29 Mar 2001, 4:00 pm - 6:00 pm

Evaluation of a Soldier Pile-Tieback Wall at Carquinez Bridge

Mahmood Momenzadeh
Caltrans, Oakland, CA

Kenneth Jackura
Caltrans, Sacramento, CA

EVALUATION OF A SOLDIER PILE-TIEBACK WALL AT CARQUINEZ BRIDGE

Mahmood Momenzadeh, Ph.D., PE
Caltrans, Office of Roadway Geotechnical
Engineering, Oakland, California

Kenneth Jackura, PE, GE, Supervising Engineer
Caltrans, Office of Roadway Geotechnical Engineering,
5900 Folsom Blvd. Sacramento, California

ABSTRACT

Soldier pile tieback walls have been widely used by the California Department of Transportation (Caltrans) for stabilizing deep landslides and retaining cuts into unstable slopes under static earth forces for several decades now. However, the seismic load-deformation behavior of this wall type is not well known even though several Caltrans designed walls have undergone moderate seismic shaking without notable signs of distress. This paper introduces a reasonably simple load-deformation process used in analyzing a proposed 20.9 m high temporary tieback wall (to be reduced to an 18 m permanent height) located approximately 32 kilometers north of the Oakland Bay Bridge in California. The design procedure incorporates slope stability and soil-pile-tendon interaction processes incorporating public domain programs. The results demonstrate that application of the seismic earth pressure increment mobilizes the reserve load bearing capacity of the wall system resulting in quantifiable wall deformations. The process indicates that application of normal design procedures can result in a wall system quite tolerable of significant design seismic events prior to tendon or pile yield due to two sources of the reserve capacity normally existing in the wall system. One is due to tendon stretch from lock-off to tendon yield, and secondarily to pile yield capacity provided the piles extend below the slide surface. These factors result in a stable, flexible wall system able to accommodate substantially increased loads with acceptable deformations as indicated by the soil-pile-tendon interaction process.

INTRODUCTION

The subject site consists of multiple benches and slopes previously developed for residential use by placement of up to a 21 m thick uncontrolled fill over unprepared hillside. The fill is underlain by a stiff to very stiff colluvium of up to 4 m in thickness, which in turn is underlain by shale/claystone with sandstone interbeds. The upper part of the bedrock is extremely weathered, weak, and fractured and contains soil-like zones. In addition, the site is located in the vicinity of the active Franklin faults within a very active seismic region. Although there is evidence of site instability within the fill, the results of extensive site monitoring by slope inclinometers demonstrated no movement in the rock. The proposed tieback wall will be located between Stations 21+75 and 22+58 and provide static and seismic stability support of the slope on the upper side of the proposed elevated structures traversing this site.

The maximum wall height for the short term static loaded construction stage during which structure footings will be constructed is 20.9 m, while the maximum long term wall height (designed for the seismic condition subsequent to the footings being backfilled to Elevation 20 m) will be 18 m. Figure 1 illustrates the inferred soil and rock strata and associated static and seismic design lateral earth pressures for specific stations along the wall layout line. Only the evaluation conducted for the wall at station 22+20 is described in this paper.

SEISMIC PARAMETERS

Probabilistic seismic analyses were performed using a version of FRISK program modified by the first author to determine the design seismic input parameters. The San Andreas, Hayward, Rodger Creek, Green Valley, Geenville, Concord, Calaveras, Franklin, West Napa, and CRCV faults were included in the seismic risk assessment. The peak bedrock acceleration (PBA) vs. the earthquake returns periods were computed for several recent attenuation relationships reported in Seismological Research Letters (January 1997) as shown in Figure 2. A design horizontal acceleration of 0.35g was used to determine the seismic earth pressure increment. This was based on a PBA of 0.63g estimated for the 475 year earthquake at the site (solid line on Figure 2); a computed average seismic response amplification ratio of one; an allowance for an outward wall displacement of 150 mm based on the method described by Martin, G.R. (1993).

