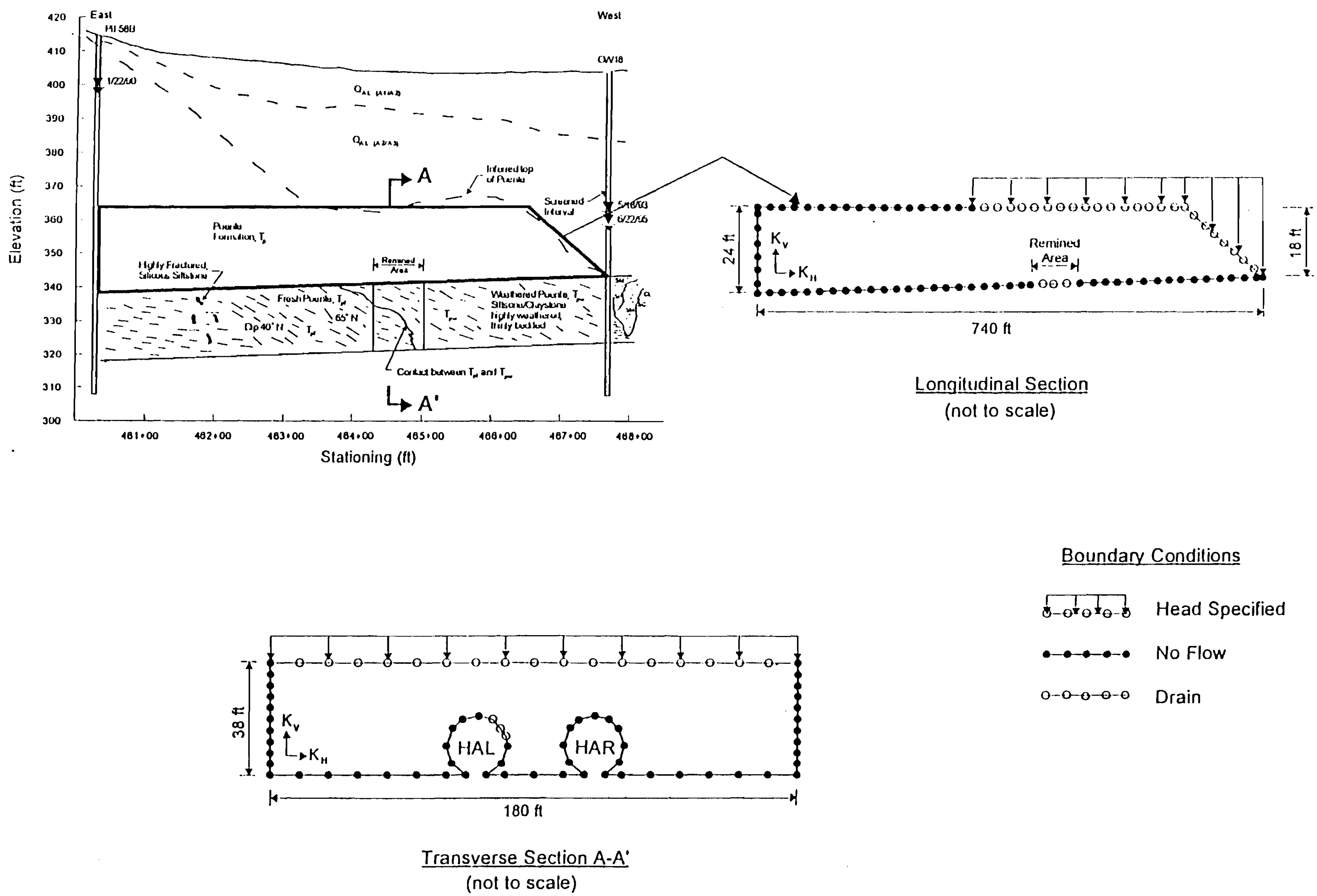
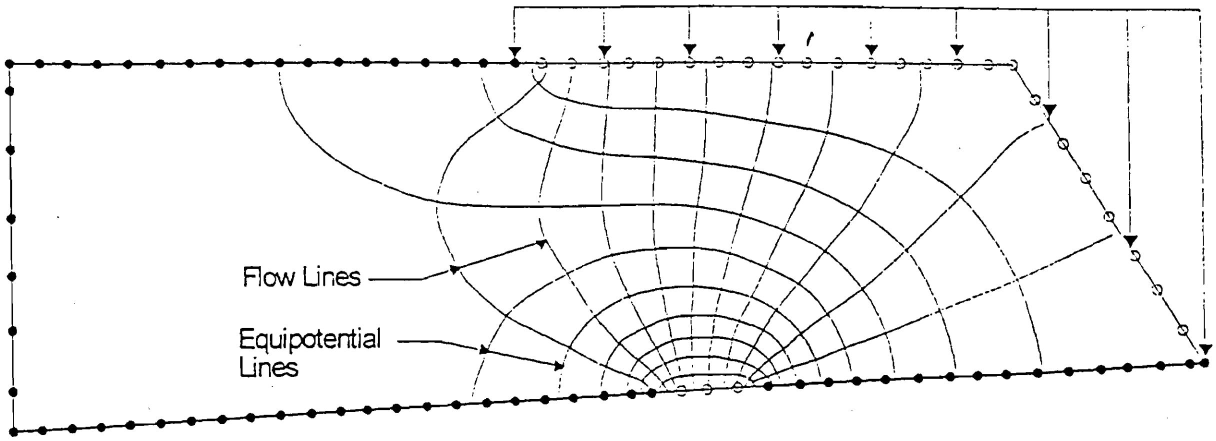


Fig. 5.2 Boundary Element Representations of Ground Water Conditions



(a) Longitudinal Section

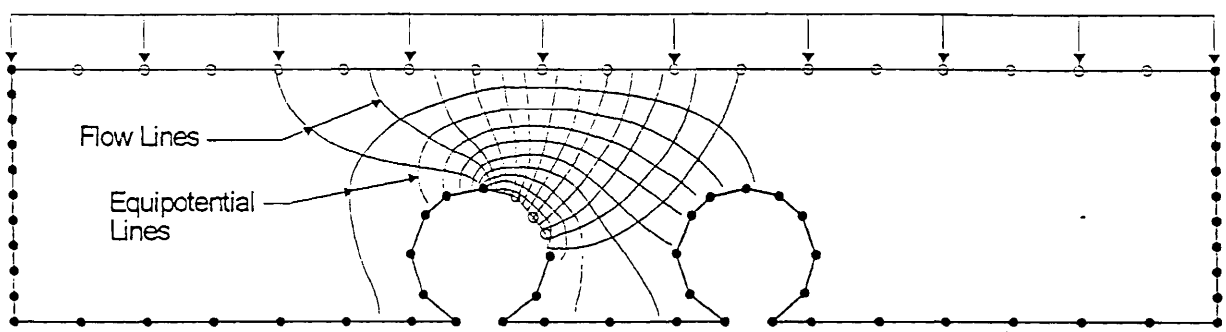
($K_H = 100\ K_V$)

Boundary Conditions

Head Specified

No Flow

Drain



(b) Transverse Section

($K_H = K_V$)

Fig. 5.3 Flow Nets for HAL Tunnel

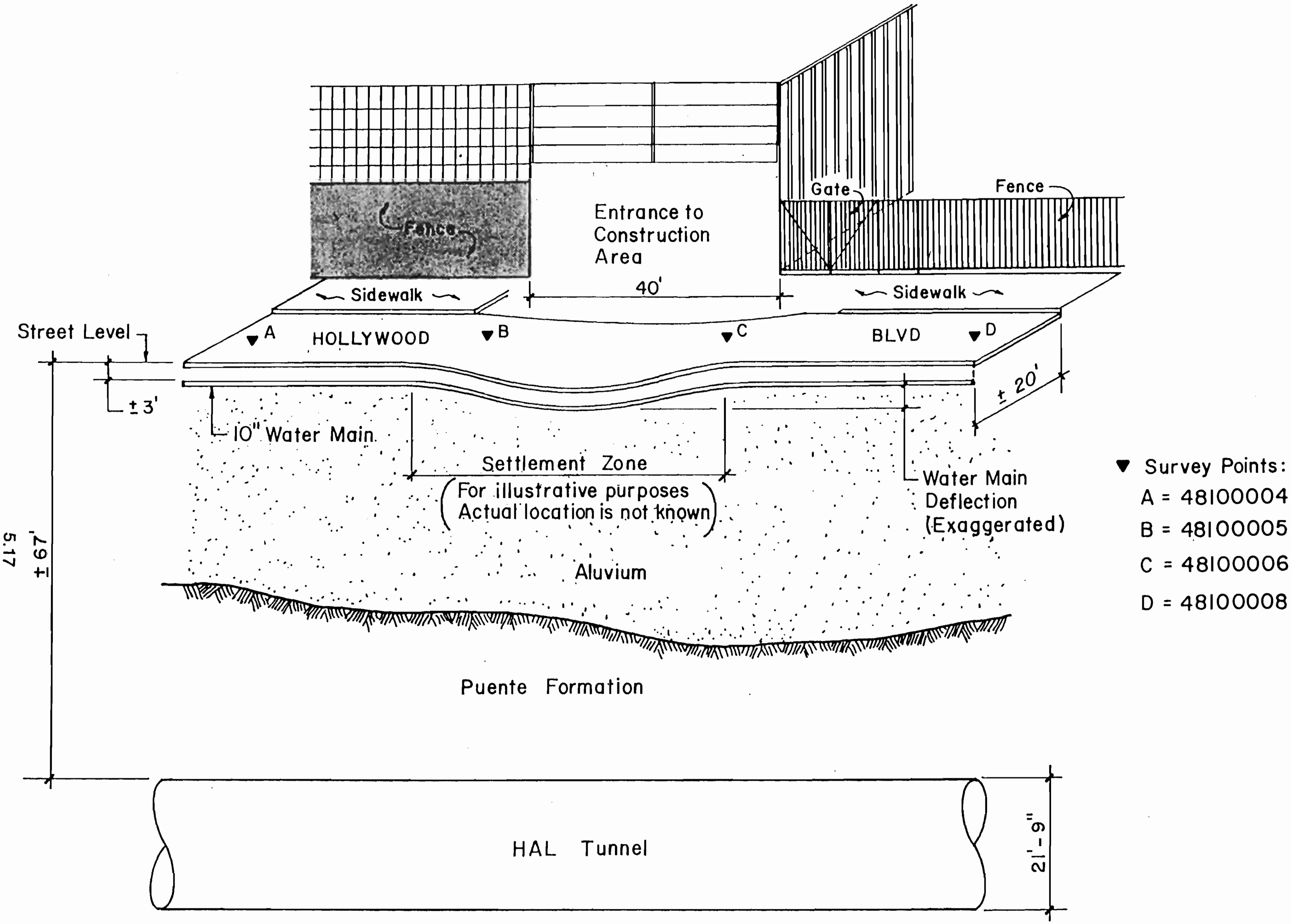


Fig. 5.4 - Illustration of water main settlement above remined area

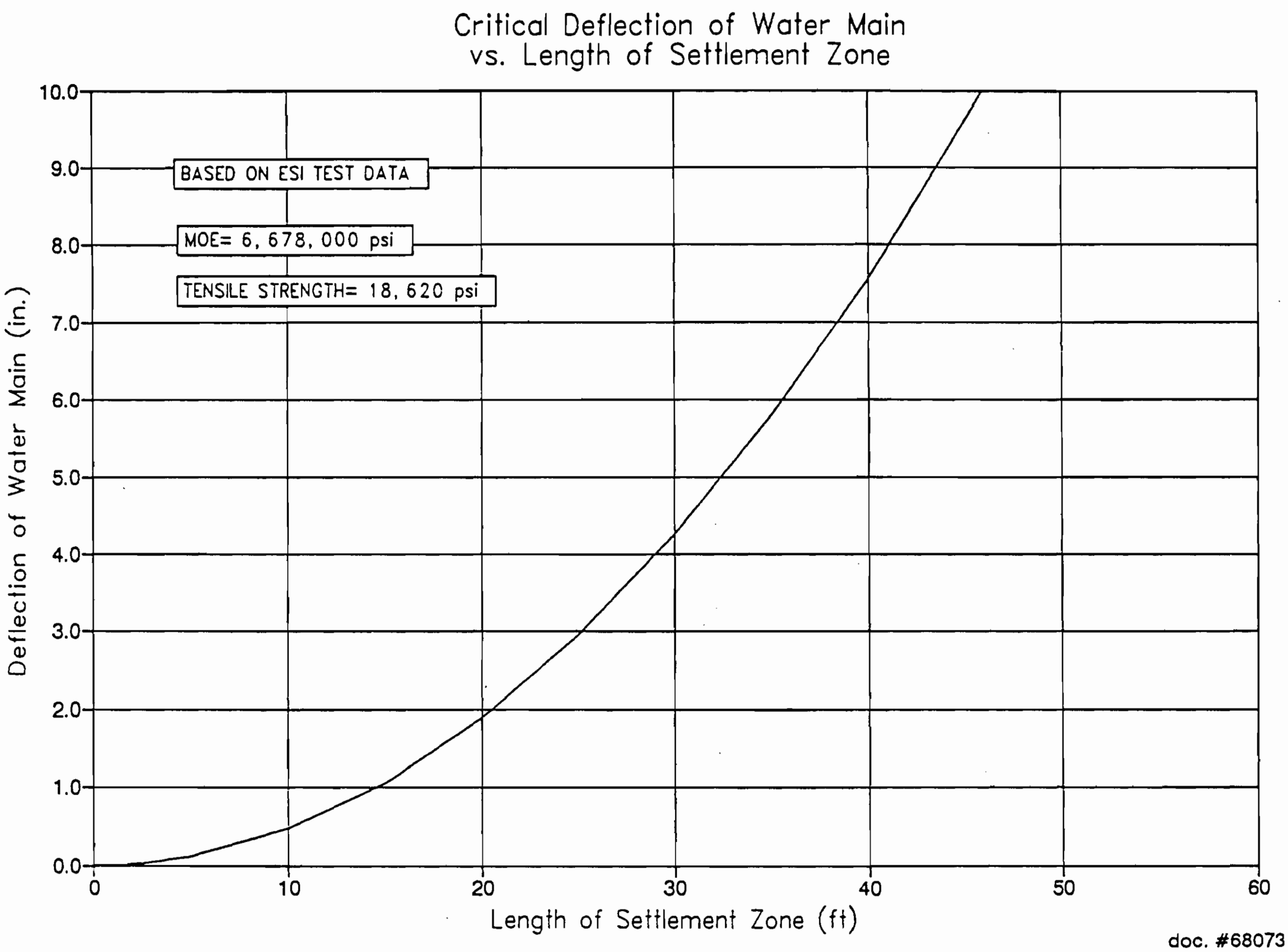


Fig. 5.5 - Critical water main deflection vs length of settlement zone

CHAPTER 6 - DISCUSSION

6.1 Comparison of HAL and VAL Remined Sections

Similar conditions existed at the HAL and VAL remined areas and the same construction procedures were used at both locations. Yet there were differences which were significant enough such that a collapse of the remined section occurred at the HAL site, but not the VAL site. The following sections discusses these differences.

6.1.1 Construction procedures - Review of the P-D daily reports made while remining both the HAL and VAL remined sections and information obtained from interviews with P-D and SKK personnel indicate that the construction procedures in the two remined areas were similar. Excavating and installing temporary support during the heading operation at the VAL tunnel began on August 31, 1994 and lasted until September 19. The bench operations at the remined area began on September 21, and the 164-ft-long remining operation was completed on September 28. Thus the resupport operation took 29 days to complete. In contrast, excavating and installing temporary support during the heading operations at the HAL tunnel began on June 15, 1995 and the collapse occurred on June 22. The last segment, no. 101, of the 80-ft-long remining operation was saw cut, and all other remined segments had steel sets placed in the heading at the time of the collapse. The soil in the crown of the VAL tunnel was exposed as much as four times longer than that at the HAL tunnel. These observations show that the system, as constructed, was adequate for dry Puente formation.

6.1.2 Stratigraphy - Based on the face records made while excavating the HAL and VAL tunnels, the stratigraphy was similar at the two areas, yet several differences could be observed. At the HAL tunnel, the ground was easily excavated and consisted of T_{pw}, T_{po}, and T_{pf}, with an increasing amount of T_{pw} encountered in the face as the excavation proceeded west. The T_{pw} consisted of friable, interbedded siltstone/claystone with thin, fine grained sand beds. The apparent dip of the bedding was approximately 65 to 70 degrees from the horizontal. The T_{pw} was jointed, highly to moderately sheared, and slickensided. At the VAL tunnel, the ground was also easily excavated and consisted of alluvial and colluvial fine grained soils and T_{pw}. The alluvium was a reddish brown, medium stiff to stiff clay with low plasticity. The colluvium was similar to the alluvium, but with inclusions of weathered Puente formation in quantities of 10 to 20%. The T_{pw} consisted of very weak, highly weathered siltstone/claystone. The bedding was mostly horizontal, but difficult to distinguish because of the degree of weathering. Design strengths of T_{pw} was 1500 psf and that of both the T_{po} and T_{pf} was 5000 psf, according to the GDSR.

There are two important differences between the ground encountered at the HAL and VAL sites. First, the Puente formation at the HAL site was more jointed and bedded than that at the VAL site, where the bedding was difficult to distinguish because of the degree of weathering. In fact, the face records classified the ground in the upper part of the face of the VAL tunnel as alluvium and colluvium, and not Puente formation, as suggested by the results of the borings in that area given in the GDSR, a testament to the high degree of weathering at that area. The second main difference was the orientation of the bedding. At the HAL site, the bedding dipped 65 to 70 degrees to the north, whereas the bedding was dipped less than 10 degrees at the VAL site.

These conditions and the soil strength test data (Section 3.2.5) were known when the remining submittal was prepared and approved. They were based on face records of already-constructed tunnel or were contained in the GDSR. Apparently, the differences in the stratigraphy at these sites were not considered to be significant because the submittal pertained to any remining within the Puente formation. The strength data would have impacted bearing capacity calculations for the foot blocks, but no such calculations were included in the remining submittal.

<u>6.1.3 Groundwater conditions</u> - The basic model of groundwater flow consists of flow in the alluvium in an outward direction from Barnsdall Park. It is the topographic high as well as the piezometric high in the area, and serves as a recharge area for the ground water. The thin sand beds and the bedding, joints and secondary structure of the siltstone and claystone form the hydrologic pathways from the Alluvium to the Puente Formation. Thus, flow through the Puente is primarily through the structure of the soft rocks; the more massive the rock, the less permeable it would be.

In February 1994, when SKK made their initial remining submittal, the water level on OW-18 was approximately el. 343 ft (Fig. 3.3). When the submittal was approved in May 1994, the water level had risen to el. 353 ft, and by the start of remining in June 1995, the water had risen to el. 360 ft. Assuming a gradient of 0.022, the water levels above the remined area at each of these times are given in Table 6.1. These results indicate the ground water would be as much as 16 ft above the crown at while the design of the remining scheme was underway, and by the time remining began, the water level was 24 ft above

Table 6.1 Computed Water Levels at HAL and VAL Remined Areas

Time	HAL water el.(ft)	HAL ht. above crown(ft)	VAL water el.(ft)	VAL ht. above crown(ft)
SKK initial submittal	349.6	7.6	340.1	10.1
SKK submittal approved	358.3	16.3	353.4	23.4
Remining	365.8	23.8	351.6	21.6

the crown.

Similar estimates were made for conditions at the VAL remined area. There is more uncertainty in these estimated levels because of the greater distance to the observation wells near the remined area and the questions concerning the integrity of the bentonite seal at OW-16 (Section 3.2.4). In February 1994, when SKK made their initial remining submittal, the water level in OW-16 was approximately el. 346.6 ft (Fig. 3.5). When the submittal was approved in May 1994, the water level was el. 359.9 ft, and by the start of remining in September 1994, the water level was el. 358.1 ft. Assuming a gradient of 0.013, based on water levels between OW-16 and OW-14 before the dewatering system near OW-14E was in operation, the water levels in the area at each of these times are given in Table 6.1. These water levels were computed on the assumption that the dewatering near OW-14E had no effect on the water levels at the remined area. If it is assumed that the piezometric variation between OW-16 and 14E was linear at the time of the VAL remining, the corresponding water level at the VAL remined area would be close to the crown of the tunnel. The most likely scenario would place the water level somewhere between these limits, but conceivably as high as 22 ft above the crown. Thus the water levels were probably similar at the two sections when the remining was done at each.

As shown in Table 6.1, analysis of the data available prior to HAL remining indicates that the water level was above the tunnel, as it was when the tunneling was done. The tunnels were driven successfully through the Puente formation with perched water in the Alluvium because of the low permeability of the siltstone and claystone of the Puente formation. The primary difference between the initial drives and the remining operations is the initial drive fully supports the ground, except at the face, which can be partially supported. Also, the seepage face continually changes position as the tunnel is advanced in the initial drive. In contrast, once the concrete segments have been removed in the remined area, the seepage face remains between the crown and the springline until the ribs and lagging have been concreted. Thus there is more time for seepage to affect remining operations.

The fact that the remined section at the VAL tunnel was exposed for a month without evidence of seepage indicates that the earth at this site remained sufficiently impervious such that water did not flow to the exposed surfaces. However, this was not the case at the HAL section, where seepage was observed 7 days after the start of the remining operation. This behavior perhaps could have been anticipated based on the description of the excavated faces at each area (Appendix A). The excavated face was more massive with less bedding at the VAL site as compared to the HAL site. Additionally the bedding dipped 10 degrees at the VAL site, but dipped as much as 65 to 70 degrees at the HAL site, providing more direct access to the overlying, water-bearing alluvial soils. This more extensive bedding would have allowed water to flow much easier at the HAL site, and thus a seepage face would develop much sooner, as was observed. Alternatively, the down-gradient alluvial channel was about 250 ft and 400 ft from the HAL and the VAL remined areas, respectively. Thus if the primary seepage path was horizontal, then the HAL site would again have been more conducive to seepage than the VAL site.

6.1.4 Summary - Similar procedures were used to remine sections along reaches of the VAL and HAL tunnels. The conditions at the remined areas also were similar in that the material excavated was primarily siltstone and claystone of the Puente formation and that the original tunneling through the two areas proceeded smoothly with no observations of water in the face. However, the Puente formation at the HAL site was more bedded and jointed than that at the VAL site, and included bedding planes that dipped 65 to 70 degrees as compared to the 10 degree dip at the VAL site. After tunneling had loosened the joints, the increased permeability along the joints would provide a pathway from the overlying perched water (or the down-gradient alluvial channel in the scenario of predominantly horizontal bedding) to the exposed rock in the crown during remining operations. With an estimated 24 ft of water head and more pervasive bedding at the HAL site, seepage forces were able to develop and load the steel sets at the HAL site. While the estimated levels of water at the VAL site may have been similar to those at the HAL site during remining (the water levels may have been lower), the less pervious and more horizontal bedding resulted in conditions where seepage did not develop at the exposed ground during remining.

6.2 Design of Resupport System

The approach used to design the support system apparently was to size the temporary structural support elements for a load corresponding to 60 ft of overburden pressure, the same philosophy used in design of the precast segmental concrete liner, and to rely of the inherent strength of the Puente formation to temporarily support the steel sets as the heading was made. If the ground was self-supporting, then after the segments were removed, small inward movements at the exposed surface would occur and the amount of load on the steel sets would be a function of how much of this small movement had occurred before the lagging was placed, and thus before the ground came in "contact" with the steel sets. If all the movement had occurred, then the only load on the foot block would be that from the self-weight of the steel sets. The foot blocks were of sufficient size to resist these loads. If all the movement had not occurred before the lagging was placed, then the foot blocks would have to resist this resulting load, as well as that from the self-weight of the steel sets. A scenario which resulted in small loads on the foot blocks was reasonable for remining in "dry" Puente formation based on the response of the Puente during the tunneling operations for the four Vermont and Hollywood tunnels; that is, good stand-up time was observed throughout tunneling operations and water was not encountered at the face. Indeed, the best indicators of future performance in underground construction are full scale field tests, which, in this context, the tunneling operations can be considered. One can not design the supports for all stages of temporary construction without recognizing the benefits of arching within the Puente formation. However, when considering the arching, it is prudent to include a component of earth loading in the design of the foot blocks.

Evaluating the bearing capacity of the earth on which a foot block rests is a standard step in design of rib linings, as noted in Proctor and White.[7] Safe support of the foot blocks is essential during the heading operation. The design of the resupport system was deficient in that the bearing capacity of the foot blocks apparently was never considered. Therefore, the question of appropriate loading for the steel sets under the temporary loading conditions never was an issue. While it is unrealistic to require the foot blocks to carry 60 ft of overburden pressure, it is also unrealistic to *design* the foot blocks only for self-weight of the steel sets without provisions for earth loading. In addition, the bearing capacity

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[7] Proctor, R.V. and White, T.L., "Earth Tunneling with Steel Supports" Commercial Shearing Inc. 1977, pp. 143 and 161.

of the concrete wall beam was never considered in the calculations accompanying the remining submittal. Only the contact pressure (4878 psi) from the full design overburden pressure of 7200 psf was calculated.

The ultimate load the foot blocks can sustain, based on consideration of the bearing capacity of the Puente formation at the remined area, was computed to vary between 9 to 23 kips, depending on the assumptions concerning the shear strength of the Puente and the size of the timber blocks (Table 5.1). This capacity is less than the 307 kip load corresponding to the design pressure of 7200 psf, the 81 kip load corresponding to the SKK-anticipated pressure of 1888 psf, and the 58 kip load corresponding to the WJE estimate of 1340 psf. Nevertheless, the system as it was constructed at the 164 ft long VAL remined area performed adequately, a testament to the self-supporting nature of the Puente formation. The presence of water and the resulting seepage forces, in combination with relatively small earth pressures, were all that was required to initiate a failure of the temporary support system.

