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# Scour Hazard Mitigation for Tick Canyon Wash Bridge

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## Abstract

This paper presents the case history, with particular emphasis on the geotechnical aspects, of a bridge scour countermeasure design project in California. In addition to local, contraction and long-term degradation scour, this bridge site is susceptible to a potential head-cut of about 10 meters due to downstream aggregate mining operations. After considering a range of options, a 46-m long tieback check dam retaining wall system consisting of cast-in-place concrete panels, and supported by Cast-in-Shell-Steel soldier and anchor piles, was proposed to mitigate scour hazard at this site. Design of the proposed check dam retaining wall system was controlled mainly by geotechnical considerations including soil piping potential due to seepage forces, lateral soil and water pressures, and pile lateral capacity. Results of the analyses performed to determine the various design parameters including the minimum panel embedment depth, pile embedment depth, and lateral pile capacity by considering pile-soil-pile interaction are presented in this paper. It is shown that a simple numerical analysis method utilizing p-y soil spring model can more accurately predict the complete response of a pile supported tieback retaining wall than the conventional empirical or semi-empirical methods.

## Introduction

Loss of foundation support due to scour has been the most common cause of bridge failure in the United States (Kattell and Eriksson 1998, Briaud et. al 1999). In California, about 4,700 State owned bridges crossing active rivers, streams, creeks or washes could be subjected to catastrophic failures due to scour. In response to Federal requirements, California Department of Transportation has initiated a comprehensive program to identify, evaluate and mitigate scour hazards for all the State owned bridges. More than one half of these bridges have been evaluated so far, and approximately three percent will require mitigation. Mitigation work began in 1998-99, and was to be completed in 10 years.

This paper summarizes the results of a site-specific geotechnical investigation conducted for a scour evaluation and hazard mitigation project in California. The project involved design and construction of a scour countermeasure for the Tick Canyon Bridge on Route 14 in Ventura County. An elevation view of the bridge is shown in Figure 1. This is a three-span continuous bridge supported on two end diaphragm abutments and two bents, all founded on pile foundations. The structure was built in 1963, and widened and seismically retrofitted in 1997 utilizing 610-mm diameter cast-in-place concrete piles. A detailed scour analysis (Avila, 1998) confirmed the categorization of this bridge as scour critical. Both the abutments and piers are

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supported on scour susceptible sandy alluvial soils. Aggravating the local scour hazard is the general degradation and scour of the wash bottom that has resulted from aggregate mining operations at its confluence to the Santa Clara River approximately 0.8 km downstream of the bridge. Field review indicated that the two separate grouted riprap check dams, previously constructed on the downstream side of the bridge as scour countermeasures, were severely undermined by continued erosion. A total head cut depth of about 10 m was estimated for this natural trapezoidal channel in the event of a 100-year design flood. The bridge structure was determined to be unstable for the calculated scour depth and in need of a scour countermeasure. After considering various scour mitigation measures, a tieback retaining wall consisting of cast-in-place concrete panels and supported by Cast-In-Steel-Shell (CISS) soldier and anchor piles was selected for design.

Significant geotechnical issues related to the design of the proposed check dam included lateral load capacity of the soldier and anchor piles by considering soil-structure interaction, and the effects of differential water levels on the opposite sides of the wall including seepage forces and soil piping potential. As in the cases of most California bridge sites, the geotechnical issues also included seismic hazards including fault rupture potential, ground motion, soil liquefaction and lateral spreading. Geotechnical issues during construction include excavation stability and pile driveability.

**Scour Evaluation**

The Tick Canyon Wash is a natural trapezoidal canal with a bottom width of about 15 m and side slope of 2:1 (H:V). The channel bottom slope at the site is about 0.012. The streambed consists of mostly sand with some gravel and cobbles, and is therefore highly susceptible to scour. The streambed remains mainly dry during most of the year with the exception of very little water within the pilot ditch in the center of the stream.

The bridge has a history of scour problems with short piles, steep streambed slope, high flow velocity and downstream gravel mining operations. It was noted in late 1970, that the channel had degraded by about 1.2m, probably due to the partial failure of a downstream check dam, and that the bridge was threatened by a mining operation in Santa Clara River about 0.8 km downstream of the bridge. To protect the bridge from scour resulting from this mining operation, two check dams were constructed about 15 m and 30 m, respectively, downstream from the bridge site. These check dams were considered essential to the stability of the channel bed. An existing earthen construction access road adjacent to the Santa Clara River also provided some protection.

The Wash drains approximately 14.8 km$^2$ mountainous areas with steep rocky ridges above the bridge site. A discharge of $Q_{100}$ = 67 m$^3$/s was estimated for a 100-year flood by using the USGS Regional Regression equation with urbanization only in the lower basin and a mean annual precipitation of 457 mm. Water surface profiles and flow velocities were calculated by using the computer program HEC-2 (U.S. Army Corps of Engineers 1991). These profiles were extended approximately 60 m upstream and downstream of the bridge. The values of the Manning roughness coefficient "n" of 0.045 and 0.03 were chosen for the channel bank and the streambed, respectively. A water depth of 2 m at the upstream side of the bridge was calculated based on an additional 915 mm width added to each of the two bents in the waterway, and the streambed elevation at 501.7m. The corresponding stream velocity was estimated to be 4.5 m/sec. Potential total scour (local, contraction and long term degradation) was estimated to be about 3.1 m.

Currently, the bent piles are protected by gabions. Scour hazard at abutment is not of concern since the abutments are not reachable by the 100-year flood. Recent field observations indicated that the two existing check dams were severely deteriorated. In the event of a failure of these dams, the channel would degrade rapidly and pose a significant threat to the bridge. Because of the close proximity to the Santa Clara River, the mining effect would be felt at the bridge site in the form of a head cutting (dropping of the channel bottom), and the earthen road and the deteriorated check dams could be washed out with a vertical elevation change of 10 m at the bridge. Removal of streambed material at the bridge site would reset the channel bottom elevation to about 491.9 m, essentially to pile tip elevation at the supports.