PROCEDURE

The wall analysis procedure included two stages. In the first stage, only wall stability (Factor of Safety, FS) was assessed for 3 earth pressure conditions, Cases a, b, and c using program SNAILZ. The purpose of this stage was to determine the required number of tendons and design lock-off loads (60% of tendon yield) to maintain a reasonable FS for the 20.9 m wall against the short-term static construction condition (Case a). FS was again checked at this same wall height at 90% of tendon yield (Case b). Finally, the long term seismic load condition for the permanent wall height of 18 m was checked (Case c). The second stage (discussed below) is the governing stage and uses the soil-pile tendon interaction process to assure that the combined static and seismic earth pressures do not exceed 90 percent of any individual anchor yield condition.

The strength parameters assigned to each of the inferred soil and rock strata are shown on Figure 3. These parameters were back calculated assuming that the site is in a slow but progressive failure mode (FS < 1.0). The static earth pressures shown on Figure 1 are based on a procedure recommended in a 1990 Caltrans Memo 5-12 to Designer for static conditions. The earth pressure calculated for short-term condition was reduced by a factor of 0.85. The seismic load increments shown in Figure 1 are based on the Mononobe-Okabe relationship using a horizontal acceleration coefficient of 0.35.

For the second stage evaluation, all 3 load case conditions incorporated 6 rows of tendons as determined from stage one, each with a 1380 KN yield. All tendons are considered bonded into the reasonably competent rock formation below the highly weathered rock. This provided the basis for computing the tendon 'free length' necessary for the analysis. For Case a, the temporary wall system was evaluated at a tendon lock-off load of 825 KN (60% of yield). For Case b (the temporary wall) static earth pressures were increased until one of the tendons reached 90% of yield, while for Case c (the permanent wall) the full seismic design load was applied. Subsequent to conducting the stage two, tendon loads were adjusted in the final stability calculation for stage one since all tendons will not reach capacity simultaneously.

All cases involved evaluation of the actual stresses and deformation in the tendons and pile by the soil-pile-tendon interaction process using program MBC76P. Elements of the modeled wall are shown in Figure 4. A W27X94 steel wide flange beam inserted into a 1 m diameter PCC filled hole below the dredge line (weak soil-cement above) was used for the soldier pile. Beam rigidity ignored the PCC contribution. Bi-linear spring constants for the tendons were developed assuming 1% strain to yield. The total elongation for each anchor was calculated based on the anchor free length and the assumed strain from lock-off to full yield. P-Ys for the soil/rock layers behind and in front of the pile were developed based on recent published data and/or program COMP624. P-Y's were based on the developed hole diameter of 1 m and not pile c-c spacing for portions above the original dredge line.

RESULTS

Results of the analyses in the Stage 1 are shown in Figure 3. As shown, the calculated minimum Factor of Safety was 1.12 for Case a, 1.74 for Case b, and 1.04 for Case c. These stability numbers were considered acceptable and do not include the additional resistance from the soldier piles developed along the inferred failure plane. Figure 3 also indicates that the critical failure surface under the seismic load condition is wider and deeper than that generated in the static condition. The calculated anchor load and wall deflection vs. wall elevation are shown in Figures 5 and 6, respectively. Results of the analyses performed using a non-public program are also shown in these figures for the comparison. Data in Figure 6 indicate total wall deflections of 72 mm, 104 mm, and 150 mm computed for Cases a, b, and c using program MBC76P. The data indicate that for Case c (seismic load condition) the total wall deflection increased by 78 mm and 46 mm from static Cases a and b, respectively. Comparison of Figures 5 and 6 indicate that the wall deflection characteristics are governed by distribution of the loads in the tendons. For static conditions, the maximum tendon loads were developed at the 2nd tie from the base of the wall where the wall deflection was more than half of the total wall deflection. On the other hand, for the seismic load condition the maximum tendon load was developed at the top tie contributing to almost half of the total wall deflection. Also, the tendon loads for both static and seismic load conditions were below or near the design loads.