6.3 Adherence to Approved Procedures

Review of the P-D daily reports made while remining both the HAL and VAL remined sections and information obtained from interviews with P-D and SKK personnel indicate that the construction of the remined areas was conducted in substantial accordance with the procedures outlined in the SKK submittal. Exceptions to the outlined procedures included: (1) all segments were saw cut after the split bolts were placed at the VAL remined area and before they were placed at the HAL remined area, rather than being sequentially saw cut as implied by the submittal, (2) the wood lagging was placed against the excavated ground and blocked against the steel sets, rather than placed inside the flanges of the steel sets as shown in the SKK submittal, (3) the concrete wall beam was placed along the first several foot blocks before the entire bench was excavated and the top quadrant of the tunnel supported, rather than waiting until the entire bench was excavated, (4) one split bolt was used in each segment rather than two, as stated in the submittal, and (5) no split bolts were placed through the Dutchman, but rather the Dutchman was bolted to the concrete segments. The order of the saw cutting, the number of split bolts, and the type of connection at the Dutchman most likely had little impact on the system. The second and third differences resulted in construction procedures which were better than those proposed in the SKK submittal in that they provided a system with better support than originally proposed.

6.4 Influence of a Preexisting Leak in the Water Main

Seepage was first observed at about 12:30 a.m. The water source was thought by P-D personnel to be a leak in the 10-in. main. A preexisting leak in the water main is unlikely considered for three reasons: (1) the surface settlements prior to movements occurring on June 22, were probably too small to cause cracking of the water main, (2) there was no indication of water on the street surface which usually accompanies significant water main leaks, and (3) the metallographic examination of the salvaged water main segments did not show any evidence of a preexisting leak (relatively few sections were recovered, however). Had a small leak occurred, the discharge would have added to the ground water in the area. This contribution would be very small compared to that from the recharge from the record amounts of rainfall which occurred in January 1995. Therefore, the possible presence of the preexisting leak is not relevant.

6.5 Failure Sequence

The collapse of the remined area at the HAL tunnel was initiated by a bearing failure of the foot blocks for the steel sets. The large settlements, estimated from 12 to 18 in., are evidence of this bearing capacity failure. It was reported that the foot blocks settled 6 to 12 in. by as early as 2:30 a.m., and presumably continued to settle as the night progressed. Loads on the steel sets initially were induced by the effects of seepage and, to a lesser extent, by stress redistribution after removing the concrete segments. As the foot blocks settled, the rock in the crown began to ravel. As the ravelling progressed, more and more load was transferred to the steel sets and remaining portions of the saw-cut concrete segments. The movements of the foot blocks resulted in a flattening of the crown of the tunnel. These large deep-seated movements at the crown would have resulted in settlements at the ground surface, which would have induced stresses in the water main, and eventually would cause the pipe to rupture. This raveling process continued as the seepage face extended from west to east in the exposed rock in the crown area. The evidence suggests that the pipe broke after the foot blocks settled 12 to 18 in. and just before the tunnel collapsed. The collapse occurred at 6:10 a.m., and the LADWP records indicate that the water main ruptured 10 minutes earlier. Apparently, the sudden influx of water at a rate of about 14 cfs directly lead to the first collapse at 6:10 and development of the sinkhole at 6:15. Water and earth from this first collapse flowed to the west beyond CP 28 and into the HAR tunnel. Had the water main been shut off before it ruptured, damage may have been limited to large ground movements in the tunnel and resulting surface subsidence. It is also conceivable that the water main broke as the sinkhole developed. However, it is unlikely that a flow of earth and water for several hundred feet to the west could occur without the prior addition of large quantities of water from the broken main.

After this first collapse, water from the broken main filled the resulting sinkhole. At 11:35 a.m., this water broke through the remaining portion of the remined area and resulted in a surge of water, earth, and debris eastward through the HAL tunnel, past the Barnsdall shaft, and into the VAL tunnels. Had the water main been shut off before the initial collapse, this second collapse would not have occurred.

CHAPTER 7 - CONCLUSIONS

7.1 Failure Sequence

The collapse of the remined area at the HAL tunnel was initiated by a bearing failure of the foot blocks for the steel sets. Loads on the steel sets initially were induced by the effects of seepage and, to a lesser extent, by stress redistribution after removing the concrete segments. Personnel at the site believed the source of the seepage was a leak in the 10-in. main above the tunnel. LADWP was asked to shut off the main at about 3:00 a.m. This investigation indicates that a preexisting leak is unlikely. Had a leak occurred, it would have made only a small contribution to the groundwater in the area. The ground water level was already well above the tunnel crown due to heavy rainfall in the preceding months. Therefore, the possible presence of a preexisting leak in the water main is not relevant.

As the foot blocks settled, the rock in the crown began to ravel. As the ravelling progressed, more load was transferred to the steel sets and remaining portions of the saw-cut concrete segments. These movements of the foot blocks resulted in a flattening of the crown of the tunnel. These large deep-seated movements would have resulted in settlements at the ground surface, which would induce stresses in the water main, and eventually cause the pipe to rupture. This raveling process continued as the seepage face extended from west to east in the exposed rock in the crown area. The water main probably broke before the collapse of the tunnel supports since the collapse reportedly occurred at 6:10 a.m. and the LADWP records indicate the water main ruptured 10 minutes earlier. Apparently, the sudden influx of water at a rate of about 14 cfs directly lead to the first collapse at 6:10 and development of the sinkhole at 6:15. Water and earth from this first collapse flowed to the west beyond CP 28 and into the HAR tunnel. Had the water main been shut off before it ruptured, damage may have been limited to large ground movements in the tunnel and resulting surface subsidence. After this first collapse, water from the broken main filled the resulting sinkhole for at least 45 minutes until the main was shut off. At 11:35 a.m., this ponded water broke through the remaining portion of the remined area and resulted in a surge of water, earth, and debris eastward through the HAL tunnel, past the Barnsdall shaft, and into the VAL tunnels.

7.2 Support System Design

The approach used to design the support system apparently was to size the temporary structural support elements for a load corresponding to 60 ft of overburden pressure, the same philosophy used in design of the precast segmental concrete liner, and to rely on the inherent strength of the Puente formation to temporarily support the steel sets as the heading was made. Based on the observed performance of the Puente formation at the excavated face in the remined areas during tunneling operations, the ground was self-supporting for the time it was exposed during tunneling.

The design of the resupport system was deficient in that the bearing capacity of the foot blocks apparently was never considered. Safe support of the foot blocks is essential during the heading operation. Evaluating the capacity of a foot block is a standard step in design of rib linings. In addition, the bearing capacity of the concrete wall beam also was never considered in the remining submittal. Therefore the question of appropriate temporary loading for the steel sets apparently never was an issue in the design. While it is unrealistic to require that the foot blocks carry 60 ft of overburden pressure, it is also unrealistic to *design* the foot blocks without provisions for any earth loading.

7.3 Geologic and Groundwater Conditions at the HAL and VAL Sites

The design of the resupport system did not incorporate information obtained in the face records and the water level data relating to the Puente formation at the remined areas. Face records indicated the Puente formation at the HAL site had more well-formed secondary structure than that at the VAL site. Bedding planes dipped 65 to 70 degrees at the HAL location, as compared to the near horizontal bedding at the VAL section. The Puente at the HAL site could transmit water more freely than at the VAL site, especially after loosening the joints and bedding planes during initial tunneling. The failure occurred after very high monthly rainfalls were recorded in Los Angeles. The significance of the water level rises at OW-18 was not appreciated, and its consequences not anticipated.

7.4 Adherence to Approved Remining Procedures

The construction of the remined areas was conducted in substantial accordance with the procedures outlined in the SKK submittal. Minor variations occurred which did not significantly affect the performance.

7.5 Conclusion

In conclusion, the tunnel failure is due to a deficient design of the temporary support for the steel sets. This design deficiency became apparent when seepage within the Puente formation was encountered during remining. Flow from the 10-in. water main did not contribute to the initial foot block failure, but damage would have been significantly reduced if the water main were shut off earlier.

69937

APPENDIX A

Alignment Repair

Original Submittal (Rejected)

R81 Metro Red Line Seg-2

FILE CO[illegible]

Original submittal & comments: From Girish Roy - 9/20/95

SUBMITTAL: B251
02344-1.3-5.00

CDRL #: 696

DATE TO RE: 03/21/94

FILE: CA160 02344-1.3-5.00

SUBMITTAL REVIEW RESPONSE

REVIEWER TO RE

TO: S. J. CALVANICO, RESIDENT ENGINEER

CC:

FROM: B. Ghadiali, Proj Unit Manager, B251

CONTRACT #\TITLE: B251 / VERMONT/HOLLYWOOD TUNNEL

SUBJECT: SUBMITTAL 02344-1.3-5.00
ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

CATEGORY	ISSUES/DISCIPLINES	ORIGINATOR
SD CLC	DE400 - FACILITY	SHEA-KIEWIT-KENNY J.V.

DOCUMENTS SUBMITTED:

TYPE	ID NUMBER	REV. ID	TITLE/COMMENT	DISPOSITION
H	COM PANTEX		DRAWINGS AND CALCS FOR RESUPPORT STL. SETS	REJ

REVIEW DISTRIBUTION:

REVIEWER	DUE DATE	REVIEW DATE	COMMENT	DISPOSITION
SMIRNOFF	03/25/94	03/17/94	REJECTED	REJ

PROCESS DATES:

FROM CONTRACTOR:	02/24/94	CRE-1938	RESPONSE RETURNED:	03/17/94
TO REVIEW:	02/25/94		RESPONSE TO RE:	03/21/94
RECEIVED BY REVIEW:	02/25/94		RECEIVED BY RE:	
DISTRIBUTED:	02/25/94		RESPONSE TO CONTRACTOR:	REC-

REVIEW RESPONSE: REJ

REJECTED; REVISE AND RESUBMIT.

PLEASE REFER TO ATTACHED REVIEWER'S COMMENT LETTER DATED MARCH 17, 1994.

RESPONSE BY:

APPROVED BY: B M Ghadiali. 3/21/94.

ROBERT TOMAS
03/21/94 08:06
rev 2.4 11/05/93 [illegible]

R81 Metro Red Line Seg-2

SUBMITTAL: B251-02344-1.3-5.00
CDRL #: 696

SUBMITTAL DOCUMENT LIST

JOB NO: R81 - B251
FILE: CA160 B251-02344-1.3-5.00

CONTRACT #/TITLE: B251 / VERMONT/HOLLYWOOD TUNNEL

REFERENCE: B251-CRE-01938 RECEIVED 02/24/94

SUBJECT: SUBMITTAL B251-02344-1.3-5.00 CDRL: 696
ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

RE-SUBMIT √'D ITEMS NO LATER THAN: N/A

RESUBMIT?	TYPE	ID NO:	SHEET	DESCRIPTION	DISPOSITION
√	DWG	COM PANTEX		DRAWINGS AND CALCS FOR RESUPPORT STL. SETS	REJ

TOTAL: 1

*** END OF DOCUMENT LIST ***

ROBERT TOMAS
03/21/94 08:05
rrev 2.2 01/27/94 ls

ENGINEERING MANAGEMENT CONSULTANT
an association of
Parsons Brinckerhoff Quade & Douglas, Inc.
Daniel, Mann, Johnson, & Mendenhall
ICF Kaiser Engineers (Calif.) Corp.
Escudero-Fribourg Architects
Jenkins Gales Martinez, Inc.
The Nettleship Group, Inc.

FILE COPY

707 Wilshire Blvd.
Suite 2900
Los Angeles, CA 90017
(213) 362-4700
Fax: (213) 362-3112

MEMORANDUM

Date: March 17, 1994

To: Salvatore Calvanico

From: Timothy P. Smirnoff TPS

Subject: **Resupport of Tunnel Arch Remining Submittal 02344-1.3.5.0**

We have reviewed the subject submittal and offer the following:

1. The design is appropriate if only horizontal remining and resupport is required. Quick review of the remining cross-sections indicate remining will require section with vertical offset as well for which such a scheme is not appropriate. Provide details for the full range of conditions encountered.

2. The precast concrete segment design load differs greatly from that used by the structural steel designer (Commercial). The two designs must be compatible. It appears the structural steel ribs would be adequate, provided bearing plates and details are sized accordingly but the wall plate and intermediate support details will require modification.

3. No calculations are provided for the rock anchors. Please provide calculations and details.

4. These details are appropriate for sections wholly with Puente.

5. Mining of such a section require careful control of exacavation and control of ground. Provide detail excavation sequence, intermediate support details and face stability methods.

6. Provide post and foot plate design and bearing capacity calculations.

This submittal lack primary calculations and necessary support details which are critical to the safe execution of this remining operation. Please provide these details and calculations for review and approval.

TPS:rr\920482

R81 Metro Red Line Seg-2

SUBMITTAL: B251
02344-1.3-5.00
CDRL #: 696

SUBMITTAL REVIEW REQUEST

RE TO REVIEWER

CRE #: B251-CRE-01938
DATE: 02/25/94
NEED DATE: 03/25/94

TO: EMC
ATTENTION: B. Ghadiali, Proj Unit Manager, B251
FROM: SALVATORE CALVANICO, RESIDENT ENGINEER

cc: ___ SQ020.1
___ CA160 02344-1.3-5.00
___ J. ADAMS/R. DAMES, CONSTRU
___ DCC

CONTRACT #\TITLE: B251 / VERMONT/HOLLYWOOD TUNNEL

SUBJECT: SUBMITTAL 02344-1.3-5.00
ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

CATEGORY	ISSUES/DISCIPLINES	ORIGINATOR
SD CLC		SHEA-KIEWIT-KENNY J.V.

Your prompt attention to this submittal review is appreciated.

DOCUMENTS SUBMITTED: # SEPIAS: ________ # PRINTS: ________ # OTHER: 5 EA **DISPOSITION**

TYPE	ID NUMBER	REV. ID	TITLE/COMMENT
H	COM PANTEX		DRAWINGS AND CALCS FOR RESUPPORT STL. SETS

REVIEW DISTRIBUTION: **DISPOSITION**

REVIEWER	DUE DATE	COMMENT

PROCESS DATES:

FROM CONTRACTOR:	02/24/94 CRE-01938	RESPONSE RETURNED:
TO REVIEW:	02/25/94	RESPONSE TO RE:
RECEIVED BY REVIEW:		RECEIVED BY RE:
DISTRIBUTED:		RESPONSE TO CONTRACTOR: REC-

REVIEW RESPONSE: (TO BE COMPLETED BY REVIEWING ORGANIZATION)

See attached Memo
Submittal Rejected!

RESPONSE BY: [signature] 3-16-94.
APPROVED BY:

J. HARRINGTON
02/25/94 08:04
rev 2.3 10/20/93 ls

CONTRACTOR'S SUBMITTAL TRANSMITTAL

VERMONT/HOLLYWOOD TUNNEL — PARSONS-DILLINGHAM CONTRACT NO. B251

FOR ALL CONTRACTOR SUBMITTALS, INCLUDING SHOP DRAWING, SAMPLES CALCULATIONS, WORKING DRAWINGS, ETC.	DATE 2-24-94	RECEIVED FEB 24 1994	DISTRIBUTION
TO: PARSONS-DILLINGHAM 4021 ROSEWOOD AVENUE, #103 LOS ANGELES, CA. 9004-2923 ATTN: STEPHEN NAVIN		CRE NUMBER R.E. CRE - 1938 CDRL NUMBER CDRL - 696	Cavaniglo SQQ0.1 QA 160
FROM: SHEA-KIEWIT-KENNY JV P.O. BOX 27097 LOS ANGELES, CA. 90027-0097		SUBMITTAL NUMBER B251-02344-1.3-5.00	02344-1.3-5.00 B. Chadwali

RE: ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

GENTLEMEN: WE HEREWITH TRANSMIT THE FOLLOWING ITEMS:

✓ SHOP DRAWINGS
___ PRINTS
___ PLANS
___ SAMPLES/PRODUCT DATA
___ COPY OF LETTER
✓ CALCULATIONS
OTHER: ___

DESCRIPTION/TITLE	DRAWING, SPE OR ITEM #	DATE	REVISION
RESUPPORT ARCH DRAWING	C-819-3A-9	2-24-94	0
CALCS. FOR REALIGNMENT STL. SETS	-	2-24-94	0

THESE ARE TRANSMITTED AS INDICATED BELOW:

X FOR APPROVAL
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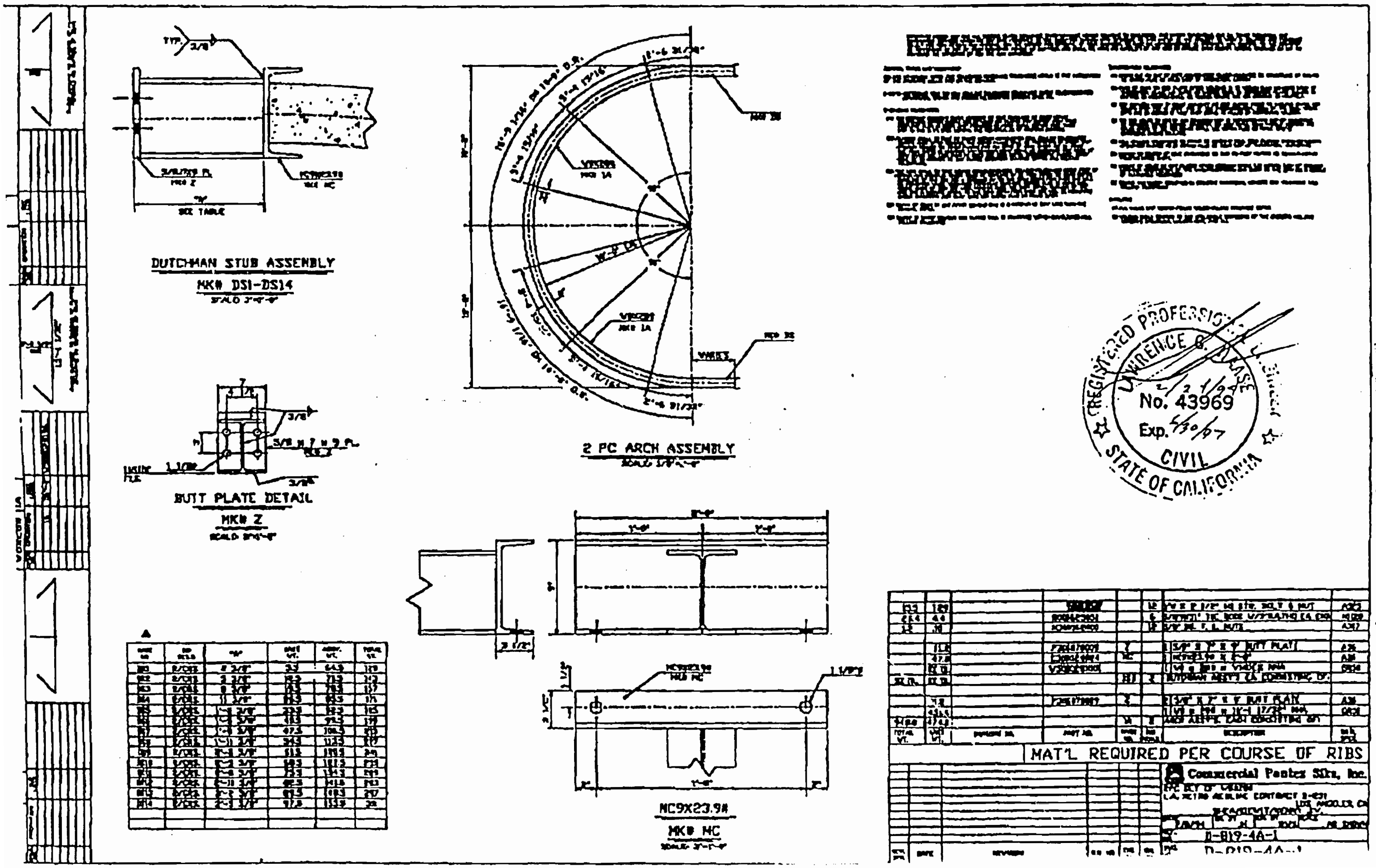
___ CORRECTED AND RETURNED
___ FOR YOUR USE
___ FOR REVIEW AND COMMENT

REMARKS

CONTRACTOR'S AUTHORIZED REPRESENTATIVE

Stuart [illegible]

B251-0234-13-5.00



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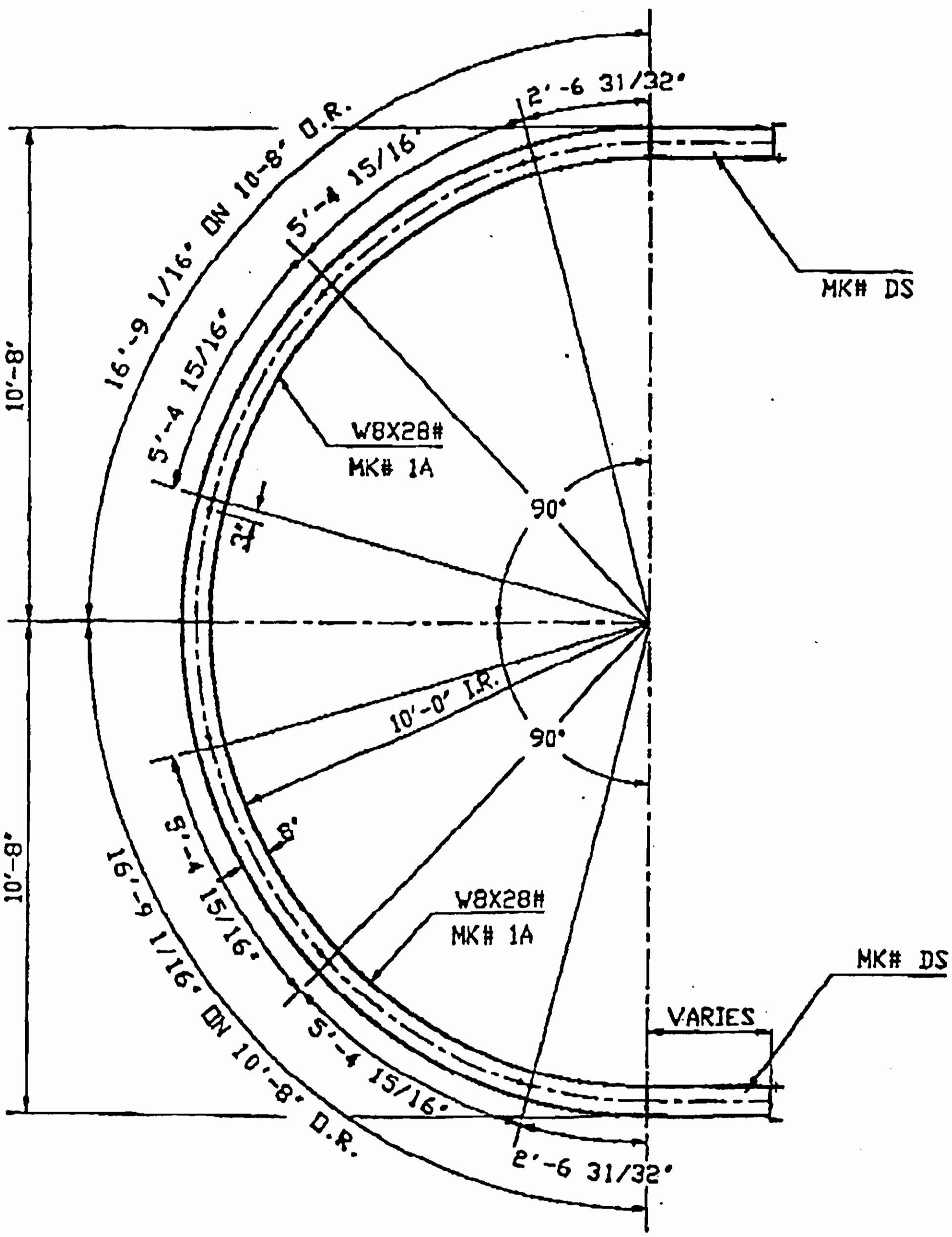

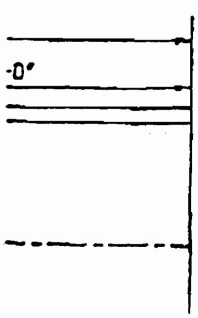

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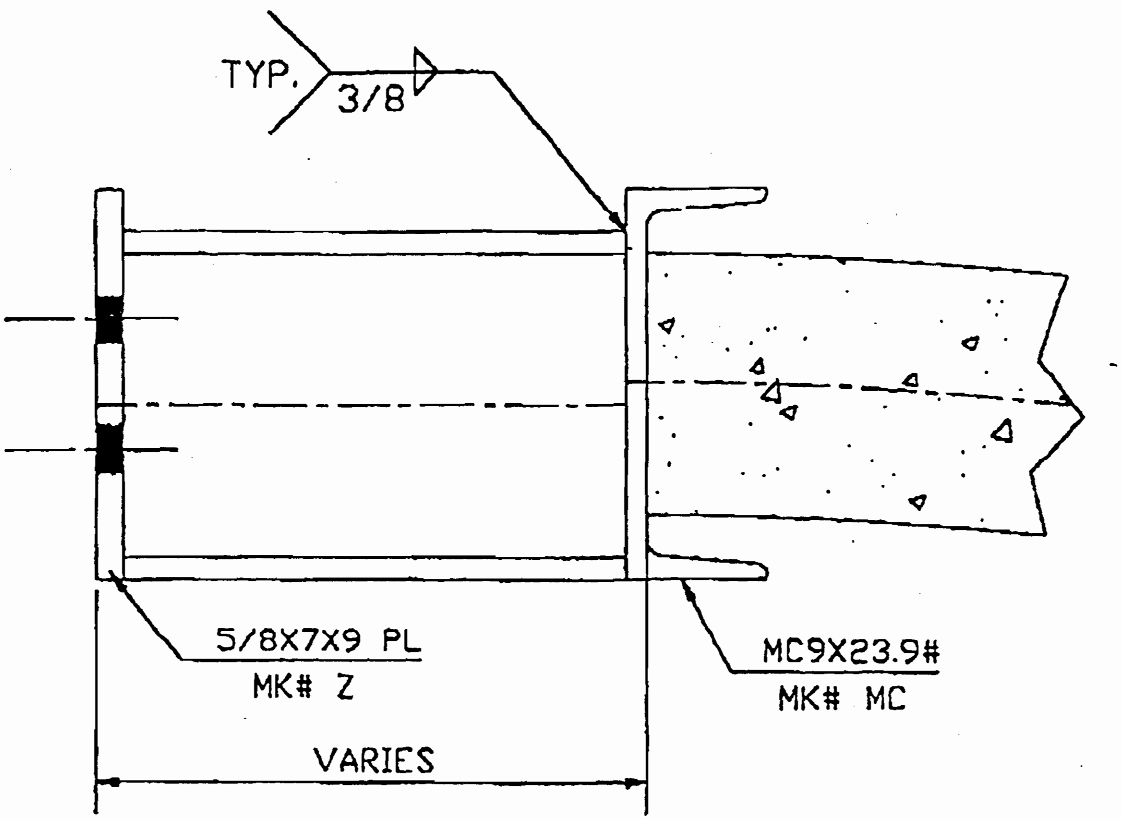