**Scour Countermeasure**

Potential scour countermeasures and their drawbacks are summarized in Table 1 below:

**Table 1. Potential Scour Countermeasures**

| Countermeasure | Drawbacks |
|---|---|
| 1. Install a series of engineered check dams that could accommodate the potential head-cut and scour. | A) This will necessitate going out of the State Right-of Way (ROW).<br>B) An existing sanitary sewer between the structure and the existing first check dam will be an issue.<br>C) Check dams will need to be inspected after significant events to ensure their structural integrity.<br>D) Will require passive monitoring system such as “float outs” |
| 2. Install one large check dam within or outside the ROW. | A) Check dam will be greater than 14 m in height, and would, therefore, need to be a concrete dam. The free-fall of water would be substantial.<br>B) The existing sanitary sewer will be an issue.<br>C) The check dam will need to be inspected after significant events to ensure their structural integrity<br>D) Will require passive monitoring system such as “float outs” |
| 3. Replace the bridge structure and install new piles to much greater depth to accommodate the anticipated scour and head-cut. | A) Costly.<br>B) Both structures were just widened and seismically retrofitted in 1997. |
| 4. Super bents with foundations deep to accommodate the projected scour and head-cut. | A) Costly.<br>B) Placement of the bents will reduce the width of the waterway possibly causing a water way adequacy problem.<br>C) Will not protect the abutments if potential head-cut migrates upstream. |

| | |
|---|---|
| 5. Footing strengthening for both abutments and piers. This will entail installing cast-in-drilled hole piles to a depth low enough to mitigate both scour and head-cut hazards. | A) Costly.<br>B) Construction will necessitate drilling through deck.<br>C) Traffic control and detour will be an issue.<br>D) Will result in long unsupported pile length. |
| 6. Monitor the bents, the existing check dams and the earthen construction road with electronic devices (float outs). | A) This is a passive scour-monitoring alternative, and will not eliminate the potential of scour or head cutting reaching the structure. |

After discussion in an inter-disciplinary meeting, a 50-m long check dam–retaining wall system shown in Figures 2 and 3, which eliminates many of the above drawbacks, was selected. The proposed check dam consisting of two rows of 914-mm diameter CISS piles and concrete panels is to be constructed on the down-stream side of the bridge. As shown in Figure 3, the front row of piles are designed as soldier piles, with a single tieback per pile, for a retaining height of 10 meters in the event of the anticipated head-cut. The back row of piles will act as anchor piles to provide additional lateral resistance to the front piles through high strength rods. Site specific-geotechnical issues related to the design and construction of the check dam-retaining wall are summarized in the following sections.

**Subsurface Conditions**

A subsurface exploration comprising two rotary wash borings was conducted at the site of the proposed check dam. Four exploratory borings drilled in 1994 for the bridge widening and seismic retrofit project were also reviewed as part of this investigation. These borings included measurement of the Standard Penetration Test (SPT) blow count as per ASTM Standard D1586 at a depth interval of 1.5 m. Soils were continuously logged and classified in the field during drilling in accordance with the Unified Soil Classification system. Based on this information, the subsurface profile at the site consists of a 5 to 10 m thick layer of loose to low medium dense silty sand underlain by 15 to 20 m of fine to medium-grained, dense to very dense sand with some gravel and small to large cobbles. Gravel and cobble conglomerate formation underlies the above units to the maximum explored depth of about 30 m below existing ground surface. Two interbedded thin layers of stiff sandy clay were encountered at 15 m and 24.5 m below the existing the ground surface. Groundwater was measured during the field exploration at a depth of about 25 m below the existing channel bottom. Though deep groundwater was observed at this site, seasonal surface water may form a temporary groundwater mound around the channel's watercourse and saturate the sand layers.

**Seismic Hazards**

The project site is located within a seismically active region of Southern California. The primary seismic hazards include ground shaking and ground rupture during earthquakes. Based on the Department's current design practice, the San Gabriel fault zone located about 8 km southwest from site is the controlling seismic source. The San Andreas Faults is located about 24 km northeast from the bridge site. Based on California Department of Mines and Geology (CDMG, 1994), this northeast-southwest trending fault is of reverse oblique type and has experienced displacements during the Holocene period. This fault is capable of generating a Maximum Credible Earthquake (MCE) of moment magnitude, $M_w$=7.25, and the corresponding Peak Bedrock Acceleration (PBA) at the site is estimated to be about 0.5g (Maulchin, 1996). The corresponding Peak Ground Acceleration (PGA) is estimated to be the same as the PBA since the soil profile can be classified as the NEHRP (BSSC, 1994) Type D. The project site is not considered susceptible to ground rupture since no known fault crosses or extends toward the site.

Based on the depth of the permanent groundwater table, the site is not considered susceptible to soil liquefaction during seismic events. However, as mentioned above, temporary water mounds can form around the watercourse and saturate the near surface soils. In that case, localized zones of loose to medium dense sand within the upper 5.0 to 10 m are considered susceptible to liquefaction during strong seismic events. The potential for lateral spreading at the site is considered to be low due to localized zones of liquefiable soils.

Seismic design for retaining structures includes consideration of the seismically induced active lateral earth and water pressures, in addition to the static lateral pressures, due to ground motion. The potential effects of soil liquefaction include increased seismically induced active lateral pressures, and loss of passive soil resistance and bearing capacity. In the absence of any significant differential scour or head-cut, the retaining wall will not be subjected to any significant unbalanced static or seismic lateral pressures. A 100-year flood resulting in 10 m of head-cut and a MCE resulting in significant ground shaking and liquefaction are two extreme events that are very unlikely to occur simultaneously at the site. Therefore, the design of the proposed check dam is based only on the static lateral loading due to the anticipated 10 m head-cut. It is recognized that any head-cut will be repaired immediately following storm events.