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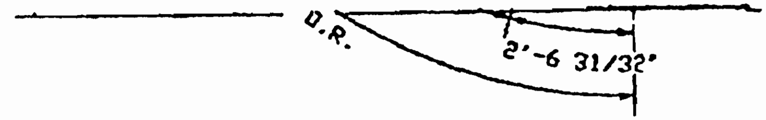

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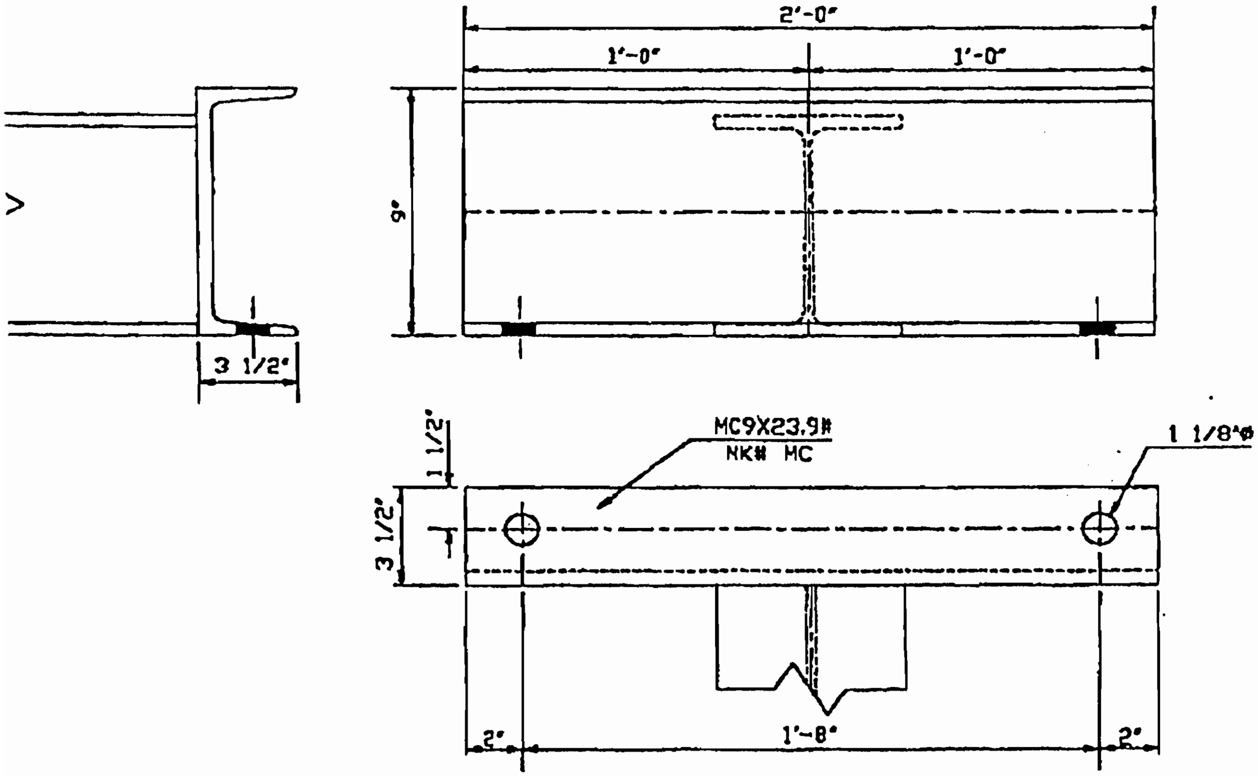

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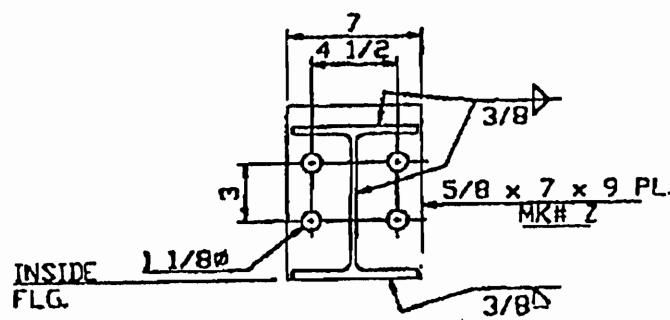

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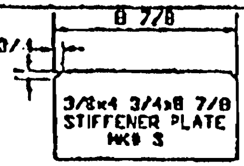

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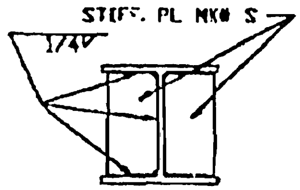

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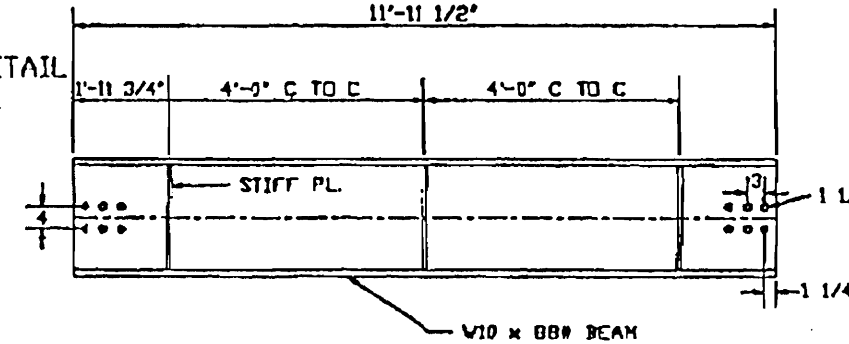

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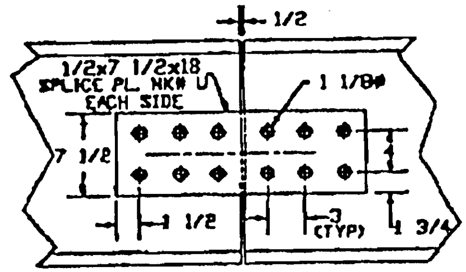

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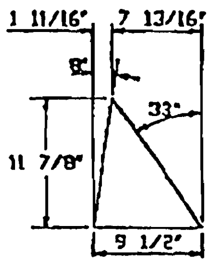

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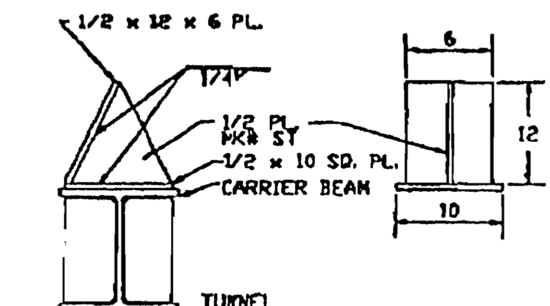


BRACKET DETAILS
MK# BK

SHOP NOTE: SHOP IS TO DEVELOP BRACKET TO BE FIELD WELDED TO RING AND CARRIER BEAM ABOVE SPRING LINE.

TOTAL WT.	UNIT WT.	DRAWING NO.	PART NO.	MARK NO.	NO REQ'D	DESCRIPTION	MAT'L SPEC
	1.35				24	1" x 2 3/4" H. S. B & N	A325
	8.0		P305093012	ST		1 1/2 x 11 7/8 x 9 1/2 PL.	A36
	14.1		P305010010			1 1/2 x 10 SQ. PLATE	A36
	10.2		P305061012			1 1/2 x 6 x 12 PLATE	A36
				BK	12	BRACKET ASSY'S EACH CONSIST OF:	
	4.4		P303040009	S		6 3/8 x 4 3/4 x 8 3/4 STIFF. PLS.	A36
	19.1		P305071018	U		2 1/2 x 7 1/2 x 18 SPLICE PLS.	A36
	1052.3					1 W10 x 88# x 11'-11 1/2" NNA	[illegible]
4467.6	1116.9			9B	4	CARRIER BEAM ASSY'S EACH CONSIST OF:	

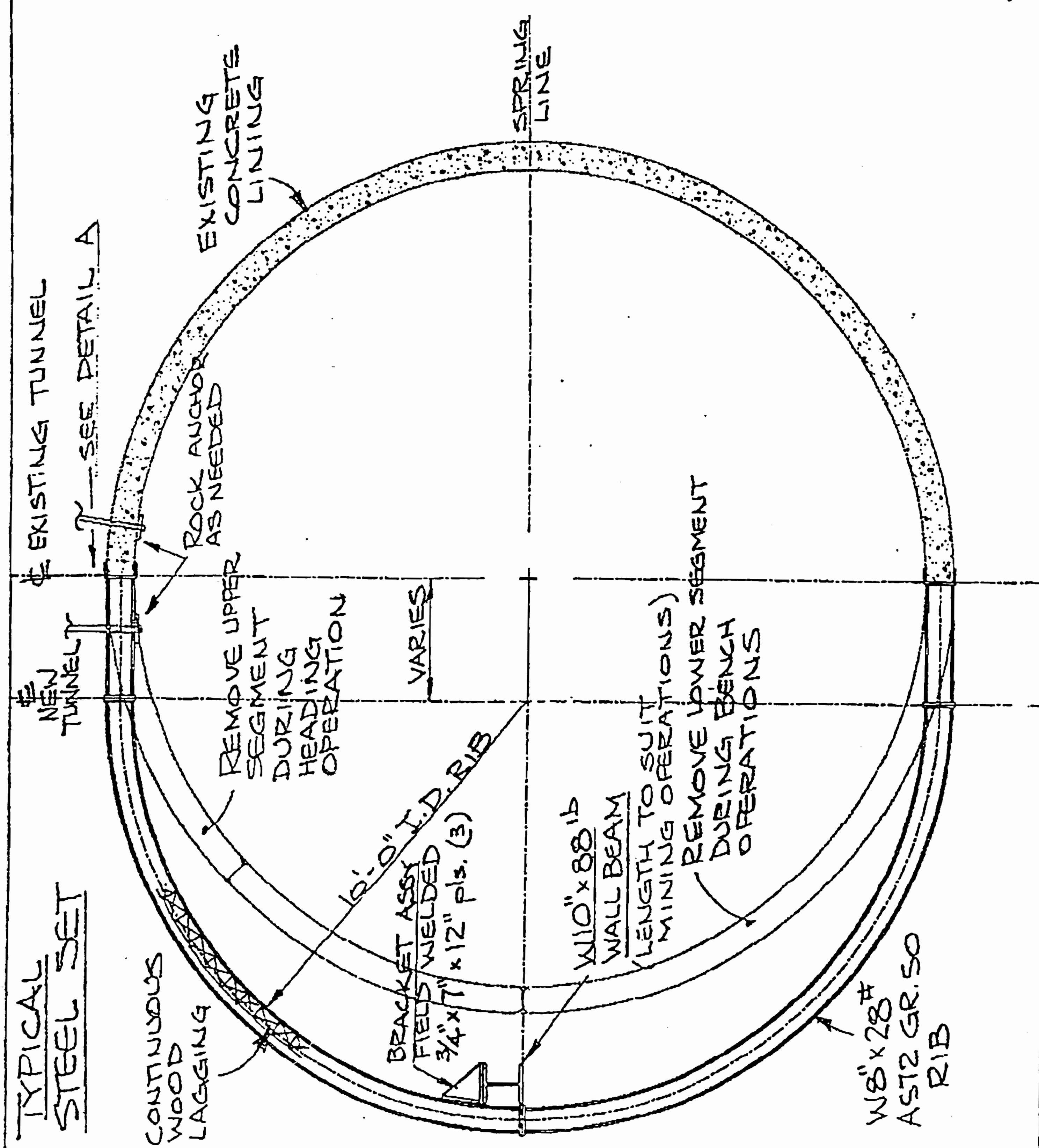
MAT'L REQUIRED PER CROSS PASSAGE

REV. SYM.	DATE	REVISION	B.M. NO.	DRG. BY	CK BY
2	2/9/94	ADDED DETAIL FOR PLATE MK# ST	4793	JH	RVS
1	8/12/93	REMOVED STRUT BEAM DETAIL	4728	DC	RVS

Commercial Pantex Sika, Inc.
CARRIER BEAM AND ADDITIONAL PIECES
METRO REDLINE B251
LOS ANGELES, CA.
SHEA/KIEWIT/KENNY J.V.

DATE 8/4/93 | DR. BY DC | CK. BY RVS | SCALE NTS
REF. NO.
DWG. NO. C-819-3A-9

Commercial Pantex Sika, Inc.		L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA SHEA-KIEWIT-KENNY,JV
DATE 2-1-94 BY R.SMITH	SHEET OF	

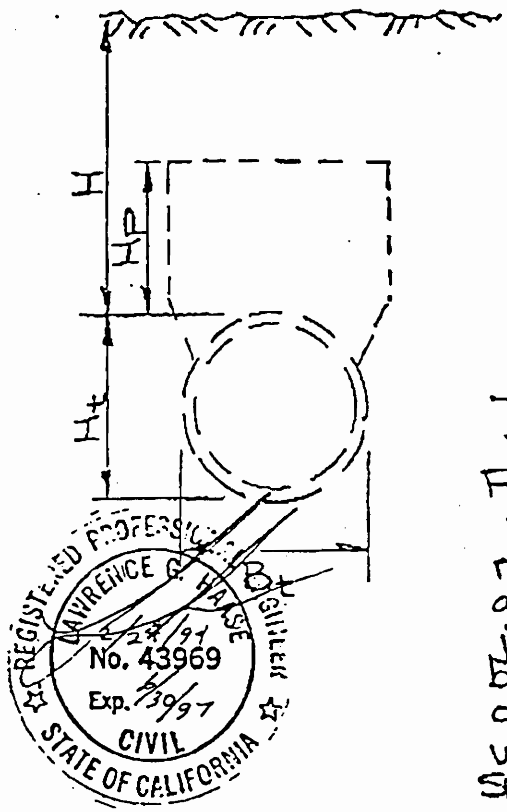


IMPORTANT NOTICE: Any and all designs, plans, drawings, specifications, advice relative to geological and safety conditions and all other technical and engineering services which we may have furnished or may hereafter furnish with reference to the matter or the project to which it relates are furnished solely for your review and approval and the approval of your engineers. We make no representation or warranty with respect to the accuracy or sufficiency of any of such documents, advice or services, nor shall we have any liability of any kind or nature with respect hereto whether ...

Commercial Pantex Sika, Inc.	L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA SHEA-KIEWIT-KENNY, J.V.	DATE 2-1-94 R. SMITH PE SHEET 1 OF 4.

ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

ANTICIPATED LOAD (ON STEEL RIBS)



B_t = tunnel diameter ≈ 21.5 ft

H_t = tunnel height ≈ 21.5 ft

H = total height of overburden
= varies

H_p = load of rock or earth (ft) on roof of support

Tunnels in Puente

From "Rock Tunneling with Steel Supports", p. 91, Table 3, assume rock condition as moderately blocky and seamy. Moderately blocky rock contains joints & hair cracks, but the blocks between joints are locally grown together or itimately interlocked so vertical walls do not require lateral support. Spalling & popping of rock may occur.

Maximum height $H_p = 0.35(B_t + H_t) = 15.1$ ft

Vertical load $P_v = w H_p = (125 \text{ pcf})(15.1 \text{ ft}) = 1888$ psf

Tunnels in Alluvium

From "Earth Tunneling with Steel Supports", p. 70, Table 7.1, assume a running ground with medium density.

$$H_p = 0.40(B + H_t) = 0.40(21.5 + 0) = 8.6 \text{ ft}$$

For circular tunnels, $H_t = 0$

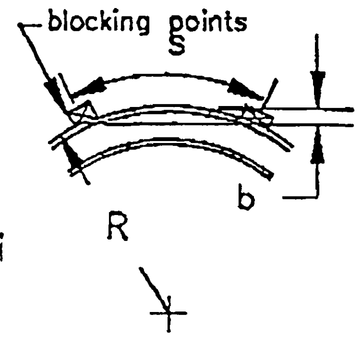
Vertical load $P_v = w H_p = (130 \text{ pcf})(8.6 \text{ ft}) = 1118$ psf

Commercial Pantex Sika, Inc.			L.A. METRO RED LINE B-251 LOS ANGELES, CA SHEA-KIEWIT-KENNY, J.V.
DATE 2-22-94	BY R.SMITH PE	SHEET 2 OF 4	

ALLOWABLE LOAD CALCULATIONS

W8 X 28 lb/ft (Grade 50 steel)

$A = 8.25 \text{ in}^2$
$S = 24.3 \text{ in}^3$
$I = 98.0 \text{ in}^4$
$r = 3.45 \text{ in}$
R = outside rib radius = 10.66 ft = 127.9 in
s = blocking point spacing = 0 in
(continuous contact with surrounding ground)
b = rise in chord between blocking pts =0 in
f_b = allowable bending stress = .75 F_Y = 37500 psi
(due to cold-working of steel)
E = modulus of elasticity of steel = 29×10^6 psi
S.F. = safety factor
d = rib spacing, c/c = varies



$$f_b = \frac{T_A}{A} + \frac{M}{S} \qquad M = 0.86\ b\ T_A$$

Allowable ring thrust $T_A = \dfrac{f_b \times A \times S}{S + .86 \times b \times A} = \dfrac{37500\ (8.25)\ 24.3}{24.3 + .86\ (0)\ 8.25}$

$= 309375$ lb / rib

Allowable thrust due to critical buckling :

$$T_{cb} = \frac{3 \times E \times I}{R^2 \times S.F.} = \frac{3\ (29 \times 10^6)\ 98.0}{(127.9)^2\ \ 1.25} = 416960 \text{ lb/rib}$$

Use $T = T_A$ or T_{cb} (the smaller value)

$$\text{Allowable load} = \frac{T}{d \times R} = \frac{309375}{(4.0)(10.66)} = 7255 \text{ PSF } (4'\text{-}0''\ c/c)$$

References:

Rock Tunneling with Steel Supports, Proctor & White,
pp 191-232

Theory of Elastic Stability, Timoshenko,
pp 297-300

Commercial Pantex Sika, Inc.	L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA SHEA-KIEWIT-KENNY JV	DATE 2-1-94 R. SMITH PE SHEET 3. OF 4.

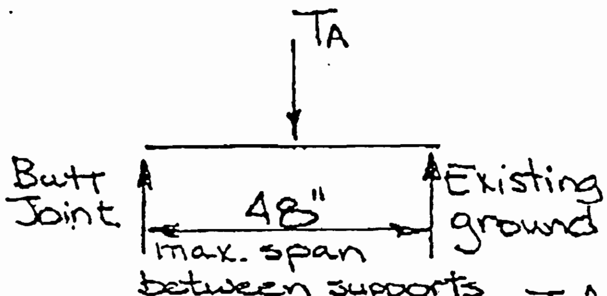
Wall beam calculations

P_V = Max. vertical load = 1888 psf ≈ 2000 psf

With sets 4'-0" c/c, arch thrust $T_A = P_V \cdot d \cdot R$.

$= 2000(4)(10.67')$

$= 85360$ lb



Actual moment $= \frac{T_A l}{4} = \frac{85360\,(48)}{4} = 1024320$ in-lb

Allowable moment $= f_b S_x$

Req'd section modulus $S_x = \frac{1024320}{(.67)36000} = 42.5\ in^3$

∴ W10" × 88# (A36) $S_x = 98.5\ in^3$

Reaction @ butt joint $= \frac{85360}{2} = 42680$ lb

Allowable bolt shear for (4) 1"φ A325 B&N = (4) 13400 = 53,600 lb
o.k.

Allowable weld shear = (2)(21000 psi)(.7071)(10-9/16-9/16)(1/4")
= 65892 lb o.k.

Commercial Pantex Sika, Inc.

L.A. METRO RED LINE
CONTRACT B-251
LOS ANGELES, CA
SHEA-KIEWIT-KENNY JV

DATE 2-1-94	BY R.SMITH PE	SHEET 4 OF 4

ALLOWABLE LOAD CALCULATIONS – WOOD LAGGING

As the lagging begins to deflect under vertical pressure, the soil will arch to a height h_A depending upon the angle of internal friction of the soil. An assumed arch shape is shown below. The soil will develop shear stresses along the dotted lines, with the arch load being carried by the lagging and transferred to the ribs. The soil load between arches will be carried directly by the ribs.

Assume lagging is 3" X 8" Allowable bending stress $f_b = 1400$ psi

Section modulus $S = \dfrac{b\ d^2}{6} = 12\ in^3.$

Allowable moment $M = f_b \times S$
$= (1400)(12.0)$
$= 16800$ in-lb

Moment for simple beam w/ triangular load:

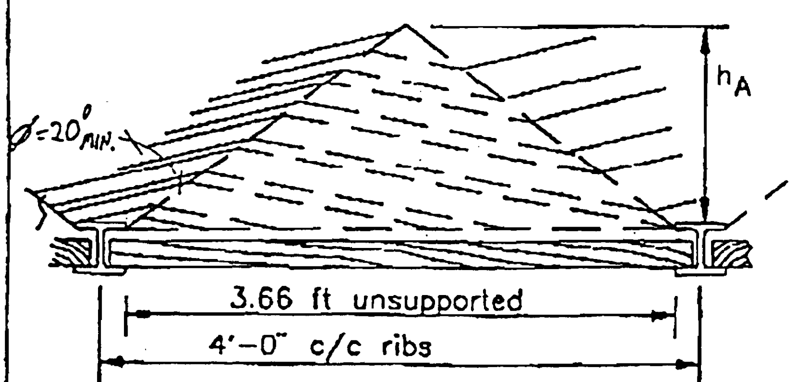
$$M = \frac{W \times l}{6} \qquad W = P_v \times l \times 0.67\ ft$$

$$M = \frac{P_v \cdot l^2 \cdot \frac{2}{3}}{6} = 16800\ in\text{-}lb = 1400\ ft\text{-}lb$$

$$P_v = \frac{6(1400)}{(.67)(3.66)^2} = 940\ psf$$

Amount of allowable overburden = 940 psf / 130 pcf = 7.2 ft of soil or rock

$h_A = \frac{3.66}{2} \tan 70° = 5.03' < 7.2'$ OK



Approved Submittal

Sheeler

PARSONS-DILLINGHAM

METRO RAIL CONSTRUCTION MANAGER

R81 Metro Red Line Seg-2

SUBMITTAL: B251-02344-1.3-5.01
CDRL #: 696

DATE: 05/19/94 JOB NO: B251

FROM: S. J. CALVANICO, RESIDENT ENGINEER

SEQ: R81-B251-REC-2104

FILE: CA160 B251-02344-1.3-5.01

LETTER OF TRANSMITTAL

RESPONSE TO SUBMITTAL

TO: SHEA-KIEWIT-KENNY J.V.
4773 HOLLYWOOD BLVD.
LOS ANGELES, CA 90027-0097

CC: ✓ B. Ghadiali, Proj Unit Manager B25
BOMI GHADIALI
J. DEVINE/R. DAMES, CONSTRUCTIO
D. MARTINEZ, SYSTEM ENGINEER
DCC SQ020.1

ATTENTION: ROBERT B. GORDON,PRJ.MGR.

CONTRACT #/TITLE: B251 / VERMONT/HOLLYWOOD TUNNEL

REFERENCE: B251-CRE-02346 RECEIVED 05/16/94

SUBJECT: SUBMITTAL B251-02344-1.3-5.01 CDRL: 696
ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

I AM SENDING YOU: [XXX] **ATTACHED** [ ] **UNDER SEPERATE COVER**

CONTRACT DRAWINGS	[ ]	RFI/C RESPONSE	[XXX]	**SUBMITTAL RESPONSE**	
SPECIFICATIONS	[ ]	CHANGE NOTICE	[ ]	OTHER:	
SKETCHES	[ ]	CHANGE ORDER			

ID NO:	DESCRIPTION		REV. ID	DISPOSITION
B251-02344-1.3-5.01	ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS		5.01	APP
	COM PANTEX	DRAWINGS AND CALCS FOR RESUPPORT STL. SETS		APP
	LETTER	SKK ALIGNMENT REPAIR FOR CONCRETE TUNNEL LINER		APP

DISPOSITION: APP - APPVD/ACCPTD AS SUBMITTED

RE-SUBMIT √'D ITEMS NO LATER THAN: N/A

Approved as Submitted.

RECEIVED
MAY 20 1994

SIGNED: J H Ostberg for
S. J. CALVANICO, RESIDENT ENGINEER

A joint venture of
The Ralph M. Parsons Company
De Leuw, Cather & Company
Dillingham Construction N.A., Inc
523 West Sixth Street
Suite 400
Los Angeles, California 90014
(213) 362-6000

J. HARRINGTON
05/19/94 14:29

B251-02344-1.3-5.01

Shea
Kiewit
Kenny
Joint Venture
Lic. No. 647809

4773 Hollywood Blvd. • P.O. Box 27097
Los Angeles, California 90027
(213) 953-7700 • Fax: (213) 953-7707

May 16, 1994

Parsons-Dillingham
4773 Hollywood Blvd.
Los Angels, CA 90027

R81-B251-CRE-2344

Attention: Mr. Salvatore Calvanico, R.E.

RE: Metro Red Line Contract B-251

Subject: Alignment Repair for Concrete Tunnel Liner

Gentlemen:

We respectively request your review and approval of the attached calculations.

The following paragraphs address the comments made in the Response to Submittal B251-02344-1.3-5.00. Paragraph numbers ( ) refer to like-designated paragraphs in Response to Submittal letter (Refer to the copy of the attached letter).

(1) The design presented in this submittal is for horizontal remining only. If other conditions are encountered, an appropriate submittal shall follow.

(2) The calculations have been revised to show the compatibility of the steel rib design and the precast concrete segment design. The intermediate supports have been redesigned as shown in the calculations.

(3) Calculations for rock anchor capacities have been submitted in the previous approved submittal B251-02312-1.5-5.00. A copy is provided for your reference.

(4) The proposed submittal is designed only for sections completely within puente. If other conditions are encountered, an appropriate submittal shall follow.

(5) For a detailed excavation sequence, see construction sequence in calculations.

(6) For post and foot plate design and bearing capacity calculations, see calculations.

Shea Kiewit Kenny
Joint Venture
Lic. No. 647809

4773 Hollywood Blvd. • P.O. Box 27097
Los Angeles, California 90027
(213) 953-7700 • Fax: (213) 953-7707

If you have any questions regarding this matter, please advise.

Very truly yours,

Robert B. Gordon

Robert B. Gordon
Project Manager

RBG:sml:sml

Enclosure

PARSONS–DILLINGHAM

METRO RAIL CONSTRUCTION MANAGER

R81 Metro Red Line Seg-2

SUBMITTAL: B251-02344-1.3-5.00
CDRL #: 696

DATE: 03/22/94 JOB NO: B251

FROM: S. J. CALVANICO, RESIDENT ENGINEER

SEQ: R81-B251-REC-1848

FILE: CA160 B251-02344-1.3-5.00

LETTER OF TRANSMITTAL

RESPONSE TO SUBMITTAL

TO: SHEA-KIEWIT-KENNY J.V.
4773 HOLLYWOOD BLVD.
LOS ANGELES, CA 90027-0097

CC:
___ B. Ghadiali, Proj Unit Manager, B25
___ BOMI GHADIALI
___ J. DEVINE/R. DAMES, CONSTRUCTIC
___ D. MARTINEZ, SYSTEM ENGINEER
___ ~~DCC / SQ020.1~~

ATTENTION: ROBERT B. GORDON,PRJ.MGR.

CONTRACT #/TITLE: B251 / VERMONT/HOLLYWOOD TUNNEL

REFERENCE: B251-CRE-01938 RECEIVED 02/24/94

SUBJECT: SUBMITTAL B251-02344-1.3-5.00 CDRL: 696
ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

I AM SENDING YOU: [XXX] ATTACHED [ ] UNDER SEPERATE COVER

[ ] CONTRACT DRAWINGS [ ] RFI/C RESPONSE [XXX] SUBMITTAL RESPONSE
[ ] SPECIFICATIONS [ ] CHANGE NOTICE [ ] OTHER:
[ ] SKETCHES [ ] CHANGE ORDER

ID NO:	DESCRIPTION		REV. ID	DISPOSITION
B251-02344-1.3-5.00	ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS		5.00	REJ
√	COM PANTEX	DRAWINGS AND CALCS FOR RESUPPORT STL. SETS		REJ

DISPOSITION: REJ - REJECTED; RESUBMIT

RE-SUBMIT √'D ITEMS NO LATER THAN: 04/21/94

Rejected: Revise and Resubmit.

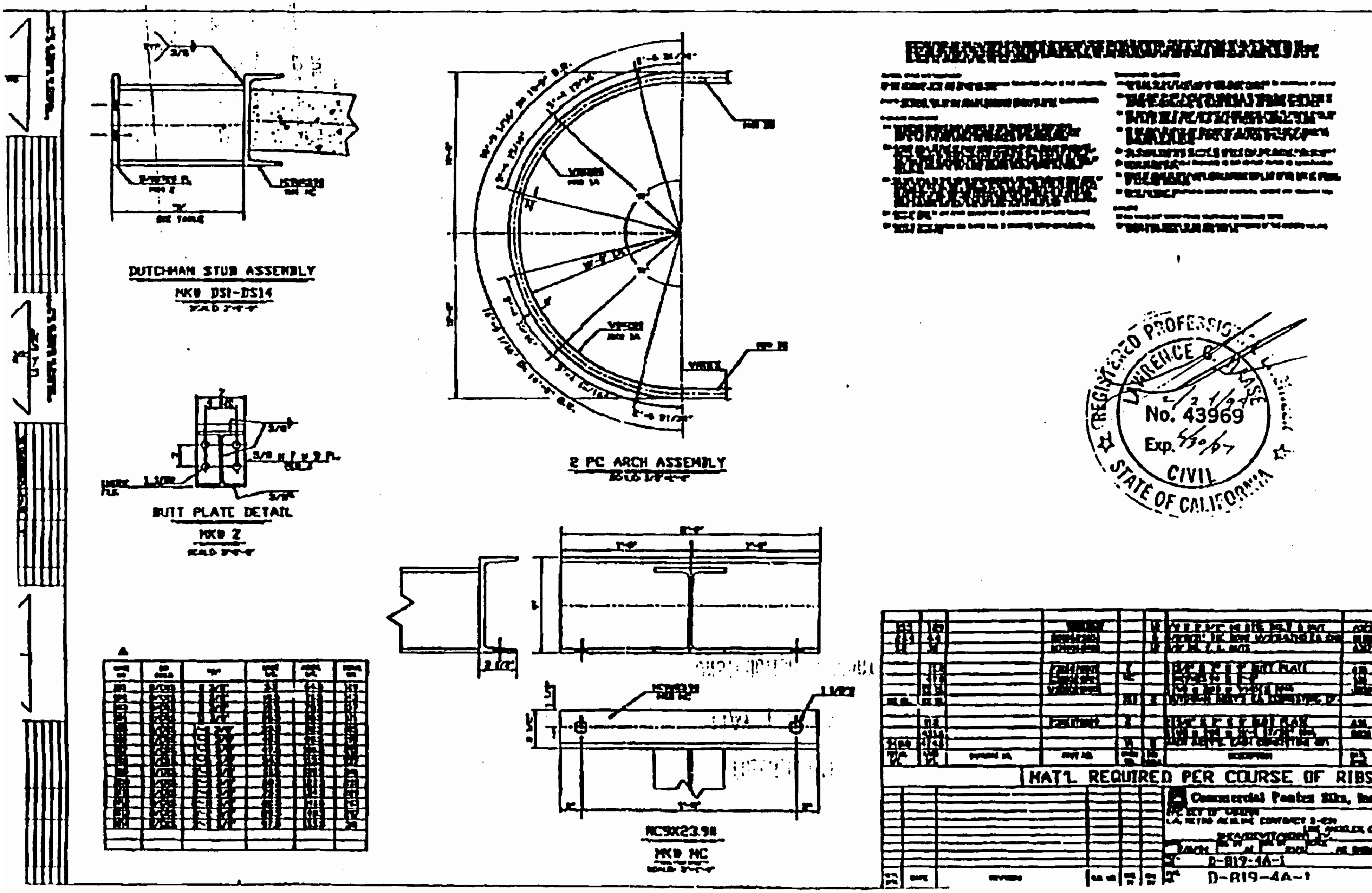
See following remarks from EMC reviewer:
1. The design is appropriate if only horizontal remining and resupport is required. Quick review of the remining cross-sections indicate that remining will require sections with vertical offsets as well, for which such a scheme is not appropriate. Provide details for the full range of conditions encountered.
2. The precast concrete segment design load differs greatly from that used by the structural steel designer (Commercial). The two designs must be compatible. It appears the structural steel ribs would be adequate, provided bearing plates and details are sized accordingly, but the wall plate and intermediate support details will require modification.
3. No calculations are provided for the rock anchors. Please provide calculations and details.
4. These details are appropriate for sections wholly within Puente.
5. Mining of such a section requires careful control of excavation and control of ground. Provide detailed excavation sequence, intermediate support details and face stability methods.
6. Provide post and foot plate design and bearing capacity calculations.

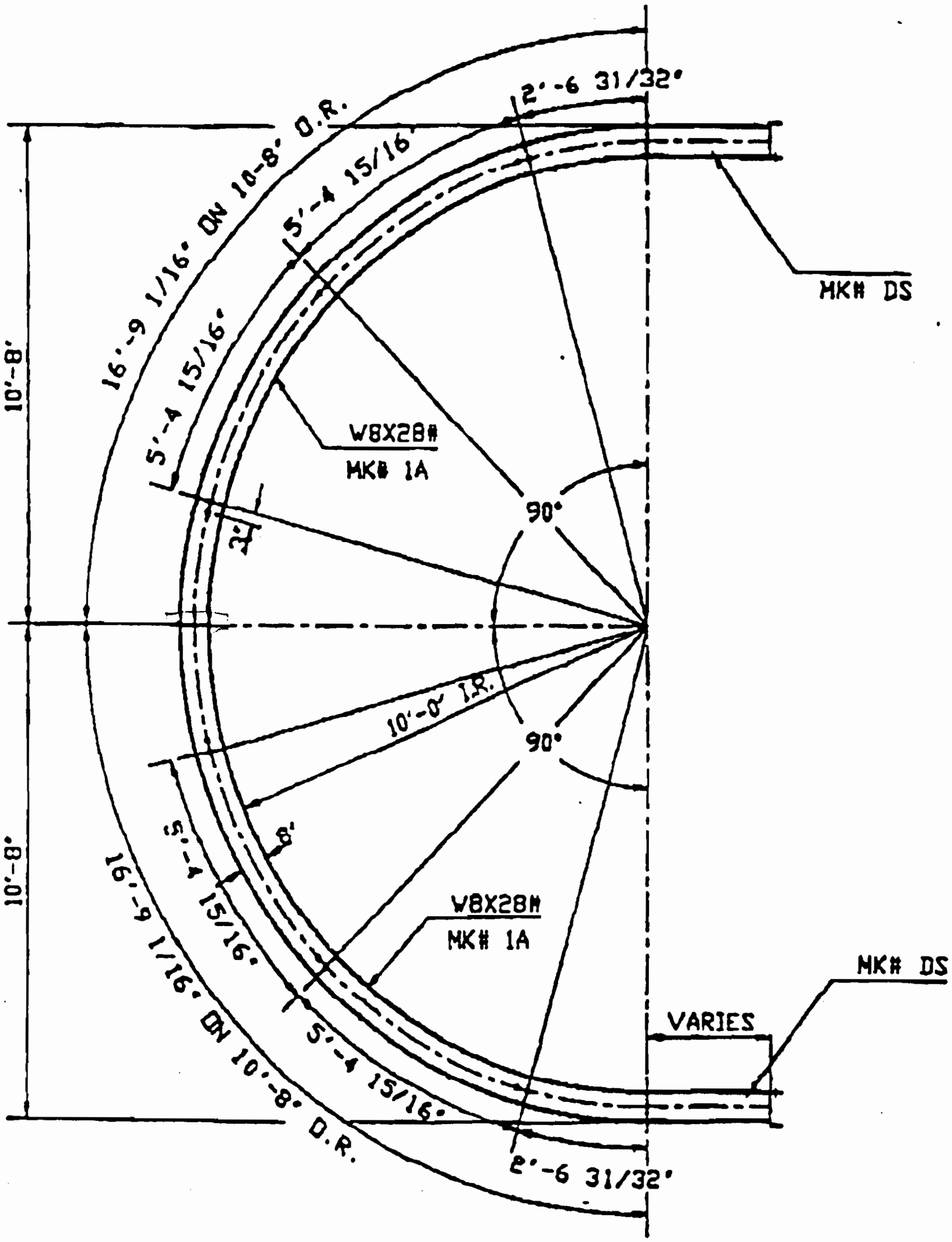
This submittal lacks primary calculations and necessary support details which are critical to the safe execution of this remining operation. Please provide these details and calculations for review and approval.

SIGNED: [signature]
S. J. CALVANICO, RESIDENT ENGINEER

J. HARRINGTON

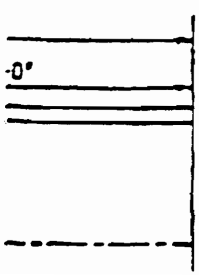
R251-02344-13-5.01

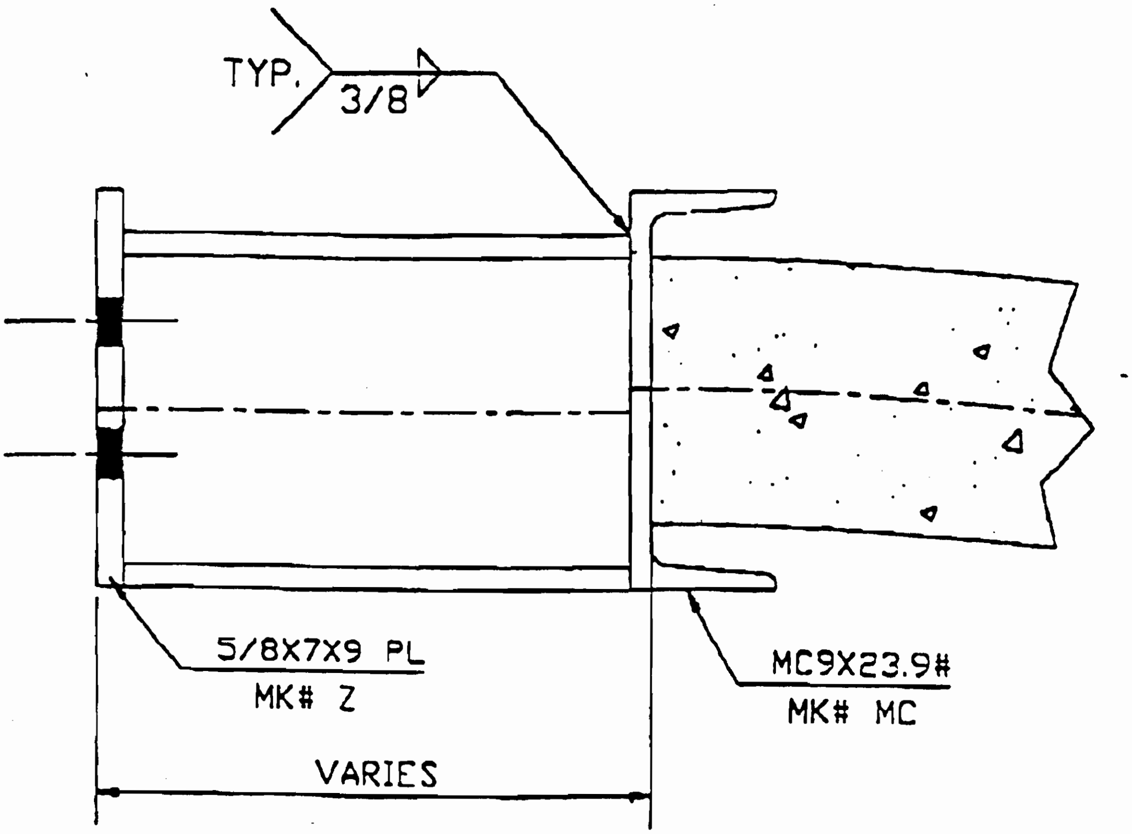




2 PC ARCH ASSEMBLY

SCALE: 3/8"=1'-0"





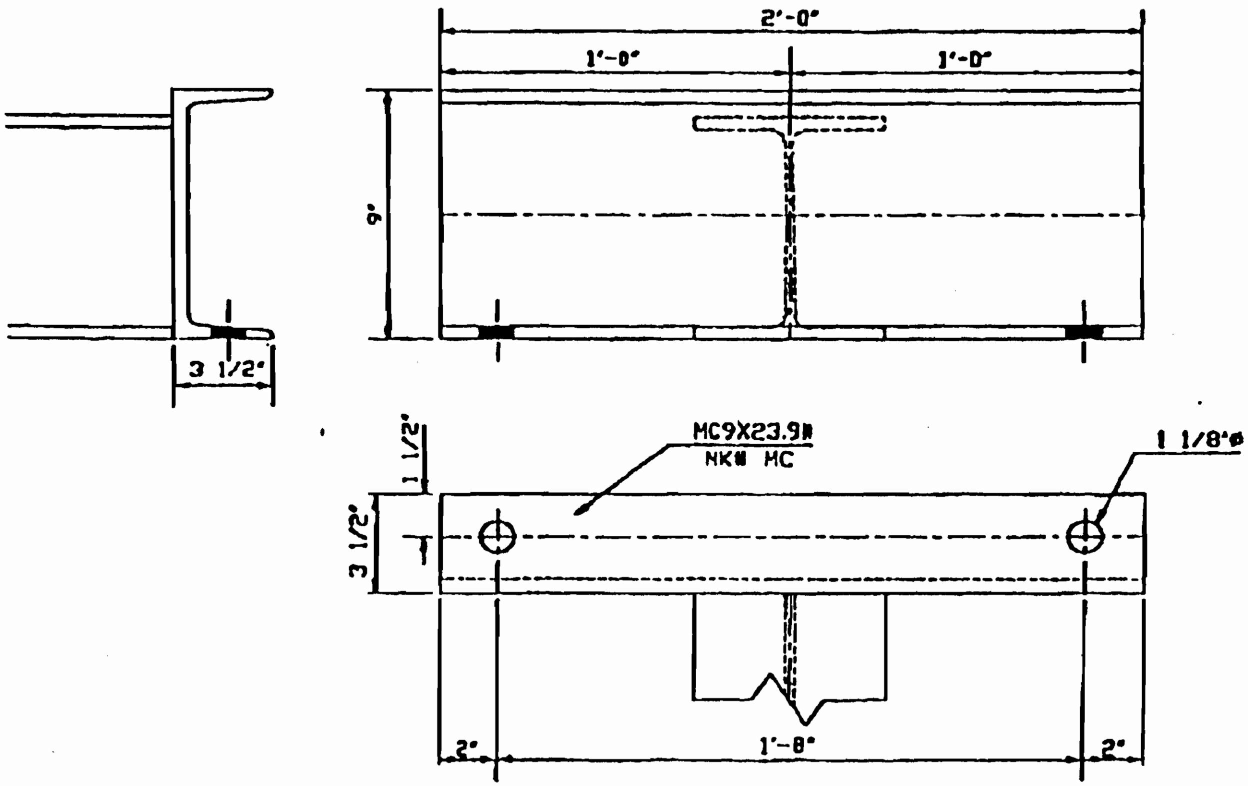
DUTCHMAN STUB ASSEMBLY

MK# DS

SCALE: 3"=1'-0"

2 PC ARCH ASSEMB

SCALE: 3/8"=1'-0"



MC9X23.9#

MK# MC

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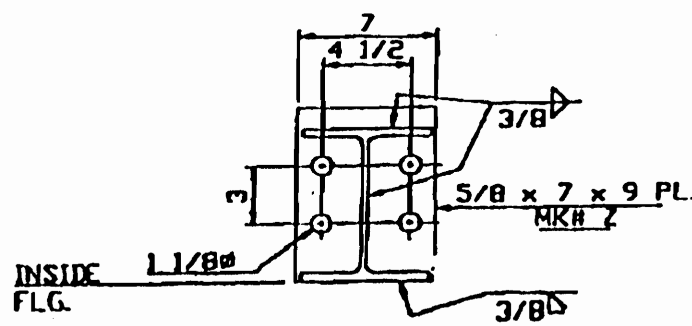
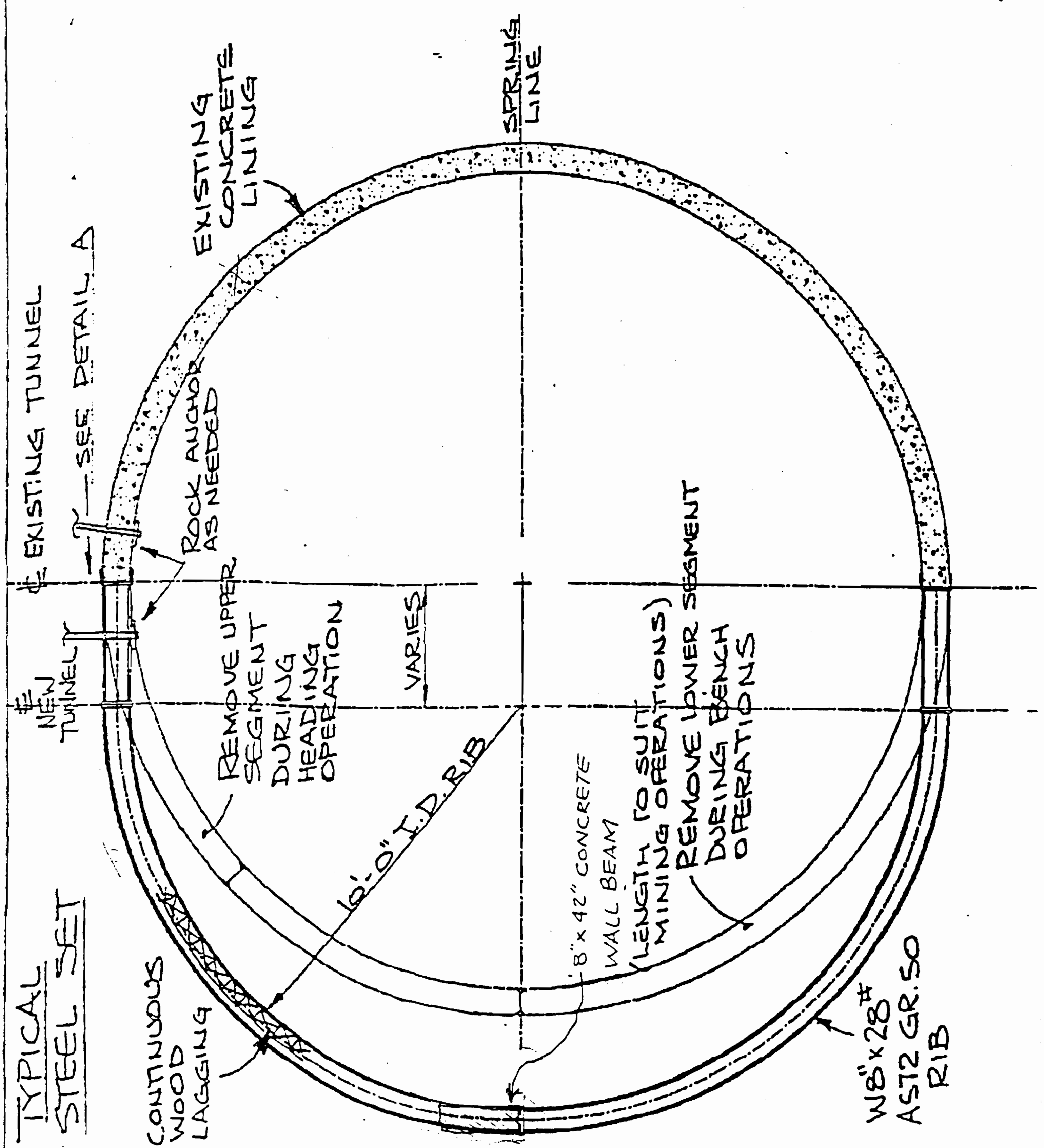


BUTT PLATE DETAIL

MK# Z

SCALE: 3"=1'-0"

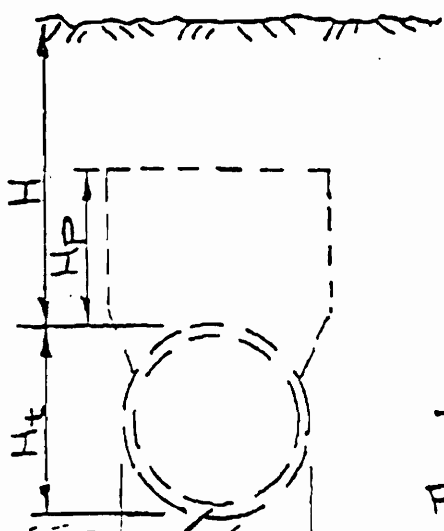
Commercial Pantex Sika, Inc.	L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA SHEA-KIEWIT-KENNY JV
DATE 2-1-94 BY R. SMITH SHEET OF	



Commercial Pantex Sika, Inc.	L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA. SHEA-KIEWIT-KENNY, J.V.	DATE 2-1-94 R. SMITH PE SHEET 1 OF 6.

ALIGNMENT REPAIR FOR CONCRETE TUNNEL SEGMENTS

ANTICIPATED LOAD (ON STEEL RIBS)



B_t = tunnel diameter ≈ 21.5 ft

H_t = tunnel height ≈ 21.5 ft

H = total height of overburden = varies

H_p = load of rock or earth (ft) on roof of support

REGISTERED PROFESSIONAL ENGINEER
LAWRENCE G. HAASE
2/24/94
No. 43969
Exp. 3/30/97
CIVIL
STATE OF CALIFORNIA

Tunnels in Puente

From "Rock Tunneling with Steel Supports", p. 91, Table 3, assume rock condition as moderately blocky and seamy. Moderately blocky rock contains joints & hair cracks, but the blocks between joints are locally grown together or itimately interlocked so vertical walls do not require lateral support. Spalling & popping of rock may occur.

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Vertical load $P_v = w\,H_p = (130\text{ pcf})(8.6\text{ ft}) = 1118$ psf

Commercial Pantex Sika, Inc.			L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA SHEA-KIEWIT-KENNY, J.V.
DATE 5-12-94	BY R.SMITH PE	SHEET 2 OF 6	

CALCULATIONS FOR TUNNEL REMINE AREAS

I. STEEL SETS MUST CARRY SAME LOAD AS CONCRETE SEGMENTS.

- ALLOWABLE SEGMENT LOAD = (60 FT OVERBURDEN)(120 PCF)
  = 7200 PSF

FROM SHEET 3;

ALLOWABLE STEEL SET LOAD = 7255 PSF > 7200 PSF
O.K

II. CALCULATE PRESSURE ON FOOT PLATE (SEE POINT 1, SHEET 5)

AXIAL BEAM THRUST T = (VERTICAL LOAD)(R.B RAD)(RIB SPAC)
= (7200 PSF)(10.67 FT)(4.0 FT)
= 307296 LB

BEARING PRESSURE = T/A A = AREA OF BUTT PL.

$$= \frac{307296 \text{ LB}}{7" \times 9"} = 4878 \text{ PSI}$$

400 PSI
63
76 K in²
17 x 27

THEREFORE SOIL OR CONCRETE MUST HAVE ALLOWABLE BEARING OF APPROX. 5000 PSI.

Commercial Pantex Sika, Inc.