**Minimum Panel Embedment below Potential Head-cut Elevation**

In the event of a 10-m head-cut, the total hydraulic head difference between the upstream and downstream sides of the wall creates an unbalanced hydrostatic pressure on soil particles. This unbalanced hydrostatic pressure may lead to “boiling” or “piping” in the soil on the down-stream side, a phenomenon in which soil particles float or flow due to reduction in the submerged unit weight from the upward unbalanced seepage pressure. Therefore, the concrete panels are need to be placed deep enough so that the unbalanced hydrostatic pressure becomes insignificant to cause any “boiling” or “piping” in the soil on the down-stream side.

The reduction in the submerged effective unit weight due to the upward unbalanced pressure may be estimated by the following expression (Terzaghi 1954 and United States Steel 1975) since the soil on the down-stream side of the wall has fairly uniform permeability.

$$\Delta\gamma' = 20H_u/D \tag{1}$$

Where $\Delta\gamma'$ = reduction in effective unit weight of soil (pcf) on the down-stream side; $H_u$ = the total hydraulic head difference between the up- and down-stream side of the wall; and D = embedment depth as shown in Figure 4.

Given that the total hydraulic head difference $H_u$ = 10 m, the required minimum embedment depth below potential head-cut elevation can be estimated from the Terzaghi's formula to be D = 4.5m (with a safety factor of 1.5). This minimum embedment depth should maintain a downward effective unit weight of soil particles at the down-stream side of the wall. A sheet pile cutoff wall or a grouted/injected cutoff wall may substitute the portion of the concrete panel below the head-cut elevation.

**Minimum Tieback Force and Embedment Depth for Soldier Piles**

Preliminary calculation indicated that the 914mm (36-inch) diameter CISS soldier piles were inadequate to support a 10 m high cantilever wall in the event of the maximum potential head-cut. Thus, a single tieback per pile supported by an anchor pile was designed to provide additional lateral support to the wall. The minimum tensile force on the tieback rod was estimated according to the Department's Trenching and Shoring Manual, Section 10 – Soldier Piles (Caltrans, 1995), as presented in Table 2 below for different tieback anchor depths and soil parameters. The corresponding minimum embedment depths for the soldier piles for static equilibrium are also presented in the table.

The minimum tieback force and the minimum pile embedment depth are functions of water level elevation and soil parameters. The soil parameters were estimated from correlations with SPT blow counts. This design was based on the maximum anticipated hydrostatic pressure due to a running water depth of 2 m on both sides of the wall corresponding to $Q_{100}$ = 67 $m^3$/s.

**Table 2. Minimum Tieback Force and Minimum Embedment Depth for Soldier Piles**

| Case | Tieback/Anchor Depth, m | Soil 1 $\phi_1$ ($^o$) | Soil 2 $\phi_2$ ($^o$) | Tieback Force | Pile Embedment |
|---|---|---|---|---|---|
| | | | | kN (kips) | M (ft) |
| 1 | 1.0 | 30 | 33 | 1517 (341) | 9.6 (31.5) |
| 2 | 1.5 | 30 | 33 | 1557 (350) | 9.5 (31.1) |
| 3 | 2.5 | 30 | 33 | 1655 (372) | 9.2 (30.1) |

$\phi_i$ – angle of internal friction of soil

Soil 1 and Soil 2 in Table 2 refer to the soil above and below the potential head-cut line. It is recommended that the minimum pile embedment depth be multiplied by a safety factor of two (2) to obtain the design pile embedment depth. From the Table 2, it can be seen that the ultimate tieback force for Case 1 – with a buried depth of one meter is approximately 9% less than that for Case 3 – with a buried depth of 2.5 meters. A shallower buried depth of tieback rods may be used if a lower ultimate tieback force is desirable for structural design.

**Lateral Load – Deflection Curves**

The minimum tieback force and soldier pile embedment depths presented above are based on a semi-empirical method in which pile deflection shape, and full mobilization of the ultimate soil passive resistance and active pressure are assumed. This approach may be appropriate for rigid short piles, but cannot predict the lateral response of long flexible piles. A long flexible pile subjected to lateral loading interacts with its surrounding soil depending on its relative stiffness. Response of a laterally loaded long flexible pile should be determined based on methods that can more accurately determine its flexural deformation shape. Such methods include numerical analyses in which soil resistance is represented by a series of soil springs such as p-y curves. With the p-y soil spring model, the pile-soil interaction can be realistically taken into consideration and more accurate embedment depth, pile moment and shear can be obtained for pile structural design. In addition, analysis of the pile-soil-pile interaction involving the tieback rod can be performed based on the uncoupled lateral load – deflection curves for piles at the check dam and the anchor wall.

Uncoupled lateral load - deflection curves for 914mm (36-inch) diameter (PP914x19mm or PP36x3/4") CISS piles at the check dam and the anchor wall were determined using the computer program LPILE 4 (Ensoft, 2000). Two representative subsurface soil profiles used for the analyses of the soldier pile and the anchor pile are presented in Figures 6 and 7, respectively. The concrete modulus of elasticity for pile material was assumed to be $E_c$ = 23.4GPa (3400 ksi) which corresponds to a concrete compressive strength, $f'_c$ = 25 MPa (3600 psi). The reduction in the submerged effective unit weight of the soil due to upward unbalanced hydrostatic pressure was considered in these analyses. Lateral load - deflection curves for the soldier piles and the anchor piles are plotted in Figures 8 and 9, respectively. In Figure 9, the tieback force on the soldier pile is presented as negative values since it is in the opposite direction to the tieback force on the anchor pile. Representative plots of pile lateral displacement, moment and shear distributions with depth for a range of tieback load are shown in Figures 10 and 11 for the Case 1 in Table 2.

Utilizing the results presented in Figures 8 and 9, it can be shown, by trial and error, that at equilibrium a tieback force, F= 1180 kN (265 kips) is developed between the soldier pile and the anchor pile at a lateral displacement of 105 mm (4.1 inches) for the Case 1. The force is much smaller than that (F = 341 kips) obtained based on the semi-empirical method. Based on results presented in Figure 10, a length of 15 m (50 feet) is adequate for the anchor piles. From Figure 11, a minimum pile length of 24.5 m (80 feet) below which the pile response is insignificant may be used for the soldier pile. The corresponding embedment depth is comparable to that given in Table 2 with a factor of safety of 2.