L.A. METRO RED LINE
B-251
LOS ANGELES, CA
SHEA-KIEWIT-KENNY, J.V.

DATE 2-22-94 | BY R.SMITH PE | SHEET 3 OF 6

ALLOWABLE LOAD CALCULATIONS

W8 X 28 lb/ft (Grade 50 steel)

$A = 8.25 \text{ in}^2$
$S = 24.3 \text{ in}^3$
$I = 98.0 \text{ in}^4$
$r = 3.45$ in
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s = blocking point spacing = 0 in
(continuous contact with surrounding ground)
b = rise in chord between blocking pts = 0 in
f_b = allowable bending stress = $.75\ F_Y = 37500$ psi
(due to cold-working of steel)
E = modulus of elasticity of steel = 29×10^6 psi
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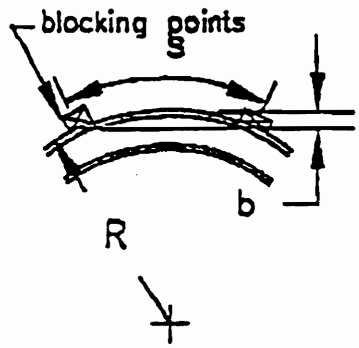
d = rib spacing, c/c = varies



$$f_b = \frac{T_A}{A} + \frac{M}{S} \qquad M = 0.86\ b\ T_A$$

Allowable ring thrust $T_A = \dfrac{f_b \times A \times S}{S + .86 \times b \times A} = \dfrac{37500\ (8.25)\ 24.3}{24.3 + .86\ (0)\ 8.25}$

$= 309375$ lb / rib

Allowable thrust due to critical buckling :

$$T_{cb} = \frac{3 \times E \times I}{R^2 \times S.F.} = \frac{3\ (29 \times 10^6)\ 98.0}{(127.9)^2\ \ 1.25} = 416960 \text{ lb/rib}$$

Use $T = T_A$ or T_{cb} (the smaller value)

$$\text{Allowable load} = \frac{T}{d \times R} = \frac{309375}{(4.0)(10.66)} = 7255 \text{ PSF } (4'\text{-}0" \text{ c/c})$$

References:
Rock Tunneling with Steel Supports, Proctor & White, pp 191–232

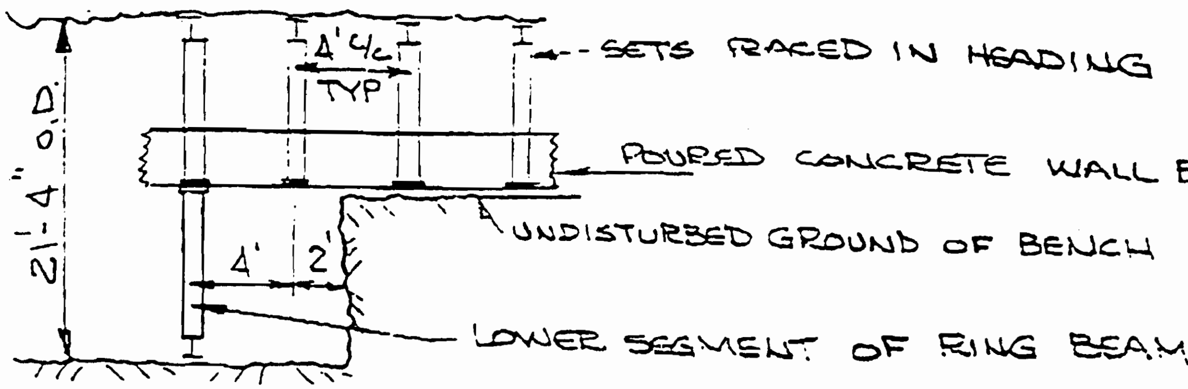
Theory of Elastic Stability, Timoshenko, pp 297–300

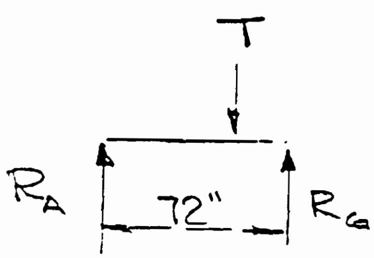
Commercial Pantex Sika, Inc.			L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA SHEA-KIEWIT-KENNY, J.V.
DATE 5-12-94	BY R.SMITH PE	SHEET 4 OF 6	

III WALL BEAM CALCULATIONS

ASSUME POURED CONCRETE BEAM
(SEE POINT 2, SHEET 3)

WORST CONDITION:





T = RING THRUST = 307296 LB.

R_A = REACTION LOAD TO LOWER ARCH SEGM.

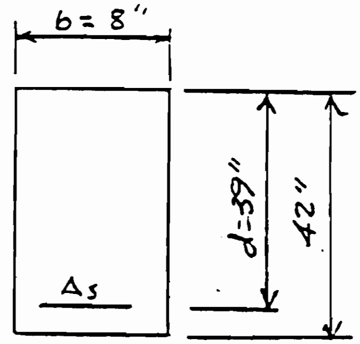
R_G = REACTION LOAD TO GROUND

$$\text{MAX. MOMENT FOR SIMPLE BEAM} = \frac{T(48'')(24'')}{72''}$$

$$= 4917 \text{ in-kips}$$

THEREFORE WALL BEAM MUST RESIST MOMENT OF 4917 IN-KIPS.

CONCRETE WALL BM:



$M_{max} = 4917$ IN-KIPS
$f_c' = 5000$ PSI
$f_y = 60000$ PSI
$b = 8"$
$d = 39"$

$F = bd^2/12000 = 8(39)^2/12000 = 1.014$
$M = 4917/12 = 410^{\text{K-FT}}$

$K = M/F = 410/1.014 = 404$

$\Rightarrow \rho = 0.0095$ ACI '91 FLEXURE 2.3 SP-17

$A_{s\,REQ'D} = \rho b d = 0.0095(8)(39) = 2.96^{\square''}$

USE 3-#9 (A_s PROV'D $= 3.00^{\square''} > A_s$ REQ'D $= 2.96^{\square''}$)

$V_u = 307296^{\#}(48)/72 = 204864^{\#}$

$V_c = 2bd\sqrt{f_c'} = 2(8)(39)\sqrt{5000} = 44124^{\#}$

$V_s = V_u - V_c = 204864 - 44124 = 160740^{\#}$

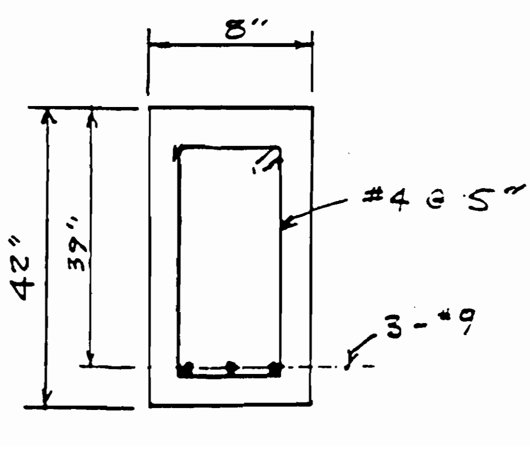
MAX $V_s = 8(\sqrt{f_c'})bd = 4V_c = 176496^{\#} > 160740^{\#}$ O.K.

MIN $V_s = 50(b)(d) = 50(8)(39) = 15600^{\#}$

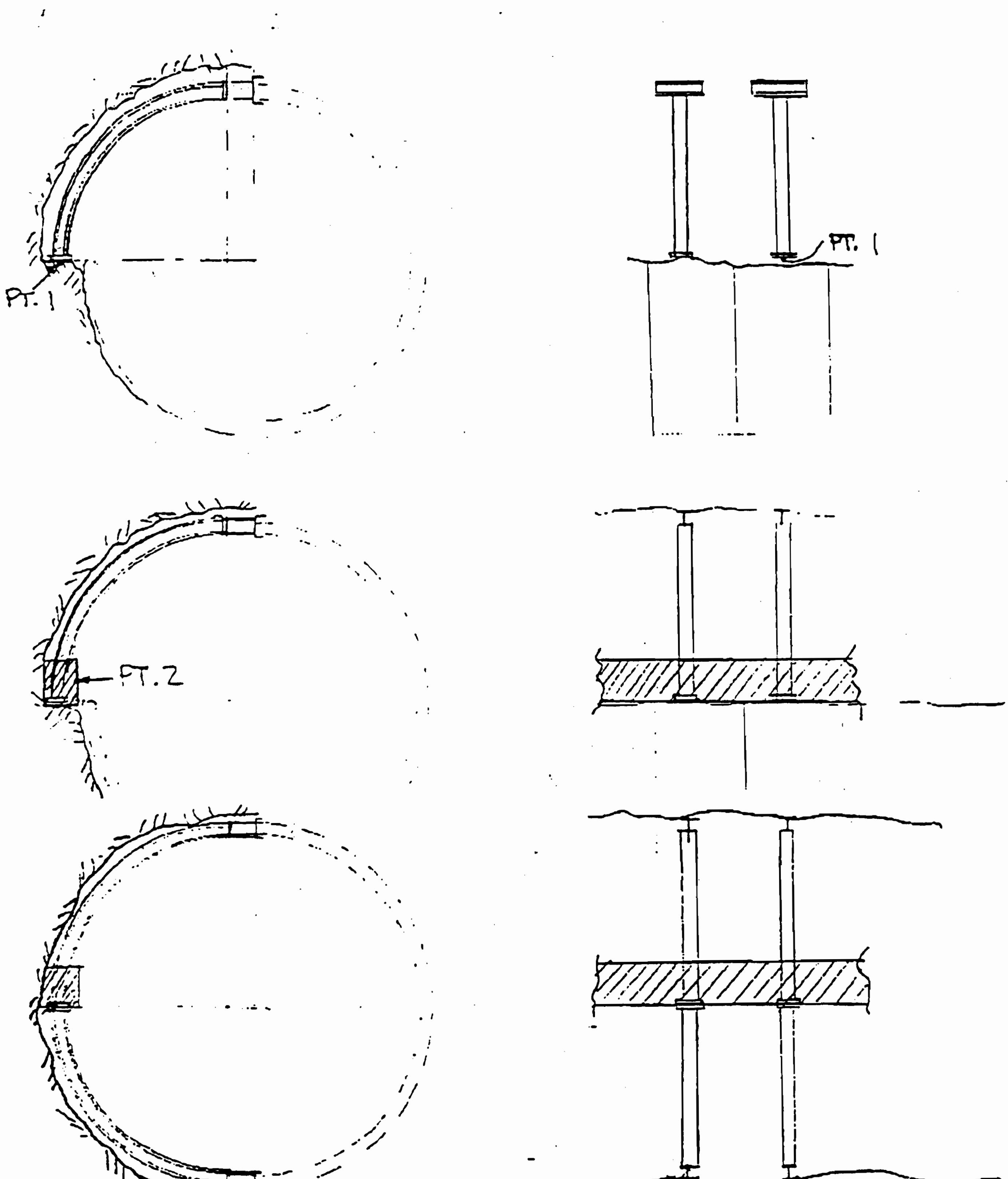
$\therefore$ USE $V_s = 160740^{\#} = A_v f_y \frac{d}{s} = 0.4(60000)(39/5.0) = 187500^{\#}$ OK

USE #4 STIRRUPS @ 5" O.C.

REGISTERED PROFESSIONAL ENGINEER
STUART M. LIPOFSKY
NO. C051652
EXP. 6-30-94
CIVIL
STATE OF CALIFORNIA



Commercial Pantex Sika, Inc.			L.A. METRO RED LINE CONTRACT B-251 LOS ANGELES, CA
DATE 2-2-94	BY R.SMITH	SHEET 5 OF 6	SHEA-KIEWIT-KENNY,J.V.



Commercial Pantex Sika, Inc.

DATE 2-1-94 | BY R.SMITH PE | SHEET 6 OF 6

L.A. METRO RED LINE
CONTRACT B-251
LOS ANGELES, CA
SHEA-KIEWIT-KENNY JV

ALLOWABLE LOAD CALCULATIONS – WOOD LAGGING

As the lagging begins to deflect under vertical pressure, the soil will arch to a height h_A depending upon the angle of internal friction of the soil. An assumed arch shape is shown below. The soil will develop shear stresses along the dotted lines, with the arch load being carried by the lagging and transferred to the ribs. The soil load between arches will be carried directly by the ribs.

Assume lagging is 3" X 8"   Allowable bending stress $f_b = 1400$ psi

Section modulus $S = \dfrac{b\ d^2}{6} = 12\ in^3.$

Allowable moment $M = f_b \times S$
$= (1400)(12.0)$
$= 16800$ in-lb

Moment for simple beam w/ triangular load:

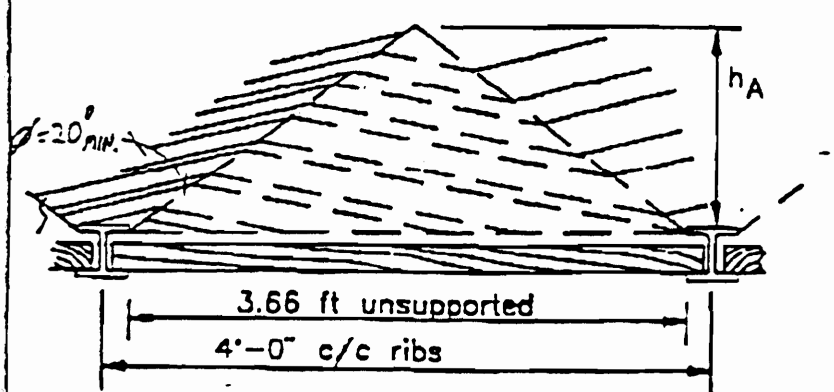
$M = \dfrac{W \times l}{6}$ $W = P_v \times l \times 0.67$ ft

$$M = \frac{P_v \cdot l^2 \cdot \frac{2}{3}}{6} = 16800 \text{ in-lb} = 1400 \text{ ft-lb}$$

$$P_v = \frac{6(1400)}{(.67)(3.66)^2} = 940 \text{ psf}$$

Amount of allowable overburden = 940 psf / 130 pcf = 7.2 ft of soil or rock

$h_A = \frac{3.66}{2} \tan 70° = 5.03' < 7.2'$ OK



Construction Sequence

1. Install 2 rock anchors (i.e. 2 ft. o.c. at 45 degrees) in each arch segment on the tunnel side that does not get removed.

2. Sawcut and remove upper quarter of tunnel precast concrete ring (i.e. 4 ft.). Only one upper quarter segment is to be removed at any time without upper quarter steel sets installed.

3. Install steel dutchman, upper quarter steel set and lagging. Install one rock anchor in the steel dutchman.

4. Repeat steps 2 and 3 until entire upper bench is completed.

5. Pour wall beam with 5000 psi concrete.

6. Sawcut and remove lower quarter of tunnel precast concrete ring (i.e. 4 ft.). Only one lower quarter segment is to be removed at any time without lower quarter steel sets installed.

7. Install steel lower quarter post and lagging.

8. Repeat steps 6 and 7 until entire lower bench is completed.

APPENDIX B

Tunnel Heading Reports

CONTRACT
B251

PARSONS - LLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 6/28/93 TIME: 1:35

HEADING: HOLLYWOOD - AL

STATION: 464+34

RING NO: 75

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: JB

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

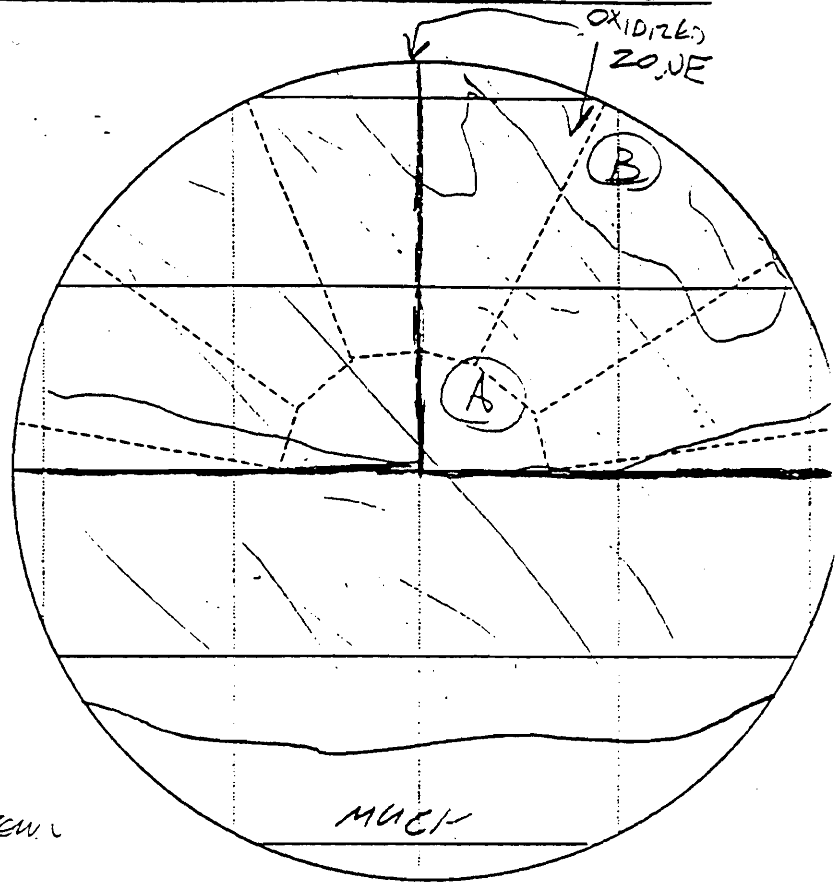
ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM:

GAS TESTING:

COMMENTS: PUENTE Fm, OLIVE GREEN OXIDIZED TO REDDISH BROWN MOIST, MODERATELY SHEARED APPARENT DIP ~65° TRUE DIP NORTH WEST SHEARING IS DOMINANTLY PARALLEL TO FACE WITH WELL DEVELOPED SLICKENSIDES, OXIDIZED ZONES NEAR CROWN DIP ALONG BEDDING; EASY MINING PUENTE IS FRIABLE, NO HARD ROCK



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	[illegible]		[illegible]			[illegible]	NONE	MOIST
[illegible]	[illegible]		[illegible]			[illegible]	[illegible]	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

JM

CONTRACT
B251

PARSONS - . LLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 6/28/93 TIME: 2:55

HEADING: HOLLYWOOD - AL

STATION: 464+42

RING NO: 77

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: JB

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

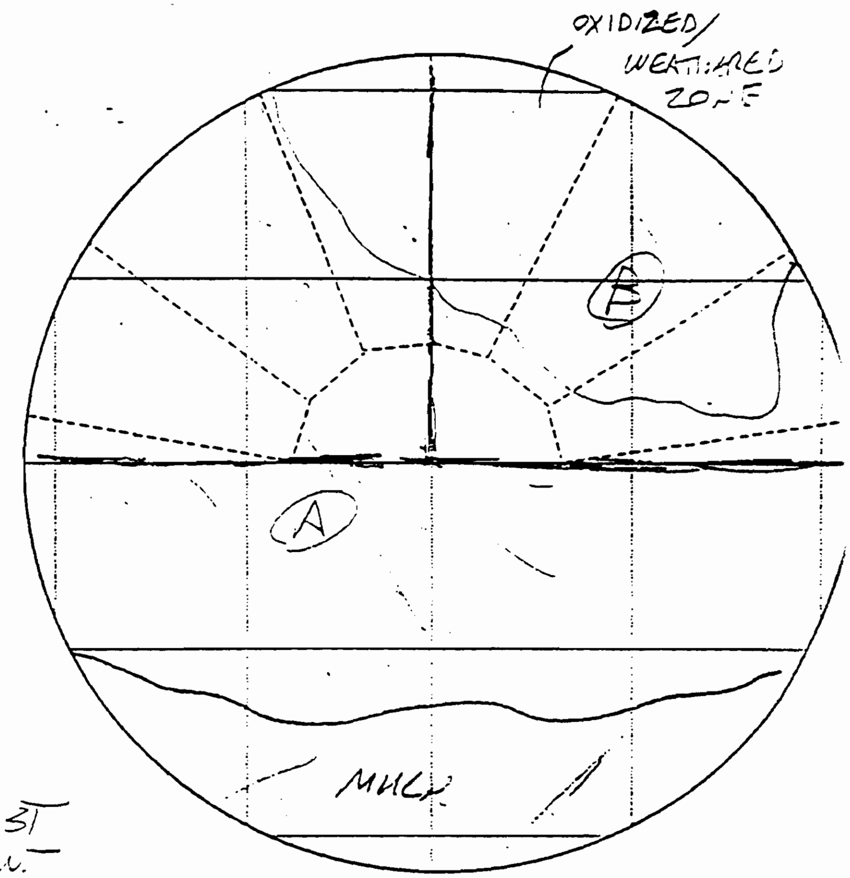
ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM:

GAS TESTING:

COMMENTS: PUENTE Fm, OLIVE GRAY - YELLOW BROWN, MOIST MODERATELY SHEARED. APPARENT DIP ~65° OXIDIZED/WEATHERED ZONE EXTENDS DOWN ALMOST TO SPRINGLINE ALONG BEDDING PLANES 3 JOINTS, NO HARD ROCK EASILY MINED (FRIABLE)



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

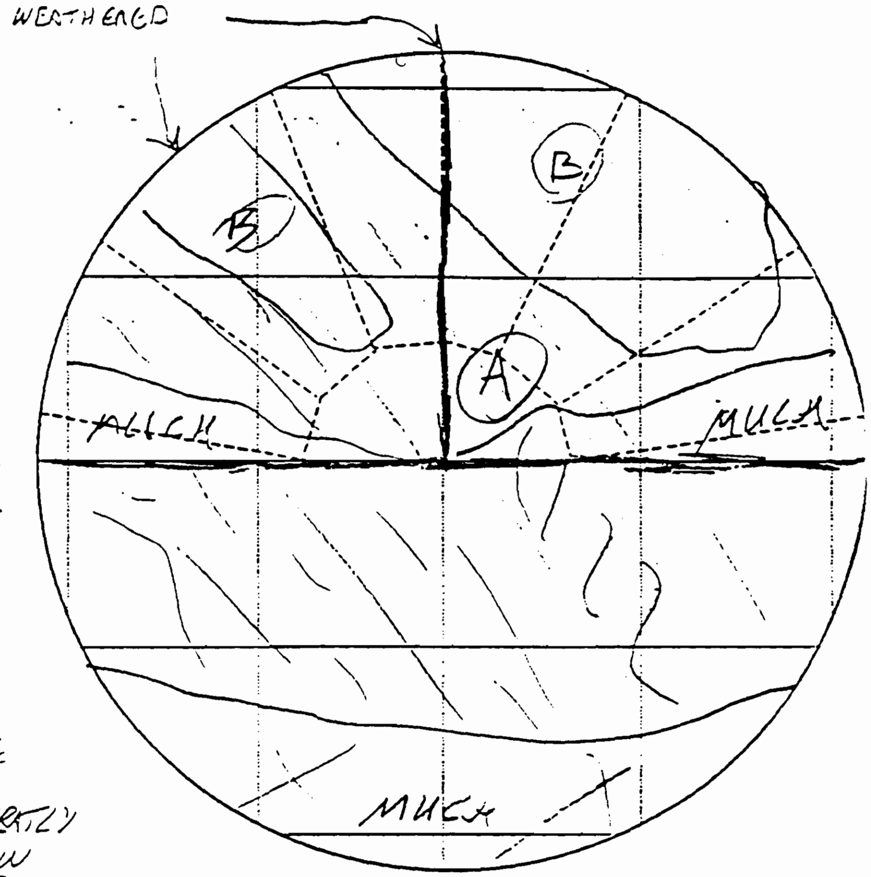
NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE Fm		OLIVE GRAY			WEAK	NONE	MOIST
B	PUENTE Fm		YEL BROWN			VERY WEAK	[illegible]	V. MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

CONTRACT
B251

PARSONS - [illegible]LLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT



SCALE 1" = 5'

DATE: 6/28/93 TIME: 5:10
HEADING: HOLLYWOOD - AL
STATION: 464+50
RING NO: 79
PHOTOGRAPH NO: ____________
SOIL / ROCK SAMPLE NO: ____________
INSPECTOR: JB
HARD ROCK IN - RING #: ____________
HARD ROCK OUT - RING #: ____________
GPR SURVEY RING #: ____________
ROCK SPLITTER USED: ____________
SLOWER PROGRESS: ____________
GROUNDWATER INFLOW GPM: 0
GAS TESTING: ____________
COMMENTS: PUENTE Fm OLIVE GRAY - ORANGE BROWN, MODERATELY SHEARED BEDDING DIPS NW @ ~45° (VARIABLE) WEATHERED ZONES ALONG BEDDING AND SHEARS DOWN FROM CROWN TO SPRING LINE, OXIDIZED OVER MOST OF THE FACE, NO HARD ROCK, EASILY MINED BY TSM (FRIABLE)

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE		[illegible]			WEAK	OXIDIZED-[illegible]	MOIST
B	PUENTE		ORANGE BROWN			VERY WEAK	WEATHERED	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

METRO

CONTRACT
B251

PARSONS - ...LLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 5/29/93 TIME: 6:35