## Construction Considerations

Construction issues related to the proposed check dam include pile driveability and stability of excavation to install the cast-in concrete panel. Hard driving conditions should be anticipated since cobbles and occasional boulders will be encountered during pile driving. Steel cutting shoes may be required to facilitate pile driving and to prevent pile damage. Predrilling can results in significant reduction in the pile lateral load capacity, and is not recommended.

It is anticipated that running surface water and/or shallow groundwater will be encountered, especially in the middle of the stream, during most of the year. Subsurface soils within the depth of the proposed excavation are predominantly granular, loose and highly permeable. Excavations in such soils are susceptible to instability when saturated and subjected to seepage forces. Thus, some seepage control, the extent of which will depend on the season and the excavation details, should be anticipated. Efforts should be made to schedule construction during the dry season when minor seepage may be expected in the excavations.

## Summary

Human activities such as aggregate mining can result in significant scour head-cut and undermine the stability of highway bridges. Designing deep bridge foundations or other common scour countermeasures may not be suitable for mitigating hazard from such deep scour. In this case, a tieback retaining wall type check dam supported by piles is considered to be a more appropriate scour countermeasure system.

Currently, pile foundations for tieback walls are designed based on empirical or semi-empirical methods such as those included in the Department's Trenching and Shoring Manual. These methods are based on an assumed pile deflection shape, uniform soil conditions and the full mobilization of the ultimate soil passive/active pressures. Such methods are usually more appropriate for short rigid piles, and may overestimate the tieback force and underestimate the minimum pile embedment depth.

Pile supported tieback walls can be more realistically analyzed by simple numerical methods using the p-y soil spring model. There are several computer programs commercially available for such analysis. A complete uncoupled pile response including lateral displacement, moment, and shear distribution with depth can be obtained to determine the required pile embedment depth and the cross section. The p-y method is very convenient for layered soils and piles with varying cross section. Furthermore, analysis can be performed to determine the complete system response including the tieback force by considering the soldier pile-soil-anchor pile interaction.

## References

1 Avila, C. (1998), "*Scour Evaluation and Preliminary Hydraulic Parameters for Advanced Planning Study (APS)*", Tick Canyon Wash, Br. No. 53-1547, File No. 07-LA-14-35.34, California Department of Transportation, Sacramento, California.

2 Briaud, J., Ting. F., Chen, H., Gudavalli, R. Perugu, S, and Wei, G. (1999), "*SRICOS: Prediction of Scour Rate in Cohesive Soils at Bridge Piers*", Journal of Geotechnical Engineering and Environmental Engineering, Vol. 25, No. 4, April.

3 BSSC (1994), *NEHRP Provisions*, 1994 Edition, Building Seismic Safety Council, Washington D.C.

4 California Division of Mines and Geology (1994), *The Fault Activity Map of California and Adjacent Areas*, Scale 1:750,000.

5 Caltrans (1995), *Trenching and Shoring Manual, Section 10 – Soldier Piles (Revised 06/95)*, California Department of Transportation, Sacramento, California.

6 Maulchin, L. (1996), *Caltrans California Seismic Hazard Map*, California Department of Transportation, Sacramento, California.

7 Ensoft, Inc., (2000), *Technical Manual of LPILE Plus 4.0 for Windows – A Program for Analysis of Piles and Drilled Shafts under Lateral Loading*, Austin, Texas.

8 Kattell, J and Ericksson, M (1998), "*Bridge Scour Evaluation: Screening, Analysis & Countermeasures*", Forest Service, U.S. Department of Agriculture.

9 Terzaghi, K. (1954), *Anchored Bulkheads*, Transactions, ASCE, Vol. 119.

10 U. S. Army Corps of Engineers (119), "*Water Surface Profile User' Manual*", HEC-2, Hydrologic Engineering Center, Davis, California.

11 United States Steel (1975), *Steel Sheet Piling Design Manual*, Updated and Reprinted by US Department of Transportation/FHWA with permission.