HEADING: HOLLYWOOD - AL

STATION: 464+58

RING NO: 81

PHOTOGRAPH NO: ____

SOIL / ROCK SAMPLE NO: ____

INSPECTOR: JB

HARD ROCK IN - RING #: ____

HARD ROCK OUT - RING #: ____

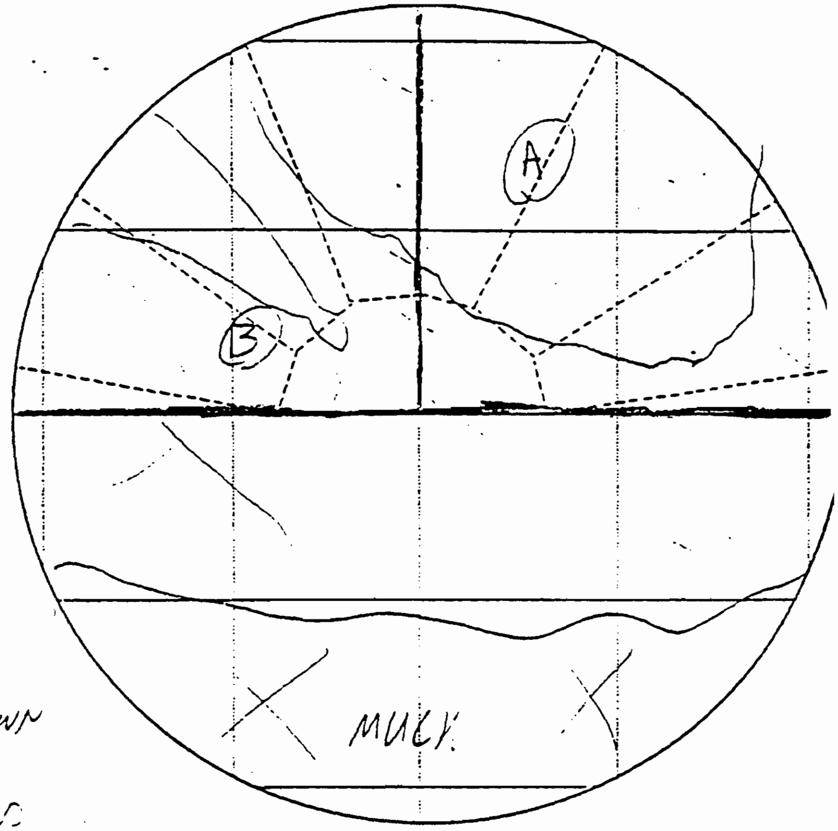
GPR SURVEY RING #: ____

ROCK SPLITTER USED: ____

SLOWER PROGRESS: ____

GROUNDWATER INFLOW GPM: 0

GAS TESTING: ____



SCALE 1" = 5'

COMMENTS: PUENTE Fm LIGHT BROWN GRAY - ORANGE BROWN MOIST, MODERATELY SHEARED, WEATHERED AT CROWN. OXIDIZED TO BELOW SPRING LINE. SHEARING SHOWS WELL DEVELOPED SLICKENSIDES. NO HARD ROCK. EASILY MINED BY TBM

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE Fm		LIGHT BROWN - GRAY			WEAK	OXIDIZED	MOIST
B	PUENTE Fm		ORANGE BROWN			VERY WEAK	WEATHERED	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

CONTRACT

B251

PARSONS - ..LLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 6/29/93 TIME: 0:15

HEADING: HOLLYWOOD - AL

STATION: 464+86

RING NO: 88

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: JB

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

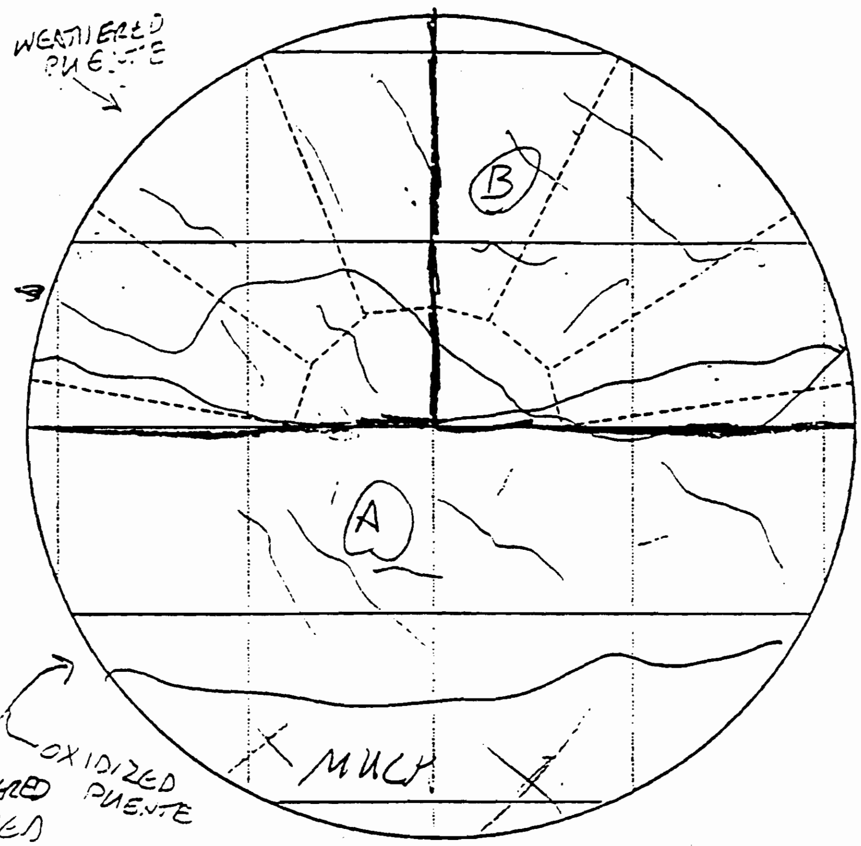
ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM: 0

GAS TESTING:

COMMENTS: PUENTE Fm ORANGE BROWN - GRAY GREEN, WEATHERED ABOVE SPRING LINE OXIDIZED BELOW MODERATELY SHEARED WITH WELL DEVELOPED SLICKENSIDES BEDDING IS NOT DISTINCT, NO HARD ROCK, MATERIAL IS FRIABLE AND EASILY MINED BY TBM



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE Fm		GRAY			WEAK	OXIDIZED	MOIST
B	PUENTE Fm		BROWN-ORANGE			VERY WEAK	WEATHERED	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

OM

CONTRACT
B251

PARSONS - [illegible]LLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 6/29/93 TIME: 3:10

HEADING: HOLLYWOOD - AL

STATION: 464+92

RING NO: 89

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: JB

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

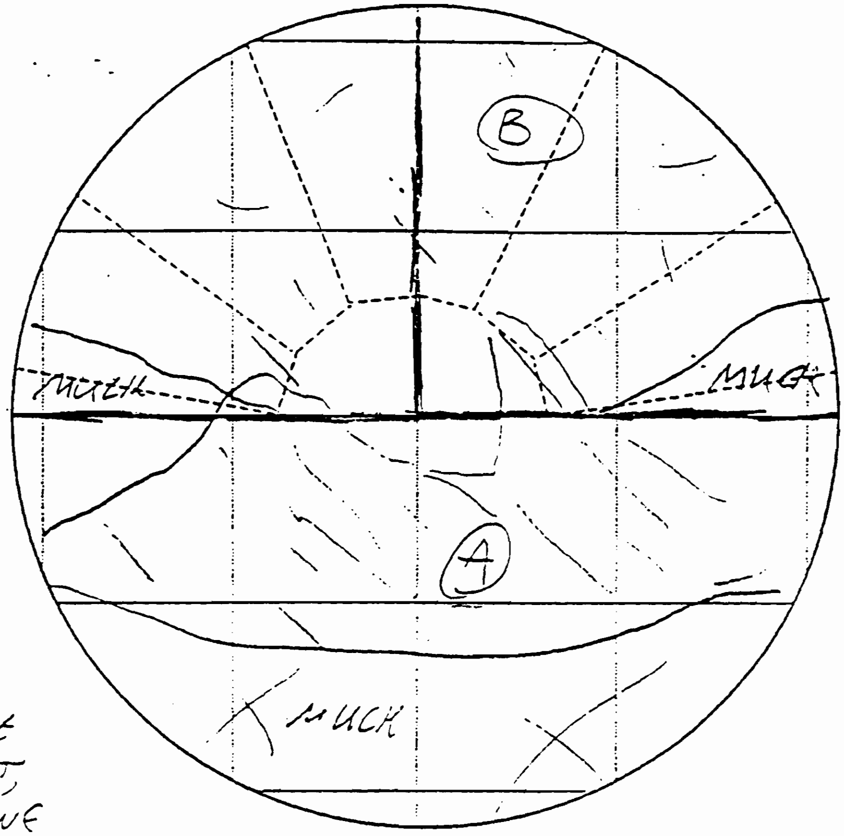
ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM: 0

GAS TESTING:

COMMENTS: PUENTE Fm. ORANGE BROWN - GRAY GREEN, MOIST, WEATHERED ABOVE SPRING LINE OXIDIZED ALONG JOINTS & BEDDING BELOW, MODERATELY SHEARED WITH [illegible] WELL DEVELOPED SLICKENSIDES, BEDDING DIPS NORTH-NORTHWEST ~65° NE HARD ROCK, FRIABLE, EASILY [illegible] BY [illegible]



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE F.		GRAY GREEN			WEAK	OXIDIZED	
B	PUENTE F.		ORANGE BROWN			VERY WEAK	WEATHERED	

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

m

METRO

CONTRACT B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

(1)

DATE: 6/29/93 TIME: 3:55

HEADING: HOLLYWOOD - AL

STATION: 464+79 ✓

RING NO: 91

PHOTOGRAPH NO: ______

SOIL / ROCK SAMPLE NO: ______

INSPECTOR: JB

HARD ROCK IN - RING #: ______

HARD ROCK OUT - RING #: ______

GPR SURVEY RING #: ______

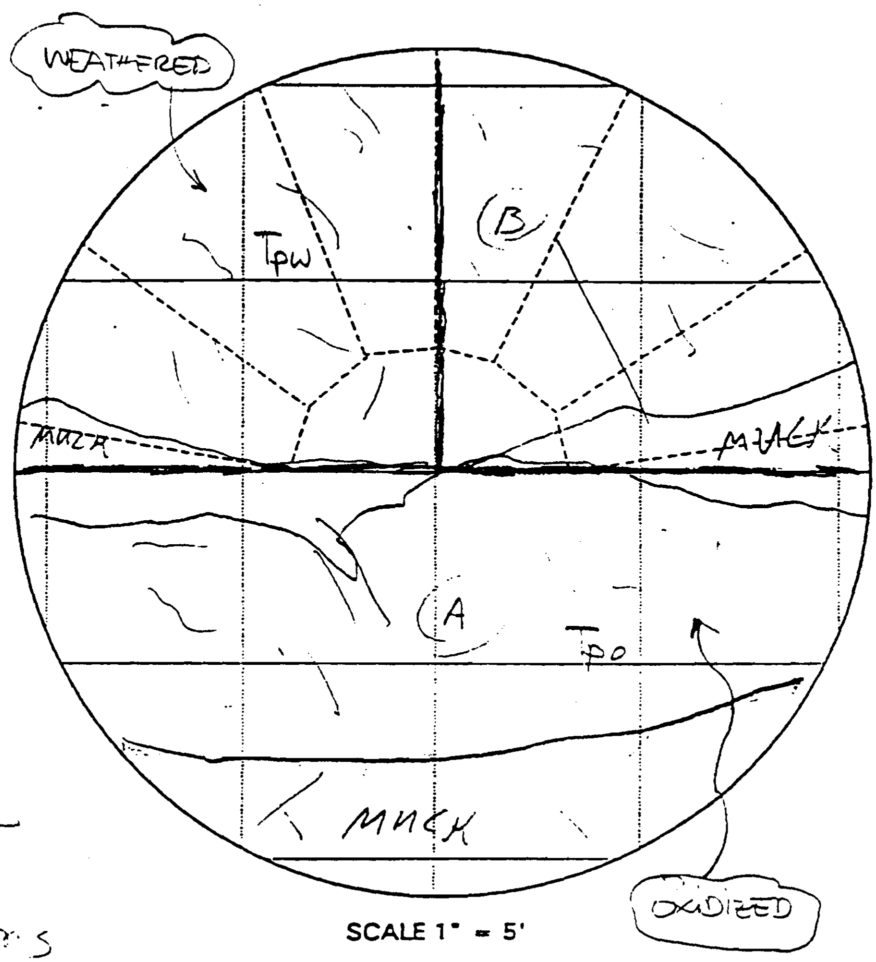
ROCK SPLITTER USED: ______

SLOWER PROGRESS: ______

GROUNDWATER INFLOW GPM: 0

GAS TESTING: ______

COMMENTS: PUENTE Fm ORANGE BROWN - GRAY-GREEN, MOIST MODERATELY SHEARED WITH WELL DEVELOPED SLICKENSIDES BEDDING IS ABSENT NEAR CROWN 3' POORLY DEFINED [illegible], WEATHERED ZONE B EXTENDS FROM CROWN TO JUST BELOW SPRING LINE PARTIALLY OXIDIZED ZONE A IS DARK GRAY AND OXIDIZED ALONG DOMINANT BEDDING PLANES. NO HARD ROCK EASILY MINED BY TBM



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE Fm		OLIVE GRAY			WEAK	OXIDIZED	MOIST
B	PUENTE Fm		ORANGE BROWN			VERY WEAK	WEATHERED	MOIST

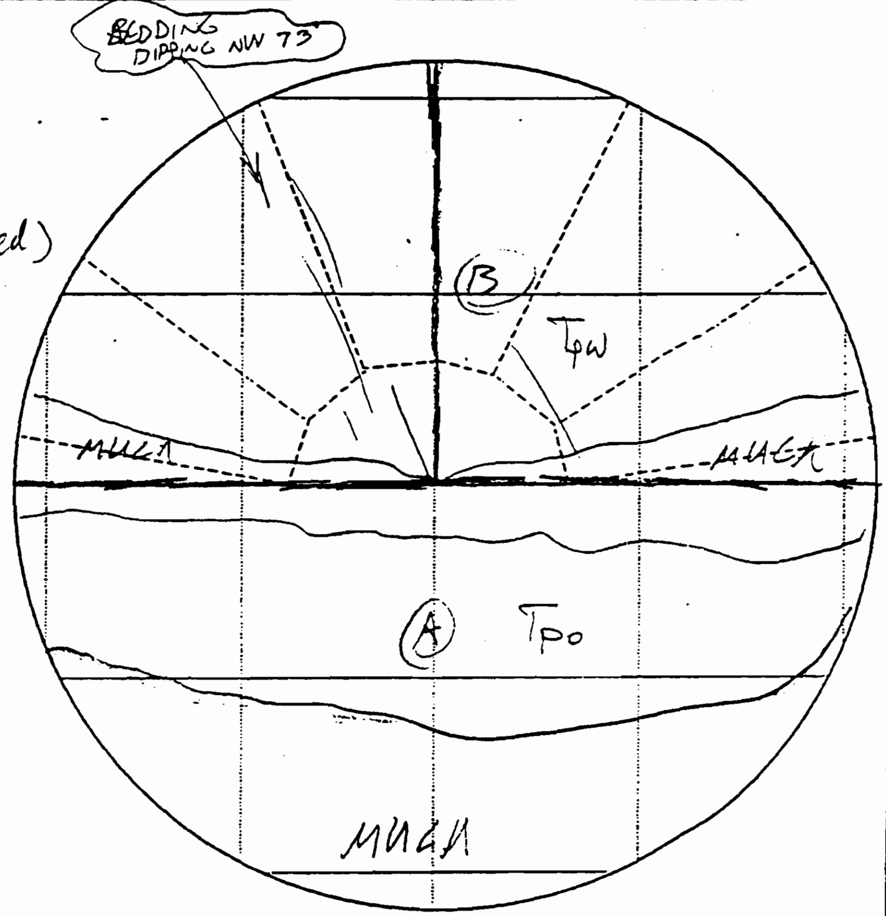
FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

METRO

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT



SCALE 1" = 5'

DATE: 6/29/93 TIME: 5:10

HEADING: HOLLYWOOD - AL

STATION: 464+96 (+9 Ft Added)

RING NO: 93

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: JB

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM: 0

GAS TESTING:

COMMENTS: PUENTE Fm
GRAY GREEN—ORANGE BROWN
MOIST, MODERATELY SHEARED
BEDDING DIPS NW@73°
STRIKE ~ 30° SOUTH OF HEADING
SHEARING SHOWS WELL DEVELOPED SLICKENSIDES, NO HARD ROCK
EASILY MINED BY TSM, WEATHERED ZONE EXTENDS BELOW
SPRING LINE

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE Fm		GRAY GREEN			WEAK	OXIDIZED	MOIST
B	PUENTE Fm		BROWN ORANGE			VERY WEAK	WEATHERED	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 6/30/93 TIME: 9:30

HEADING: HOLLYWOOD - AL

STATION: 405 + 08 ✓

RING NO: 96

PHOTOGRAPH NO: —

SOIL / ROCK SAMPLE NO: —

INSPECTOR: ND

HARD ROCK IN - RING #: —

HARD ROCK OUT - RING #: —

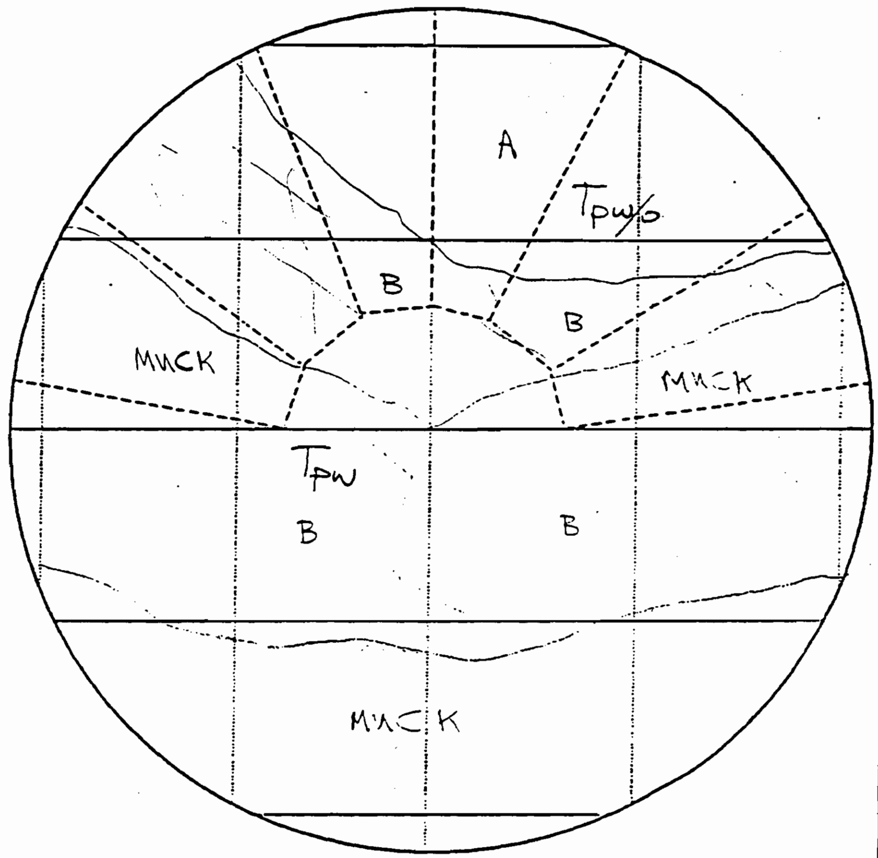
GPR SURVEY RING #: —

ROCK SPLITTER USED: —

SLOWER PROGRESS: —

GROUNDWATER INFLOW GPM: —

GAS TESTING: —



SCALE 1" = 5'

COMMENTS:

A. Brownish yellow oxidized Puente present @ the crown level.

B. Puente siltstone / claystone, highly weathered with no well defined bedding.

Face is easy to excavate. Shield operator is generally pushing out 4' to 5' with the "cutting" rams and then using the plate on the digger to draw out the muck onto the conveyor belt. Generally he is not using the "digger" to excavate. Material easily crumbles down from the face.

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	Puente	—	Brownish yellow	—	—	Weak	Oxidized	Moist
B	"	—	Dark olive gray	—	—	Weak	Fresh	"

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED	X	BLOCKY	X
SQUEEZING		SWELLING		SHEARED	X	WATER	

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

SWING

DATE: 6/30/93 TIME: 1935

HEADING: HOLLYWOOD - AL

STATION: 465+44 ✓

RING NO: 105

PHOTOGRAPH NO: —

SOIL / ROCK SAMPLE NO: —

INSPECTOR: ST

HARD ROCK IN - RING #: —

HARD ROCK OUT - RING #: —

GPR SURVEY RING #: —

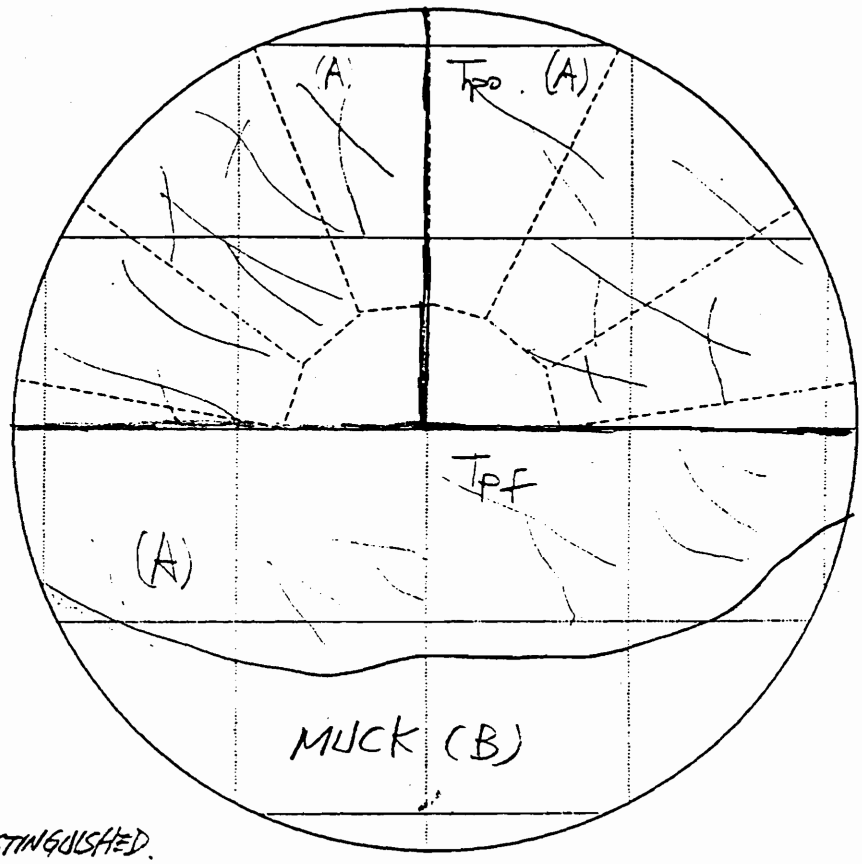
ROCK SPLITTER USED: —

SLOWER PROGRESS: —

GROUNDWATER INFLOW GPM: —

GAS TESTING: —

COMMENTS:



SCALE 1" = 5'

(A)- HIGHLY SHEARED AND JOINTED PUENTE FM. BEDDING IS UNDISTINGUISHED. OXIDIZED PUENTE IS PRESENT AT THE CROWN AREA. EASY TO EXCAVATE. NO WATER SEEPING OUT OF FACE.

(B)- MUCK EXCAVATED FROM FACE

NOTE : 1- DOWN TIME 1640 ~ 1800 DUE TO HYDROLIC PROBLEM.

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
(A)	PUENTE FM	—	OLIVE GRAY/ YELLOWISH BROWN	—	—	MOD.	FRESH	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED	X	BLOCKY	
SQUEEZING		SWELLING		SHEARED	X	WATER	

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

SWING

DATE: 6/30/93 TIME: 2220

HEADING: **HOLLYWOOD - AL**

STATION: 465+56 ✓

RING NO: 108

PHOTOGRAPH NO: —

SOIL / ROCK SAMPLE NO: —

INSPECTOR: ST

HARD ROCK IN - RING #: —

HARD ROCK OUT - RING #: —

GPR SURVEY RING #: —

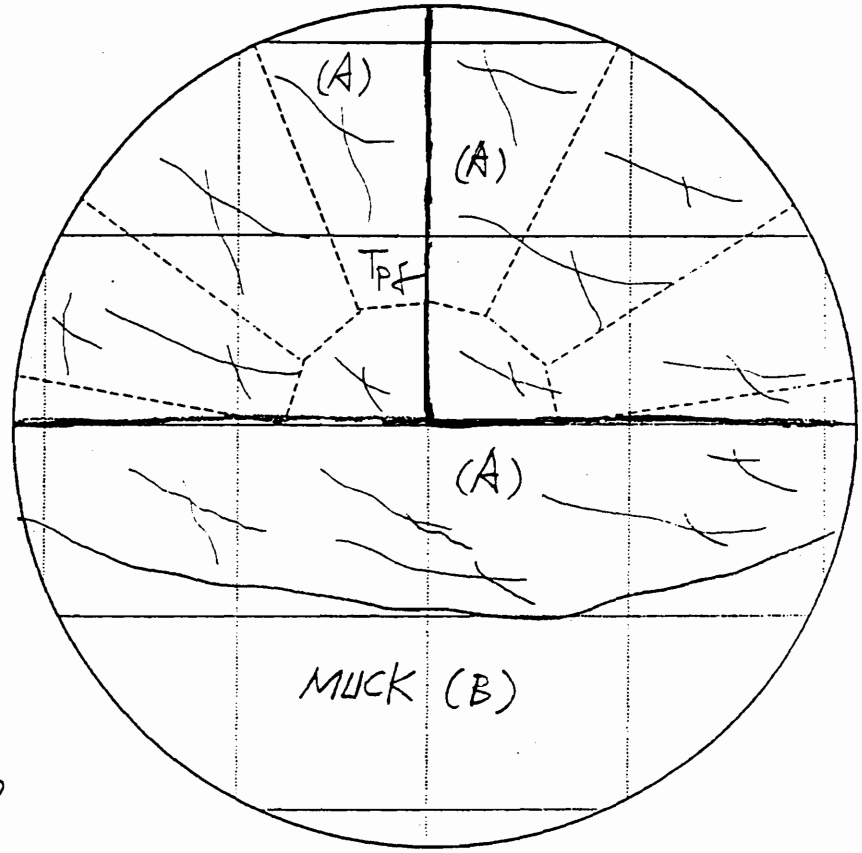
ROCK SPLITTER USED: —

SLOWER PROGRESS: —

GROUNDWATER INFLOW GPM: —

GAS TESTING: —

COMMENTS: —



SCALE 1" = 5'

(A) - HIGHLY SHEARED AND JOINTED PUENTE FM. SOME OXIDIZED PUENTE IS PRESENT AT THE CROWN AREA (YELLOWISH BROWN IN COLOR). FACE IS EASY TO EXCAVATE. NO WATER SEEPING OUT OF FACE
(B) - MUCK EXCAVATED FROM FACE

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
(A)	PUENTE FM	—	OLIVE GRAY/YELLOWISH BROWN	—	—	MOD.	FRESH	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED	X	BLOCKY	
SQUEEZING		SWELLING		SHEARED	X	WATER	

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 6/30/93 TIME: 23:40

HEADING: HOLLYWOOD - AL

STATION: 465+60

RING NO: 109

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: JB

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

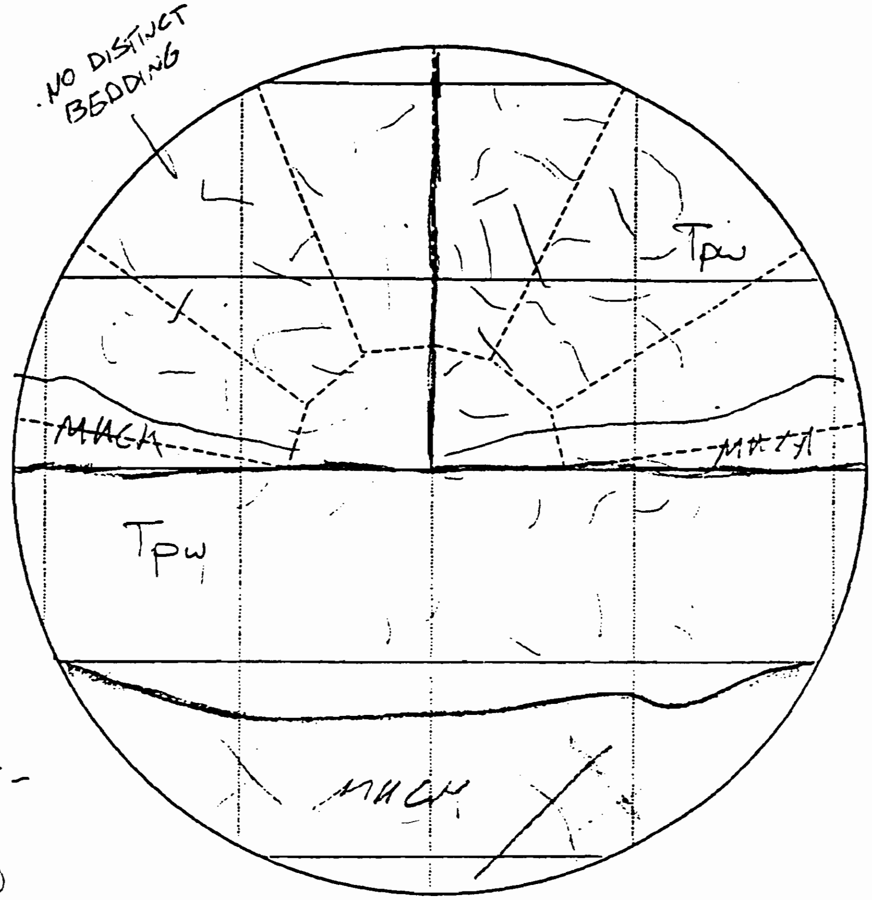
ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM: 0

GAS TESTING:

COMMENTS: PUENTE Fm, ORANGE-BROWN, MOIST MODERATLY SHEARED WITH WELL DEVELOPED SLICKENSIDES, WEATHERED OVER ENTIRE FACE, BEDDING IS HIGHLY DISRUPTED AND NOT DISTINCT, NO HARD ROCK, MATERIAL IS FRIABLE AND EASILY MINED BY TSM



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	PUENTE Fm		ORANGE [illegible]			VERY WEAK	WEATHERED	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 6/30/93 TIME: 1:10

HEADING: HOLLYWOOD - AL

STATION: 465+68 ✓

RING NO: 111

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: JB

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

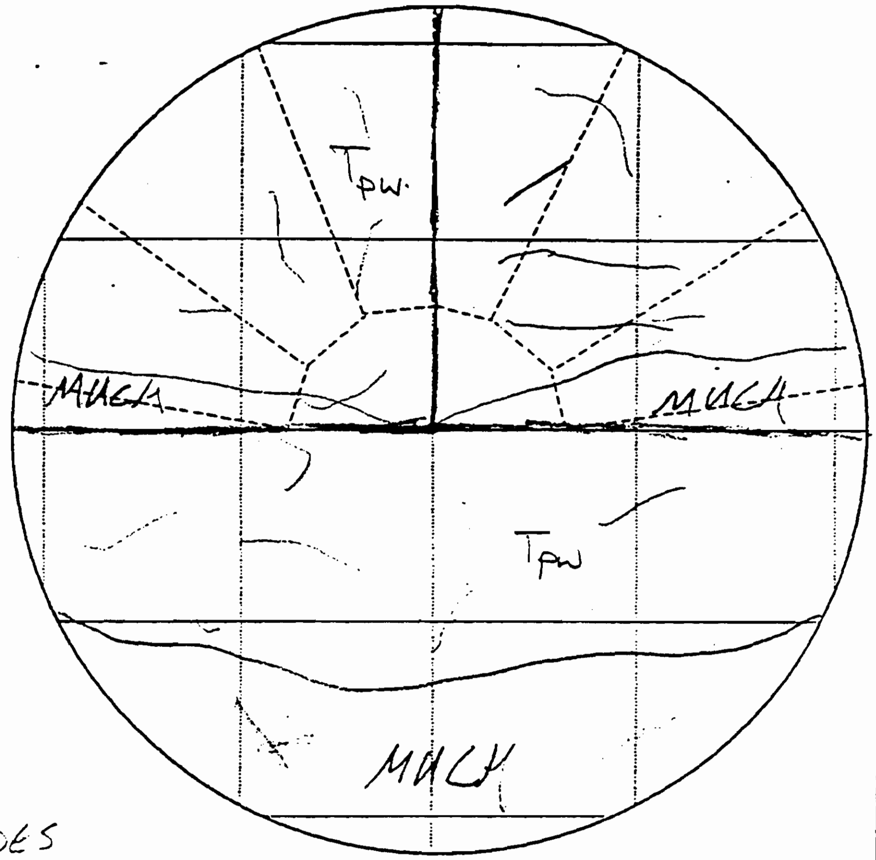
ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM: 0

GAS TESTING:

COMMENTS: PUENTE Fm, ORANGE BROWN, MODERATELY SHEARED WITH CLOSELY SPACED SLICKENSIDES HIGHLY WEATHERED, NO BEDDING APPARENT, PUENTE IS FRIABLE AND EASILY MINED BY TSM



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
(A)	PUENTE		BROWN			VERY WEAK	[illegible]	MOIST

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

159

3

CONTRACT B251

PARSONS - DILLINGHAM GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9/9/93 TIME: 845

HEADING: VERMONT - AL

STATION: 441+25

RING NO: 440

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO: –

INSPECTOR: ELLIS

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

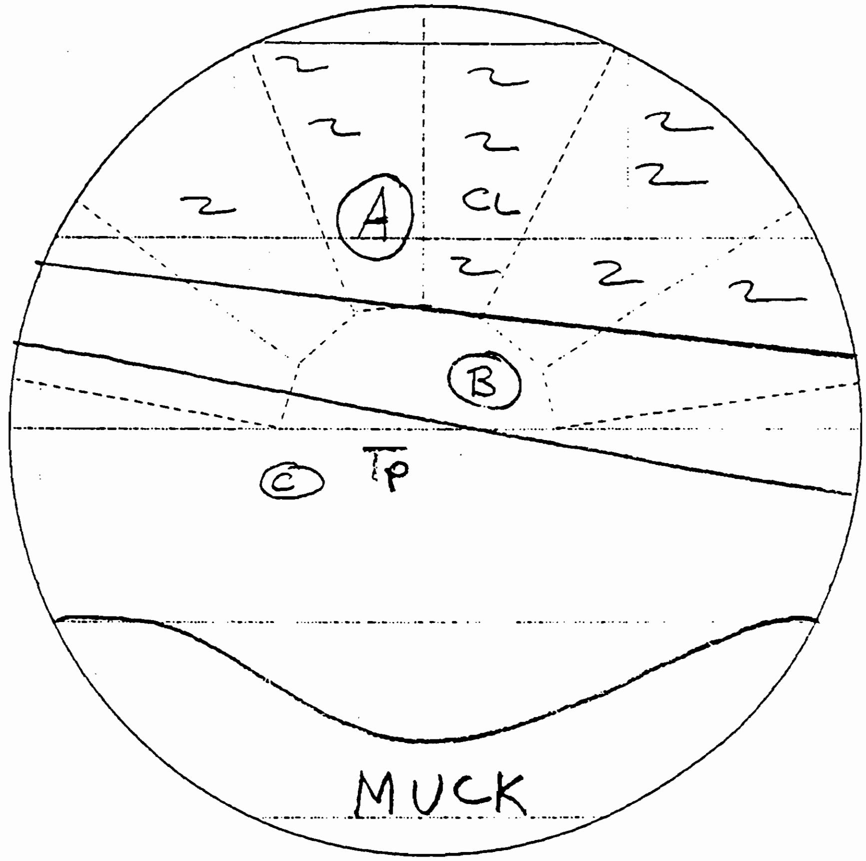
GPR SURVEY RING #:

ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM:

GAS TESTING:



SCALE 1" = 5'

COMMENTS: A. Reddish-Brown CLAY, Low Plasticity medium stiff, moist ravels during mining
B. Colluvium - sam as A with 20% chunks of weathed Puente

C. Puente formation (Tp) HIGHLY WEATHERED - crumbles to touch Grayish green. Bedding not very well defined but mostly horizontal, Very Very weak
mining - Operator using ripper tooth & spade over inner 60 % of face. Shield shoving caves rest.

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	CLAY	CL	Brn	–	medium stiff	–	–	moist
B	COLLUVIUM	CL	Brn	–	medium stiff	–	–	moist
C	Tp	Puente	Grey Grn	–	–	Very Weak	Yes Very	moist.

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

METRO

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9-9-93 TIME: 11:17

HEADING: VERMONT - AL

STATION: 440+97

RING NO: 447

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO: —

INSPECTOR: Ellis

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

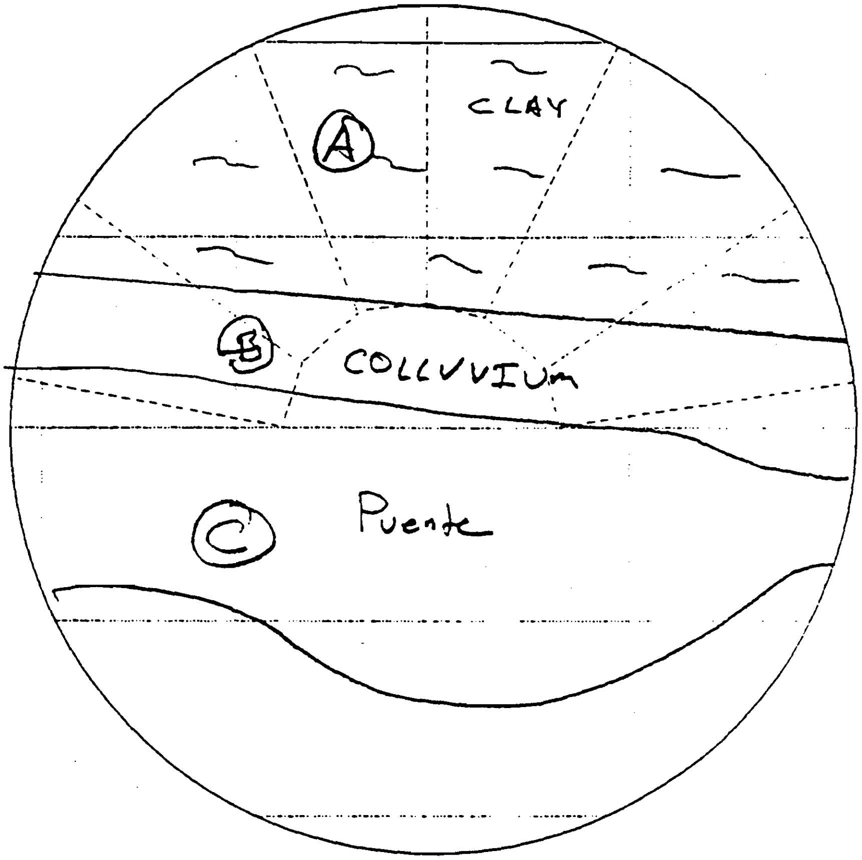
ROCK SPLITTER USED:

SLOWER PROGRESS:

GROUNDWATER INFLOW GPM:

GAS TESTING:

COMMENTS:



SCALE 1" = 5'

A. Reddish Brown CLAY
Medium stiff, moist Low Plasticity

B. Colluvium. Reddis brown CLAY with 10% chuncks of weathered Puente, moist medium stiff

C. Puente Formation Tp Olive brown highly weathered very weak bedding difficult to determine due to weathering

Mining Operator using ripper tooth and spade on puente area, Clay is raveling from the crown area in large chuncks during shove

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	CLAY	CL	Red Brn	—	Medium stiff		-	moist
B	Colluvium	CL	Red Brn	—	Medium stiff		-	moist
C	Puente	TP	Puente	—		Very Weak	Very	moist

FACE CLASSIFICATION

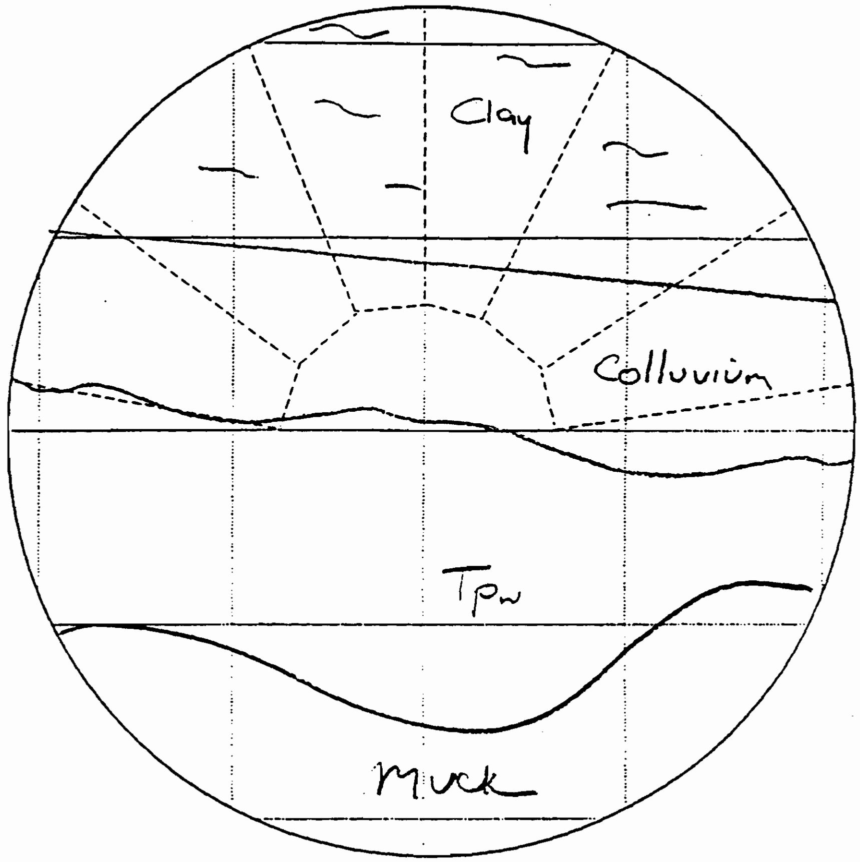
SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		~~JOINTED~~		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

161

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9-9-93 TIME: 1345
HEADING: VERMONT - AL
STATION: 440+73
RING NO: 453
PHOTOGRAPH NO: —
SOIL / ROCK SAMPLE NO:
INSPECTOR: Ellis
HARD ROCK IN - RING #:
HARD ROCK OUT - RING #:
GPR SURVEY RING #:
ROCK SPLITTER USED:
SLOWER PROGRESS:
GROUNDWATER INFLOW GPM:
GAS TESTING:
COMMENTS:
A. Reddish Brown CLAY moist, medium stiff low plasticity.



SCALE 1" = 5'

B. Colluvium – Reddish brown CLAY moist, medium stiff contains 10-20% pieces of weathered puente

C. Puente formation (Tp) Very weathered, crumbles to touch Lt Brown siltstone/claystone. Bedding not well defined
Mining – operator using tooth over puente surface and
 clay ravels during shove

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	CLAY	CL	Red Brown	—	—	Medium stiff	—	moist
B	"	CL						moist
C	Puente	Puente	Olv gr n	—	—	—	Mod weak	moist

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		~~JOINTED~~		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

CONTRACT

B251

(162)

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

SWING

DATE: 9/9/93 TIME: 1740

HEADING: VERMONT - AL

STATION: 440+53

RING NO: 458 (3rd SET OF THIS SHIFT)

PHOTOGRAPH NO: —

SOIL / ROCK SAMPLE NO: —

INSPECTOR: ST

HARD ROCK IN - RING #: —

HARD ROCK OUT - RING #: —

GPR SURVEY RING #: —

ROCK SPLITTER USED: —

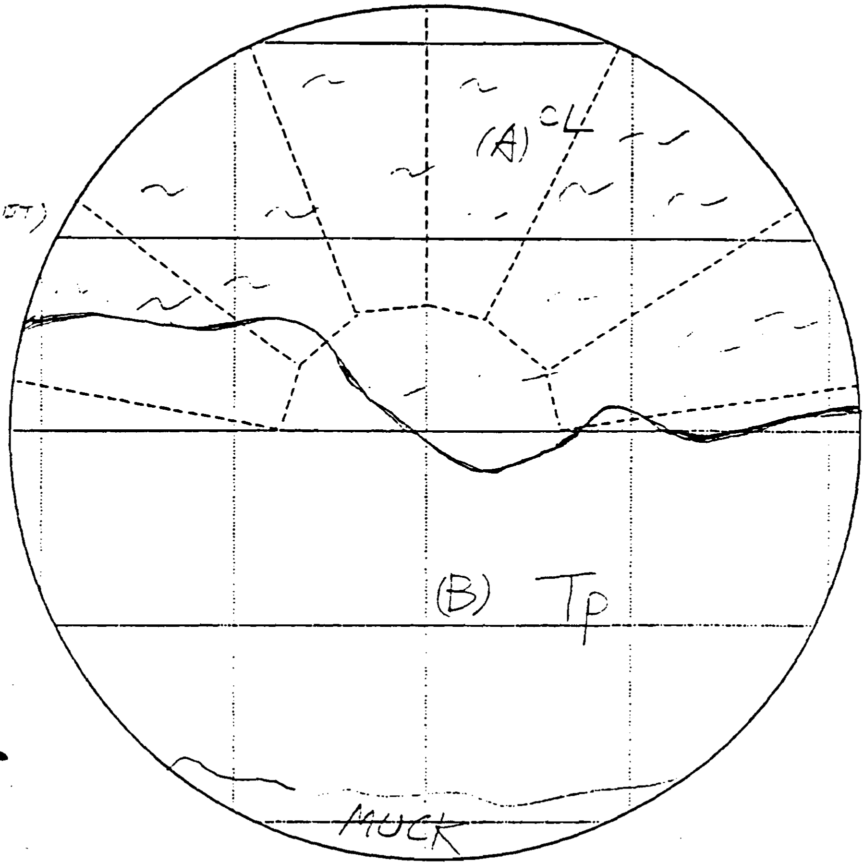
SLOWER PROGRESS: —

GROUNDWATER INFLOW GPM:

GAS TESTING: —

COMMENTS:

(A) ALLUVIUM - CLAY (CL). OLIVE GRAY IN COLOR. MEDIUM STIFF. MOIST.



SCALE 1" = 5'

(B) PUENTE FM. HIGHLY WEATHERED. BEDDING IS NOT WELL DEFINED BUT MOSTLY HORIZONTAL. MOIST. VERY WEAK. ORANGE BROWN IN COLOR.

NOTE: DOWNTIME 1800 ~ 2055 DUE TO SURVEYING LASER PROBLEMS.

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
(A)	ALLUVIUM - CLAY	CL	OLIVE GRAY	—	MEDIUM STIFF	—	—	MOIST
(B)	PUENTE FM	Tp	ORANGE BRN	—	—	WEAK	HIGHLY WEATHERED	MOIST

FACE CLASSIFICATION

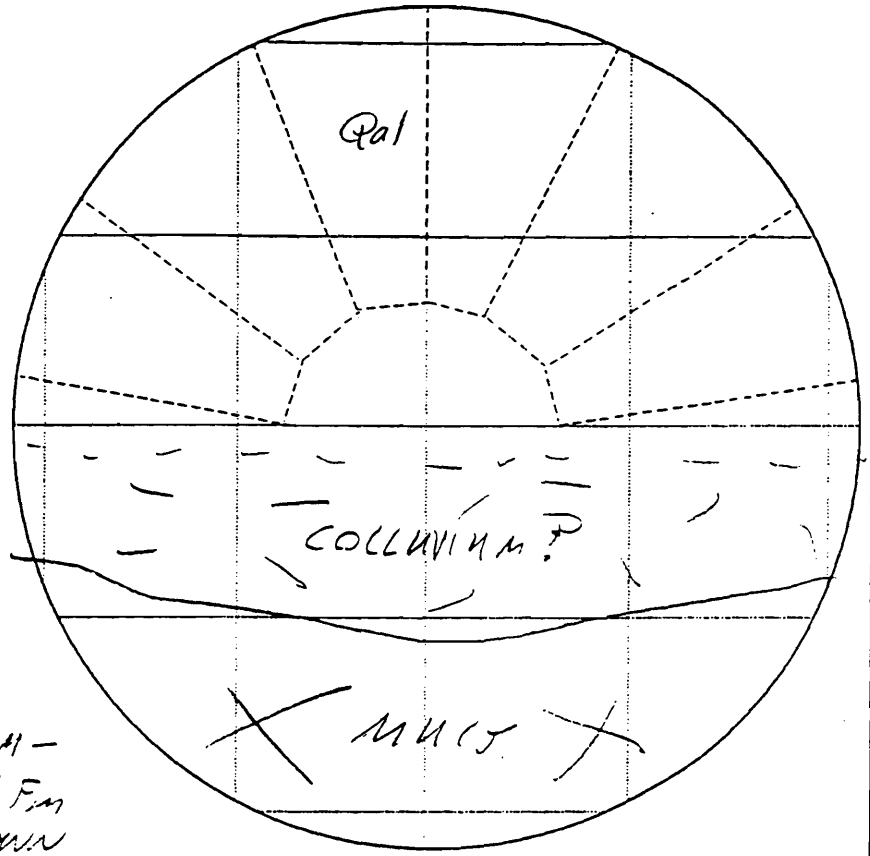
SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

163

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9/9/93 TIME: 1:05
HEADING: VERMONT - AL
STATION: 440+45
RING NO: 460
PHOTOGRAPH NO:
SOIL / ROCK SAMPLE NO:
INSPECTOR: JB
HARD ROCK IN - RING #:
HARD ROCK OUT - RING #:
GPR SURVEY RING #:
ROCK SPLITTER USED:
SLOWER PROGRESS:
GROUNDWATER INFLOW GPM: <1
GAS TESTING:
COMMENTS: ALLUVIUM / COLLUVIUM - HIGHLY WEATHERED PUENTE Fm
ALLUVIUM IS REDDISH-BROWN MOTTLED, CLAY (CL) MOIST, STIFF, NO WATER SEEPAGE
COLLUVIUM IS HIGHLY WEATHERED PUENTE CLAY, GREEN-BROWN, MOTTLED BEDDING IS IRREGULAR



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	ALLUVIUM	CL	RED BROWN		STIFF	WEAK	—	MOIST
B	COLLUVIUM	CL	GREEN BROWN		STIFF	WEAK	HIGHLY	MOIST

FACE CLASSIFICATION

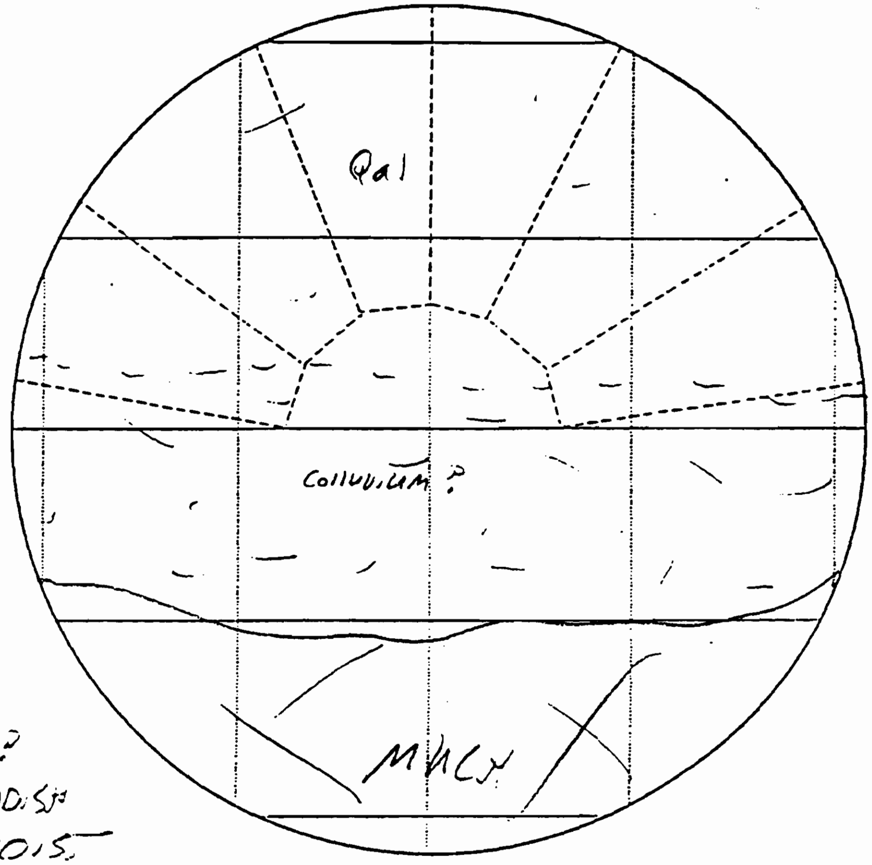
SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