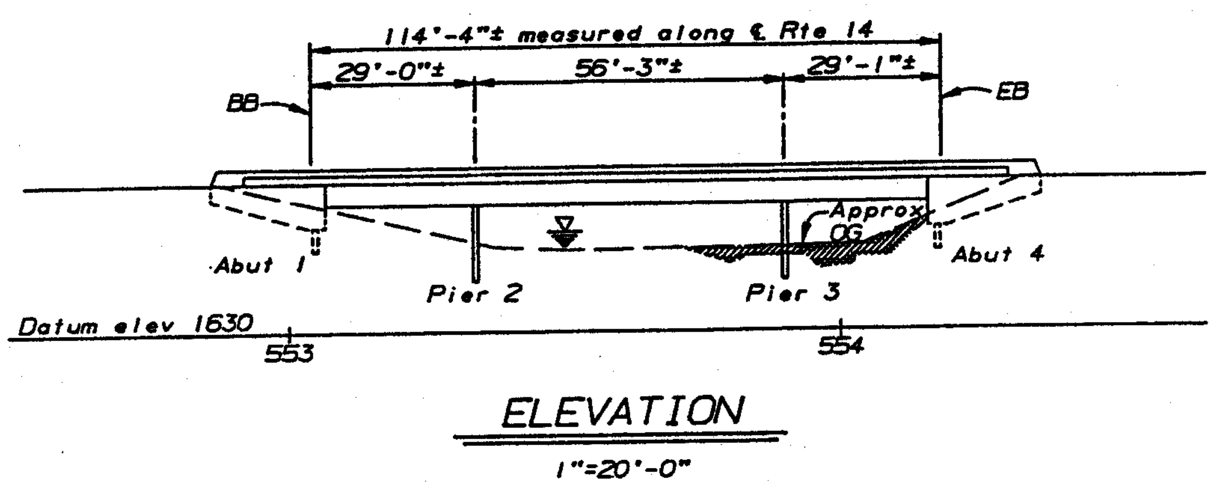



**Figure 1. Elevation View of the Tick Canyon Bridge**

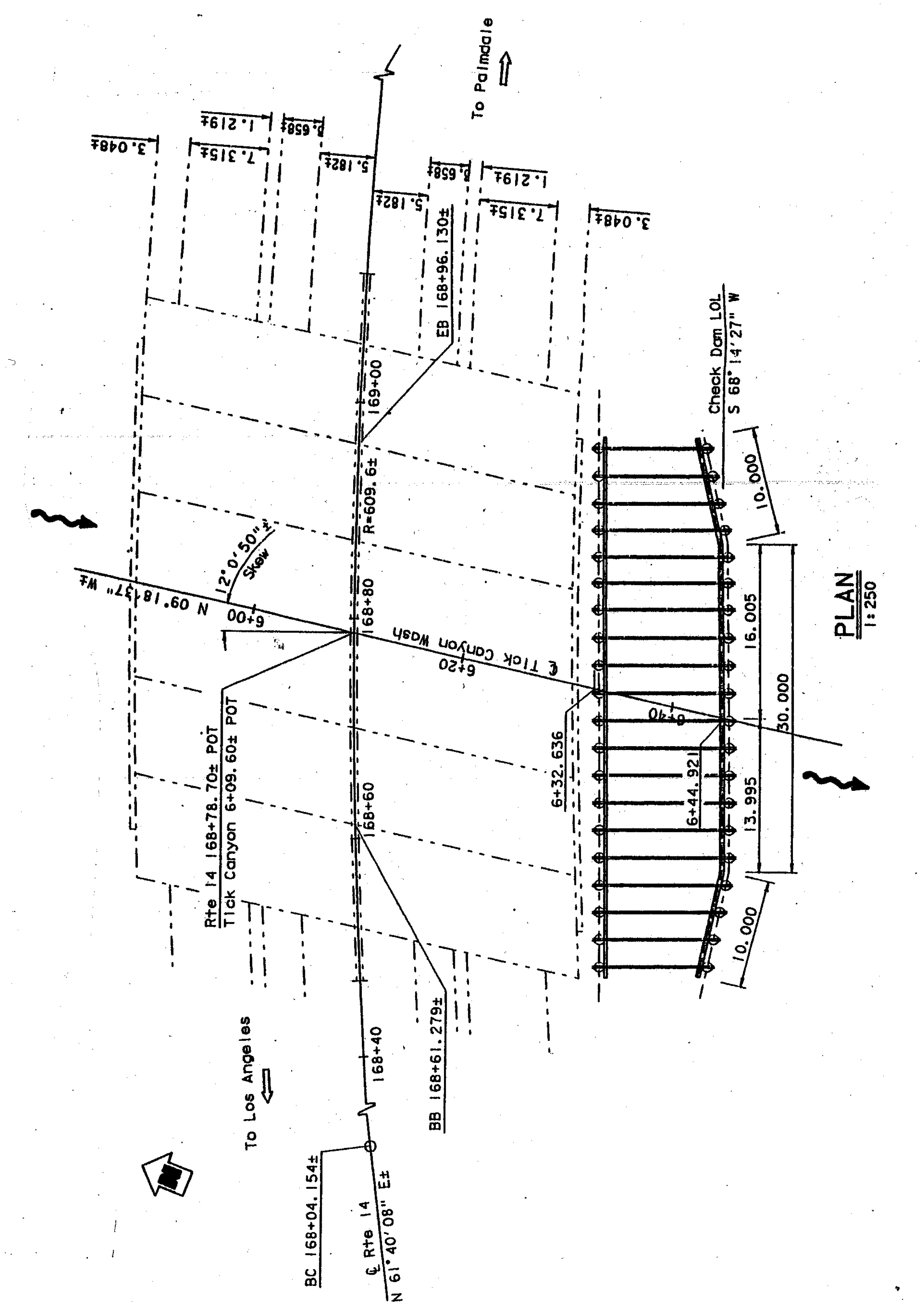
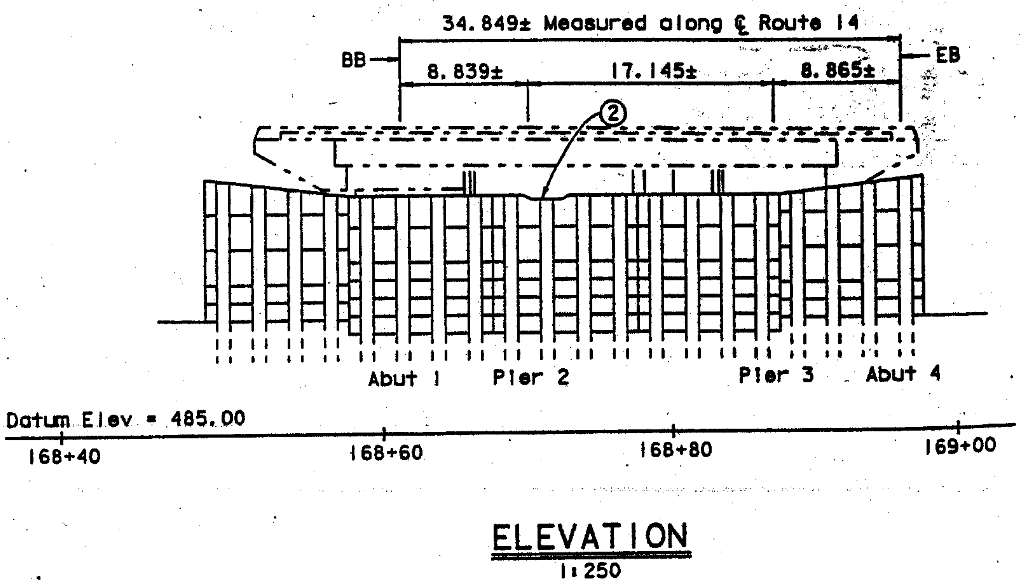


Figure 2. Plan View of the Check Dam-Retaining Wall System



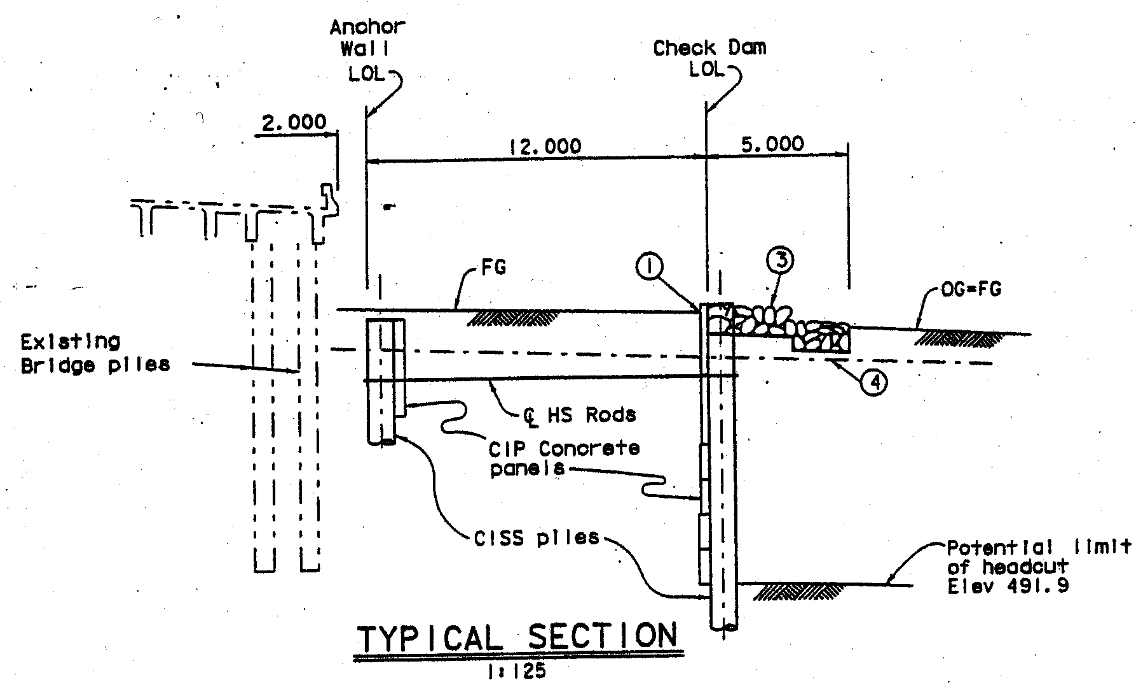




**Figure 3. Typical Section and Elevation View of the Check Dam-Retaining Wall System**

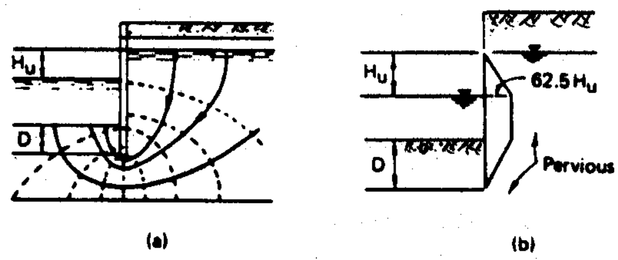



**Figure 4. Hydrostatic and Seepage Pressures (after Terzaghi , 1954)**

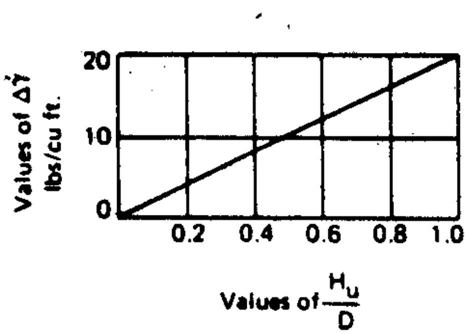



**Figure 5. Average Reduction in Effective Unit Weight of Passive Wedge due to Seepage Pressure Exerted by the Upward Flow of Water (after Terzaghi, 1954)**