(164)

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9/9/93 TIME: 3:15
HEADING: HOLLYWOOD - AL
STATION: 440+32
RING NO: 463
PHOTOGRAPH NO:
SOIL / ROCK SAMPLE NO:
INSPECTOR: JB
HARD ROCK IN - RING #:
HARD ROCK OUT - RING #:
GPR SURVEY RING #:
ROCK SPLITTER USED:
SLOWER PROGRESS:
GROUNDWATER INFLOW GPM: <1
GAS TESTING:
COMMENTS: ALLUVIUM / COLLUVIUM?
ALLUVIUM IS MOTTLED REDDISH BROWN CLAY, (CL) STIFF, MOIST NOT LAYERED, COLLUVIUM IS BROWN/GREEN MOTTLED WITH IRREGULAR BEDDING — CLAY (CL)



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE

FACE CLASSIFICATION

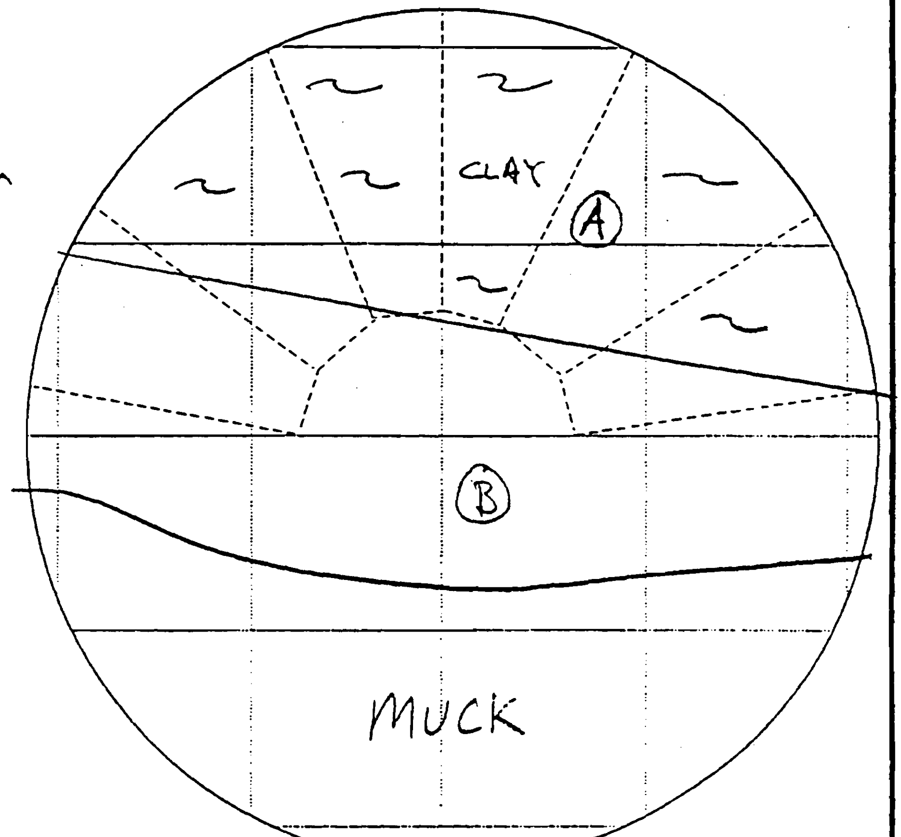
SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

(165)

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9-10-93 TIME: 818
HEADING: VERMONT - AL
STATION: 439+84 ← 21 foot correction
RING NO: 471
PHOTOGRAPH NO:
SOIL / ROCK SAMPLE NO:
INSPECTOR: ELLIS
HARD ROCK IN - RING #:
HARD ROCK OUT - RING #:
GPR SURVEY RING #:
ROCK SPLITTER USED:
SLOWER PROGRESS:
GROUNDWATER INFLOW GPM:
GAS TESTING:
COMMENTS:



SCALE 1" = 5'

A. Reddish Brown CLAY Low Plasticity, Medium stiff raveling, moist. Trace of wet areas on face

B CoAlluvium - Redding Lt Brown CLAY low plasticity contains 10% chuncks of weathered puente.

Mining - Operator using ripper tooth to excavate inner 60% of face outter 40% caving with shove of shield

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	CLAY	CL	Red Brn		medium stiff	—		
B	Co Alluvium	CL	Lt Brown		medium stiff	—		

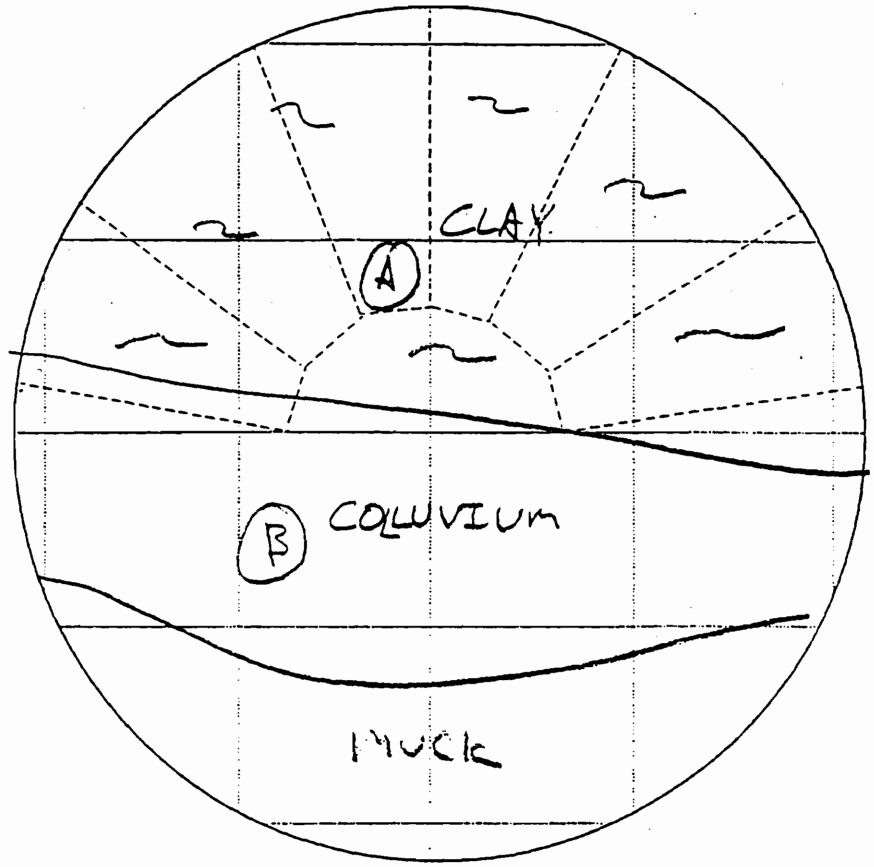
FACE CLASSIFICATION

SOIL				ROCK			
~~FIRM~~		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9-10-93 TIME: 1223
HEADING: VERMONT - AL
STATION: 439+52
RING NO: 479
PHOTOGRAPH NO:
SOIL / ROCK SAMPLE NO:
INSPECTOR: Ellis
HARD ROCK IN - RING #:
HARD ROCK OUT - RING #:
GPR SURVEY RING #:
ROCK SPLITTER USED:
SLOWER PROGRESS:
GROUNDWATER INFLOW GPM:
GAS TESTING:
COMMENTS:



SCALE 1" = 5'

(A) - Reddish Brown CLAY medium stiff, raveling during excavation moist

(B) Colluvium - Reddish light Brown CLAY with 10% pieces of weathered puente medium stiff to soft.

Mining - operator using spade over inner 50% of face.

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	CLAY		Red brown		medium stiff			moist
B	CLAY		Red Lt Brn		medium stiff			moist

FACE CLASSIFICATION

SOIL				ROCK			
~~FIRM~~		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
~~SQUEEZING~~		SWELLING		SHEARED		WATER	

METRO

CONTRACT
B251

PARSONS - DILLINGHAM
GEOTECHNICAL TUNNEL HEADING REPORT

DATE: 9-10-93 TIME: 1424

HEADING: VERMONT - AL

STATION: 439+36

RING NO: 483

PHOTOGRAPH NO:

SOIL / ROCK SAMPLE NO:

INSPECTOR: Ellis

HARD ROCK IN - RING #:

HARD ROCK OUT - RING #:

GPR SURVEY RING #:

ROCK SPLITTER USED:

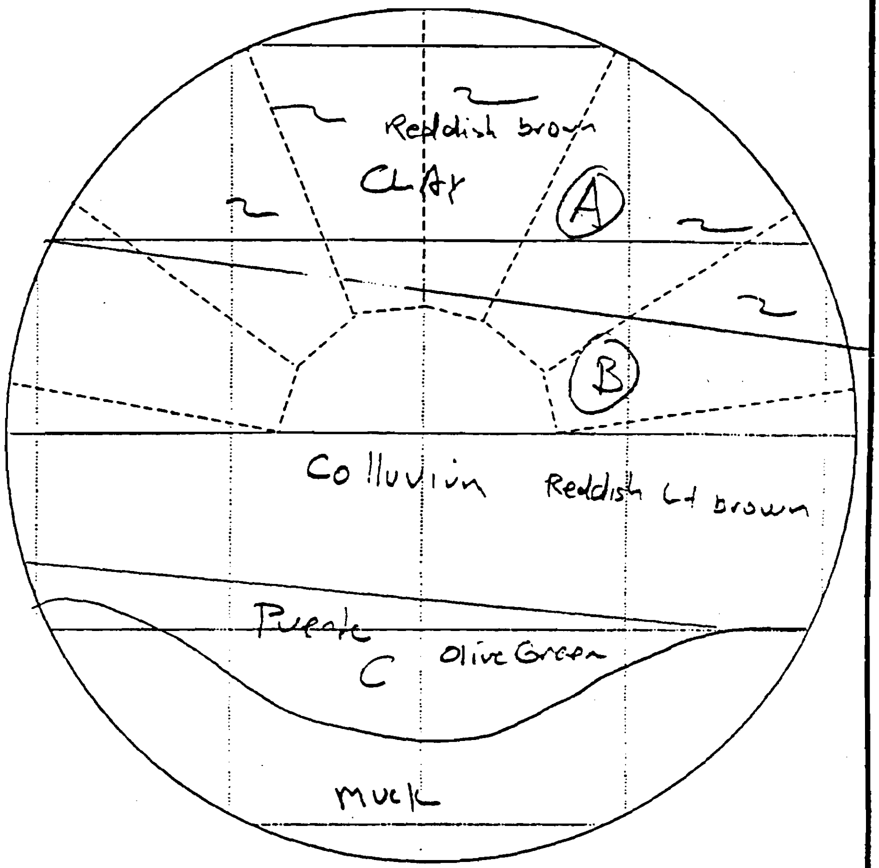
SLOWER PROGRESS:

GROUNDWATER INFLOW GPM:

GAS TESTING:

COMMENTS:

Mining - operator using spade over 60% of face



SCALE 1" = 5'

SOIL / ROCK PROPERTIES

NO	MATERIAL	USCS	COLOR	DENSITY	CONSISTENCY	STRENGTH	WEATHERING	MOISTURE
A	CLAY	CL	RB		mod stiff			moist
B	Colluvium	CL	RLB		mod stiff			↓
C	Puente	TP	OG			Very Weak	Very Weath	↓

FACE CLASSIFICATION

SOIL				ROCK			
FIRM		RUNNING		WELL BEDDED		MASSIVE	
RAVELING		FLOWING		JOINTED		BLOCKY	
SQUEEZING		SWELLING		SHEARED		WATER	

APPENDIX C

ESI Report (complete report not available as of submittal date)

Engineering
Systems
Inc.

3851 Exchange Avenue, Aurora, Illinois 60504
(708) 851-4566 • FAX (708) 851-4870

L.A. METRO RED LINE TUNNEL COLLAPSE INVESTIGATION METALLURGICAL REPORT

Report To:

Mr. George B. Morschauser
523 W. Sixth Street, Suite 400
Los Angeles, CA 90014

Date of Report: October 17, 1995
ESI File No. 3089A

COLORADO OFFICE: 4775 Centennial Boulevard, Suite 106, Colorado Springs, Colorado 80919
(719) 535-0400 • FAX (719) 535-0402

INTRODUCTION

On June 23, 1995, Mr. George Morschauser of Parsons-Dillingham contacted Engineering Systems Inc. (ESI) regarding the L.A. Metro Red Line tunnel collapse. ESI was requested to assist in the metallurgical investigation of a fractured water main that was near the collapse site. ESI was specifically asked to inspect and document the present condition of the pipe, develop a testing protocol to obtain the mechanical and metallurgical properties of the water main materials, and in conjunction with Failure Analysis Associates, Inc. (FaAA) conduct the mechanical testing.

BACKGROUND

A site inspection was conducted by ESI on June 25 and 26, 1995 at the construction site near Hollywood Boulevard and Barendo Avenue, Los Angeles, California. Mr. Donald Miner of Parsons-Dillingham provided ESI with background information regarding the tunneling activities near the point of collapse. Additionally, ESI was provided with a DAMES & MOORE report dated June 25, 1995, regarding the geotechnical evaluation of the subject area. The following contains observations of the subject pipe and appurtenances, the testing protocol, mechanical testing results, and the metallurgical evaluation of the subject pipe sections.

DOCUMENTATION & OBSERVATIONS

The tunnel collapse and subsequent sink hole developed on June 22, 1995. Shortly before the collapse, workers observed crown movement in the AL tunnel and water infiltrating through the tunnel. It was reported that the sink hole suddenly opened, resulting in a 40 feet by 60 feet by 10 feet deep hole. The final size of the hole was approximately 65 feet by 80 feet by 60 feet deep. Parsons-Dillingham was able to retrieve several sections of a 10-inch main with a valve and fire hydrant appurtenance. A total length of approximately 30' of pipe was recovered, which is roughly half of the pipe length that would have been affected by the sink hole. The following items were recovered:

1. 10" diameter, 13'-4" length of pipe. Photograph 1, segments C and D.
2. 10" diameter, 16' length of pipe with valve. Photograph 2, segments A and B.
3. 6" diameter, 9'-3" length of pipe, valve to hydrant pipe. Photograph 3.
4. Hydrant with vertical riser and 90 elbow. Photograph 4.
5. Two tie-rods connecting the hydrant to the valve. Photograph 5.

Observations

In general, the 10" diameter pipeline appeared to be a cast iron pipe with ¼" lining of a cement type material (Photograph 6). The inner and outer surfaces were coated with a bituminous material. The hydrant valve body and adjoining pipe appeared to have been fixed in a concrete thrust block, based on the appearance of concrete around the valve and adjoining pipe (Photograph 7). The 6" line was also coated with a bituminous material and contained a cement lining.

Based on observations of the collapse area, it appears the pipe segments A and B most likely were located near the AL 465 + 00 mark. Three water service connections to the 4830/4852 Hollywood Blvd. addresses were found. Distance measurements between these connections correspond with the spacing of the taps in the water main pipe. With the hydrant approximately at AL 465 +09, (approximately 9' east of the corner of the building) the hydrant would have been located within the perimeter of the sink hole. Photographs 8 and 9 and Figure 1 show the subject area, including the general location where pipe segments A and B were located, as well as the remaining water service lines to tap connections T1, T2 and T3. As mentioned previously, the location of the pipe segments C and D could not be determined.

The following are observations made at the time of the site inspection and during further inspections at FaAA on September 20, 27, and 28, 1995.

10" diameter, 13'-4" length of pipe

This section of pipe contained a full 12' length of pipe with bell and spigot ends, pipe segment C, joined to the bell end of a 1'-4" length of pipe, pipe segment D. Around this joint, outside the gasket area, was placed a cement type material (Photograph 10). Pipe fractures occurred just past the bell area of the smaller pipe section, pipe segment D, and in the bell area of pipe segment C, Photographs 10 and 11, respectively. No evidence existed indicating the position of the pipe section at the time the sinkhole developed, i.e. the top and bottom position. Additionally, the position of this pipe section with respect to the subject area sink hole could not been determined.

10" diameter, 16' length of pipe with valve and tee to hydrant

The 16' length of pipe contained two pipe segments A and B and a valve body with a 6" tee connection (Photograph 2). A 6" lateral line connected the main to a fire hydrant riser. The position of the valve was approximately at the mid length of the 16' pipe section and was contained in segment B. Photograph 12 shows the fractured end of pipe segment A. Note the crack path was jagged along half of the diameter (red arrow) and relatively smooth along the other half (black arrow). The opposing end of the pipe segment A, a bell end, remained joined within a spigot end of 10' section pipe segment B. Literature[1] states, the tension side of a cast iron specimen under a bending stress will exhibit a rough or irregular type fracture surface. Conversely, the compression side is relatively smooth and straight. Based on the literature it appears the top side of pipe segment A would be in compression and the bottom side in tension (Photograph 12). (Note Photograph 12 should be rotated 90° clockwise for proper orientation.)

The fractured end of the 10' pipe segment B is shown in Photograph 13. This spigot end fractured within the bell section of an adjoining pipe. The relative movement of the two sections could not be determined based on the fracture appearance.

---

[1]Gordon W. Powell, Shu-hong Chen, Carroll E. Mobley, Jr., A Fractography Atlas of Casting Alloys, Battelle Press, Ohio, 1992.

Three service taps were observed along the side of the pipe. Looking from the fractured end of segment A, the service taps were between the 2 and 3 o'clock position. Two 1½" taps (T1 and T2) were approximately 7' and 6' away from the hydrant valve, respectively. A 1" tap (T3) was approximately 4' from the valve. Photograph 14 shows the taps.

T3 appears to contain a fractured copper connection that was threaded into the pipe (Photographs 15 and 16). The fracture occurred at the outer diameter of the 10" pipe and the surface appeared relatively fresh. The fracture pattern indicates the fracture occurred due to a bending load, possibly combined with direct tension. The chevron pattern points (arrows, Photograph 16), to the origin of the fracture to be along the lower side of the tap with the fracture propagating toward the upper side of the tap. A thin cement paste existed on the inner diameter of the hole, indicating the connection was made before the cement lining operation (Photograph 17). Review of DWP drawing, Figure 1, indicates the cement lining operation was completed on or before April 22, 1991.

The 1½" taps, T1 and T2, were drilled and tapped to produce a threaded connection. The fittings and pipe segments that connected to these taps were not recovered. The pipe material adjacent to T2 was fractured in several areas (Photographs 18 and 19). A 2½" long by 2" wide piece of pipe material next to the tap which contained a 45-degree radial segment of threading was completely fractured from the pipe (Photograph 20). The threads contained in this piece were relatively intact, without significant signs of rusting or mechanical deformation. The height of the remaining threads appeared to have been greatly reduced (Photographs 21 and 22). Based on the deformation pattern it appears the tap was bent about the longitudinal axis of the 10" pipe. The upper threads of the tap were pulled outwardly until the pipe fractured, resulting in the lower threads, (the compression side of the tap), remaining intact.

The 1½" tap T1 was similar in construction to the adjacent tap T2 (Photograph 23). Fracturing of the pipe around the hole was not observed. However, the thread height was reduced over most of the inner diameter of the hole (Photograph 24). A cross section of material containing the pipe threads was cut out. Examination of this section revealed that the threads were mechanically deformed. Photographs 25 and 26 show the threads. It appears the threads of the tap pulled out of the 10" pipe connection, resulting in the deformation of the pipe thread with possible remnants of the service tap threads in the pipe thread's roots as shown in Photograph 27.

The cross-section of threads was examined in an scanning electron microscope (SEM). The examination revealed the direction of the deformation pattern was from the inner to the outer diameter. Photograph 28 shows the middle threads. Note the direction of the deformation pattern. Once again the deformation pattern indicated the tap was bent about the longitudinal axis of the 10" pipe with the upper threads of the tap being pulled outwardly until the tap pulled out resulting in the lower threads, (the compression side of the tap), remaining intact.

In summary, Figure 2 shows a schematic of pipe segment A and its position relative to the tap connections and fire hydrant. The directions of fracture of the taps and the tension/compression side of the fractured spigot end are indicated on this figure.

6" diameter, 9'-3" length of pipe, valve to hydrant pipe
No obvious signs of distress were observed in this pipe section. Both ends of the pipe were spigot ends that fit into the valve and elbow bells.

Hydrant with vertical riser and 90 elbow
The hydrant and riser section appeared to be free of distress. Based on the measured riser height, the bottom of the hydrant was approximately 3 feet above the centerline of the 6" diameter lateral pipe.

Two tie-rods connecting hydrant to valve
The tie rods and collars were found connected to the valve base and hydrant riser elbow. The tie-rods are currently bent and twisted most likely due to the collapse and recovery of the pipe segments. The bolted connections were observed to be relatively free from corrosion at the collar areas. However, the tie rods were observed to be moderately corroded at their mid point. Approximately 25-30% of the rod's cross section was reduced due to corrosion. It should be noted that the tie-rod diameter did not appear to have necked down due to the material yielding in the corroded areas.

METALLURGICAL ANALYSIS

Mechanical Test Protocol and Results
A total of nine tensile and nine Talbot Strip samples were obtained from pipe segments. Three strips approximately 3 inch wide by 12 inch long strips were taken from pipe segments A, B, and C. Photographs 29 through 31 show the locations where each of the strips were obtained. Note that the pipe segment A samples were marked T1, T2, and T3, corresponding to areas adjacent to tap locations 1 through 3. The other samples were obtained near the fractured ends of the respective pieces and labeled according to the corresponding pipe segment, i.e. B1, B2, B3, C1, C2, C3.

Both tensile and Talbot strip testing were conducted since ESI and FaAA were interested in different testing methods. Based on modern day American Water Works Association (AWWA) specifications, ESI requested that Talbot strips be used to determine the mechanical properties of the 10" cast iron water main. Appendix I contains the forward from AWWA C108-75, Cast-Iron Pipe Centrifugally Cast in Sand-Lined Molds, For Water or Other Liquids. The forward pertains to the history of the specification regarding the significant changes in the spec that have occurred over time. In the original standard AWWA C100-08 adopted in 1908, an acceptable method to determine the mechanical properties of the cast iron was to perform tensile tests. However, section II -Acceptance Tests of the forward states that in the 1940's it was found:

> "Correlations of the data showed that the Talbot strip modulus of rupture and secant modulus of elasticity values and the test bar load and deflection values specified in this standard represent acceptable pipe which meet the design burst and ring strengths."

Based on these AWWA findings, ESI determined the Talbot test would provide the best test method available to determine the condition of the pipe as it existed at the time the sink hole developed.

DWP indicated that the pipe most likely was pit cast in the early 1900's. Based on this information, the required modulus of rupture would be 30,000 psi and the modulus of elasticity should be no greater than 10,000,000 psi, based on AWWA C102-53, Cast Iron Pit Cast Pipe for Water or Other Liquids (Appendix II). This edition of the standard specifies the requirements for Talbot strip testing and how to determine the mechanical properties from the test results. Photographs 32 and 33 show Talbot strip test set-up at FaAA in Los Angeles, California. The test were run using an MTS load frame and electronics system.

Test results indicate the tested pipe segments, for all practical purposes, meet the minimum modulus of rupture and maximum secant modulus of elasticity requirements as shown in Table 1. Pipe segment C had two of three modulus of rupture values slightly below the 30,000 psi minimum.

Inspection of a C3 fracture surface revealed that the pipe wall thickness was reduced slightly in some areas due to local corrosion. Photographs 34 and 35 show sample C3, which exhibit local corrosion of the outer diameter. This sample actually broke outside the middle one-third length of the bar, therefore is not valid. AWWA standard requires that results be based on sound material samples and requires a re-test. This, of course, is for new materials and no guideline is given for materials that have been in service for a considerable time. C2, which broke approximately at the point of loading, and C1, which broke within the mid section, were valid tests and from an engineering perspective, the samples have retained their mechanical integrity and would most likely meet the original design conditions.

Table 2 shows the results of the tensile testing. A minimum 20,000 psi tensile strength is required according to AWWA C100-08. Note that each pipe segment had at least one sample that did not meet the minimum required tensile strength. The low value of B1 can be attributed to machining marks observed on the shoulder of the specimen. Segment C test results are lower than the required tensile strength. Considering the minimum tensile strength value is 85% of the required and given the age of the pipe, it is ESI's opinion, as stated previously, that the pipe has retained its mechanical integrity.

Physical Metallurgy Evaluation

A portion from each pipe segment A, B, and C were analyzed to determine their chemical constituents. Table 3 shows the results that indicates the pipe chemistry is consistent with gray cast iron. A microstructural analysis found the microstructure was also consistent with a gray cast iron material. Photograph 36 shows the typical observed microstructure from pipe segment A at T1. No abnormal inclusions or significant corrosion was observed in any cross section that would have abnormally affected the mechanical integrity of the cast iron pipe.

CONCLUSIONS

The following can be stated with respect to the field and laboratory observations:

1. Two sections of pipe containing four segments of pipe were inspected. Fracture surfaces were observed and fracture directions were documented when possible.

2. Pipe segment B contained a valve which fed a fire hydrant appurtenance. Segment A contained three service taps. Based on these observations, the position of the fire hydrant valve on pipe segment B was determined to be at approximately AL 465 +09, (approximately 9' east of the corner of the building located at 4830/4852 Hollywood Blvd).

The following can be concluded regarding the metallurgical analysis:

1. The 10" pipe line was determined to be a gray cast iron most likely produced using a pit casing method.

2. The Talbot strip testing concluded that the modulus of rupture and secant modulus of elasticity of at least one sample of each pipe segment met the minimum requirements of AWWA. Pipe segment C had two values below the minimum requirement due to localized corrosion. However, the values were not significantly below the required minimums and the extent of corrosion was minimal. Therefore, these factors would not have affected the overall mechanical integrity of the pipe segment. All samples from pipe segments A and B met the minimum modulus of rupture and maximum secant modulus of elasticity.

3. Tensile testing yielded the same basic trends in results as the Talbot testing. Segment C did not met the minimum requirements, but the results were not significantly lower than the required minimum required tensile strength given the materials age.

Respectfully submitted,

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