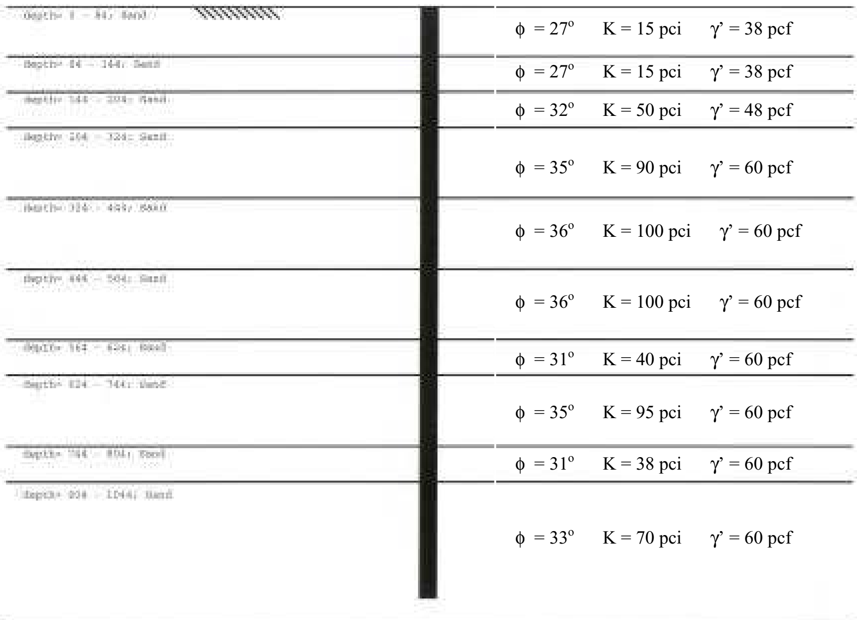



**Figure 6. Soil Profile A for Analysis of Anchor Piles**

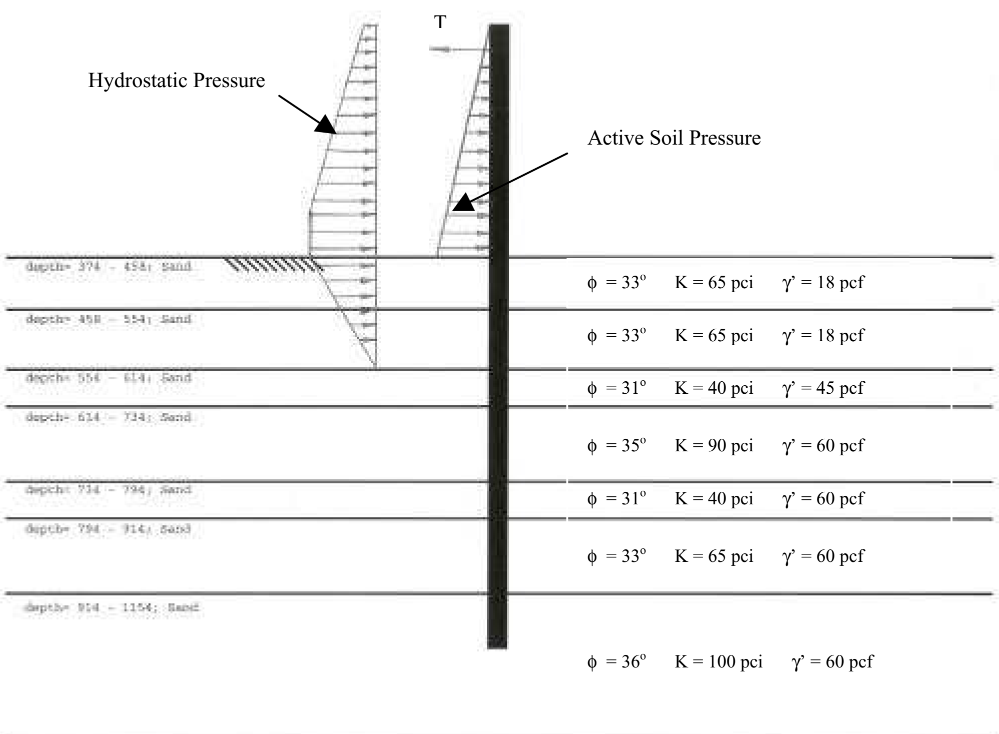



**Figure 7. Soil Profile B for Analysis of Soldier Piles**

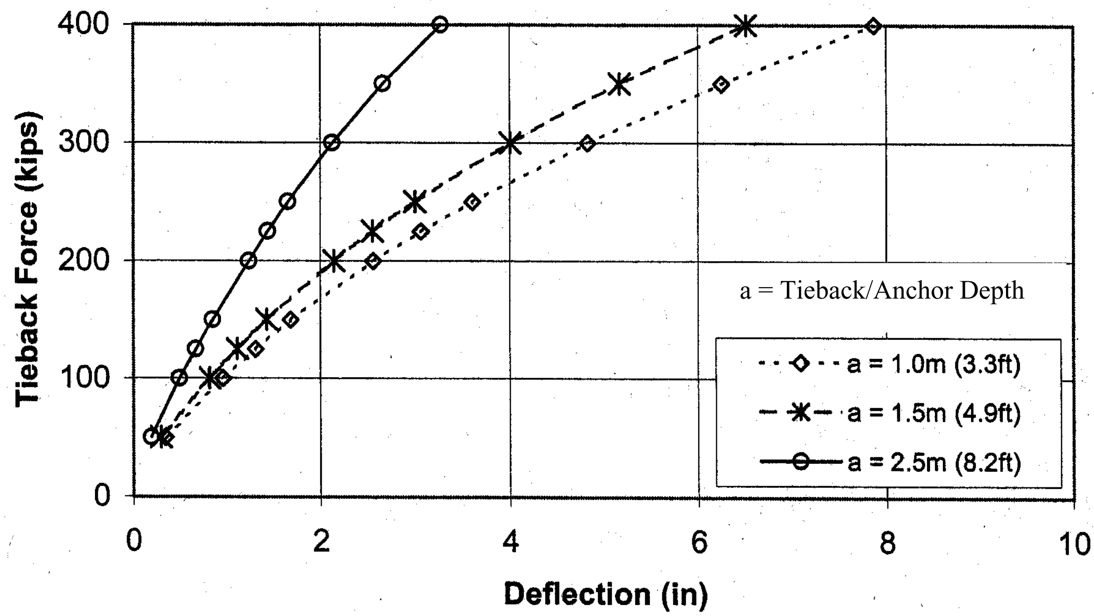



**Figure 8. Lateral Load – Deflection Curve for Anchor Pile**

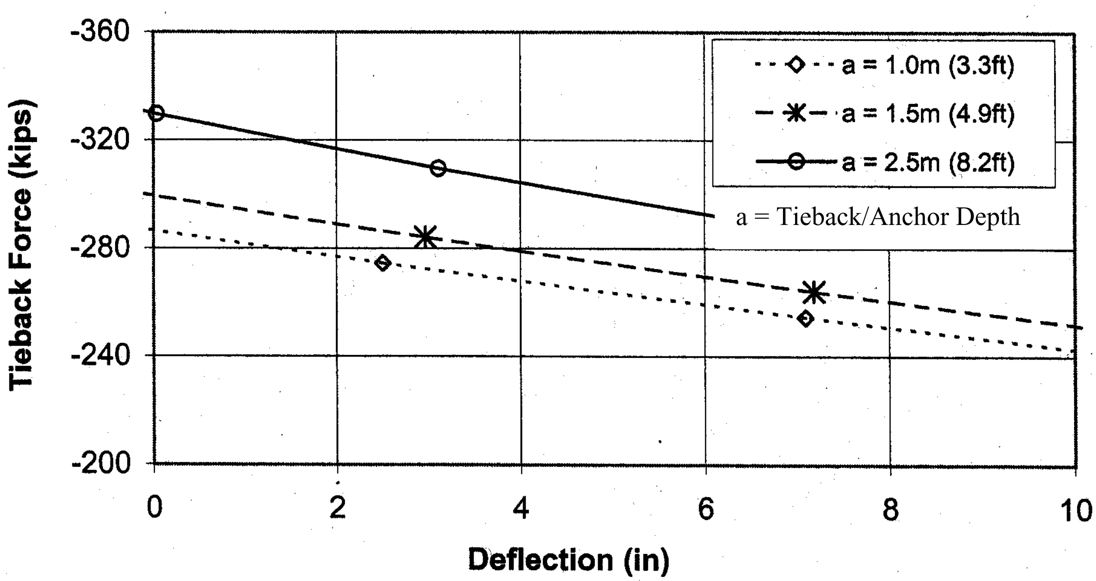



**Figure 9. Lateral Load – Deflection Curve for Soldier Pile**

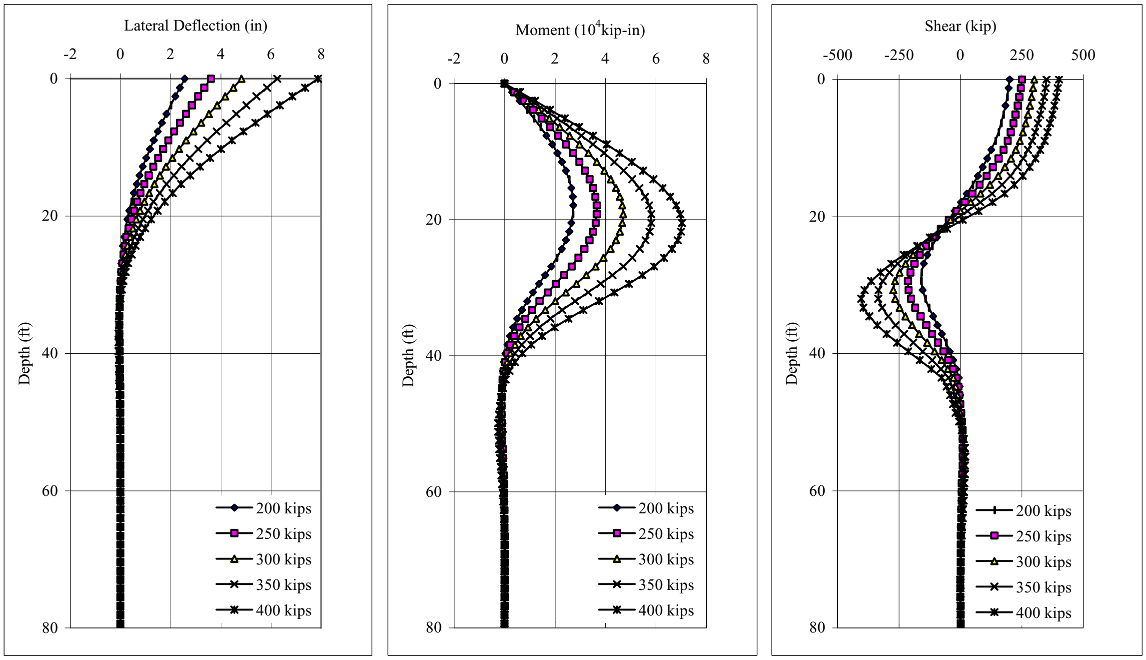



**Figure 10. Displacement, Moment, and Shear for Anchor Pile**

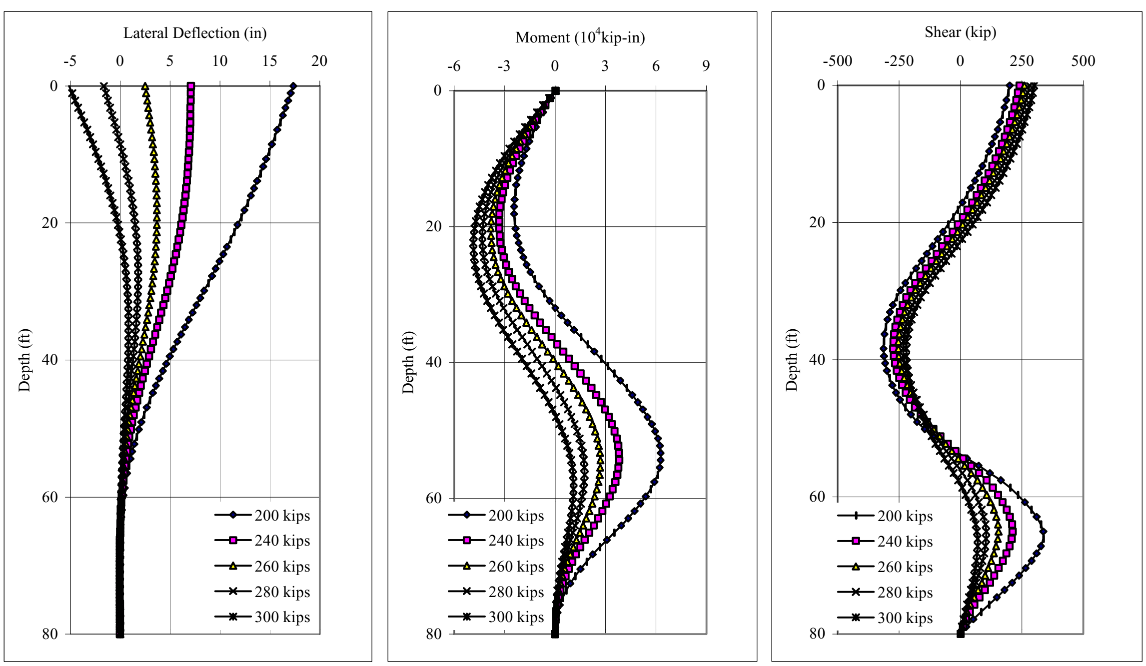



**Figure 11. Displacement, Moment, and Shear for Soldier Pile**