WALL PERFORMANCE

Additional analyses were performed in order to verify the previously applied (recommended) earth pressures and the wall performance results. The tendons P-Ys were removed from the model shown in Figure 4 and the tendon loads calculated previously were applied into the wall and wall-soil/rock pressure reactions were calculated for Cases 3-a and 3-b. As shown in Figure 7, the predicted and the recommended pressures are almost the same with the exception of their upper and lower parts. Subsequently, the analyses performed in the 2nd stage were repeated for these cases using the predicted earth pressures. The calculated loads in tendons and the wall deflections are shown in Figures 8 and 9 respectively. The previous results are also shown in these figures for comparison. The results indicate that the deflection and load in the anchor at the top of the wall are about 10 percent larger than those computed using the recommended pressures. As shown in Figure 10, the moment distribution in the pile for both cases of the static and seismic loading conditions are below the yield moment of the pile.

CONCLUSIONS

1) The procedure used is capable of evaluating both stability and deformation behavior of the soldier pile-tieback wall,

2) The procedure provides a realistic estimate of loads in tendons and deformation, shear, and moment distributions in piles based on the coupled tieback-pile reactions using non-linear tieback and soil-load-deformation behavior, and

3) Normal design methods can result in a wall system quite tolerable of significant design seismic events prior to tendon or pile yield due to tendon stretch from lock-off to tendon yield and to pile yield capacity provided the piles extend below the slide surface.

REFERENCES

Bridge Memo to Designers Manual [1999] by Caltrans.

FRISK98, [1998]. Modified Version of FRISK by USGS in 1977 for Seismic Risk Evaluation. Momenzadeh, M., Caltrans.

Martin, G.R. [1993]. Seismic Design of Retaining Structures Seismic Soil/Structure Interaction Seminar, Vancouver, B.C..

MBC76P [1994]. A Modified Version of BMCOL76 by Dr. R. Glauz @ U.C. Davis in consultation with K.A. Jackura.

Seismological Research Letters [1 &2/1997]. Volume 68, Number 1, Published by Seismological Society of America.

SNAILZ–V3.09 [1999]. A Limit-Equilibrium Program for Reinforced & Unreinforced Walls by Nguyen C. and Jackura, K.

	Short-Term Wall Height (H_T)	Long-Term Wall Height (H)
	Pressure Diagram A	Pressure Diagram B
Station "G" Line (m)	H_T	H
21+00	14.5-m	11.5-m
21+30	15.5-m	12.5-m
21+60	17.0-m	14.0-m
21+90	20.0-m	17.0-m
22+20	21.0-m	18.0-m

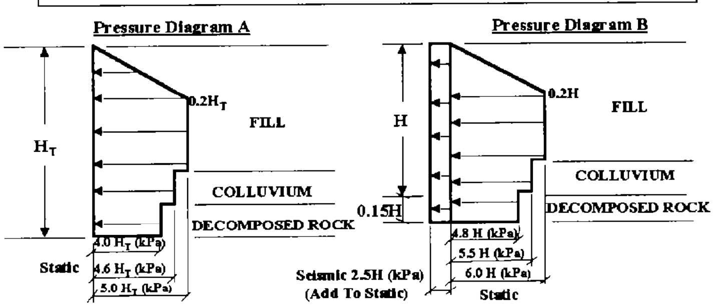

Figure 1. Wall Height and Recommended Pressure Diagrams

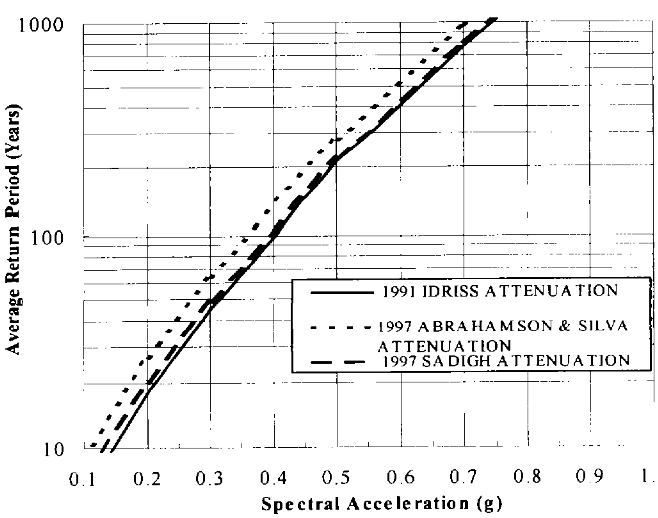

Figure 2. PBA vs. Earthquake Event Return Period

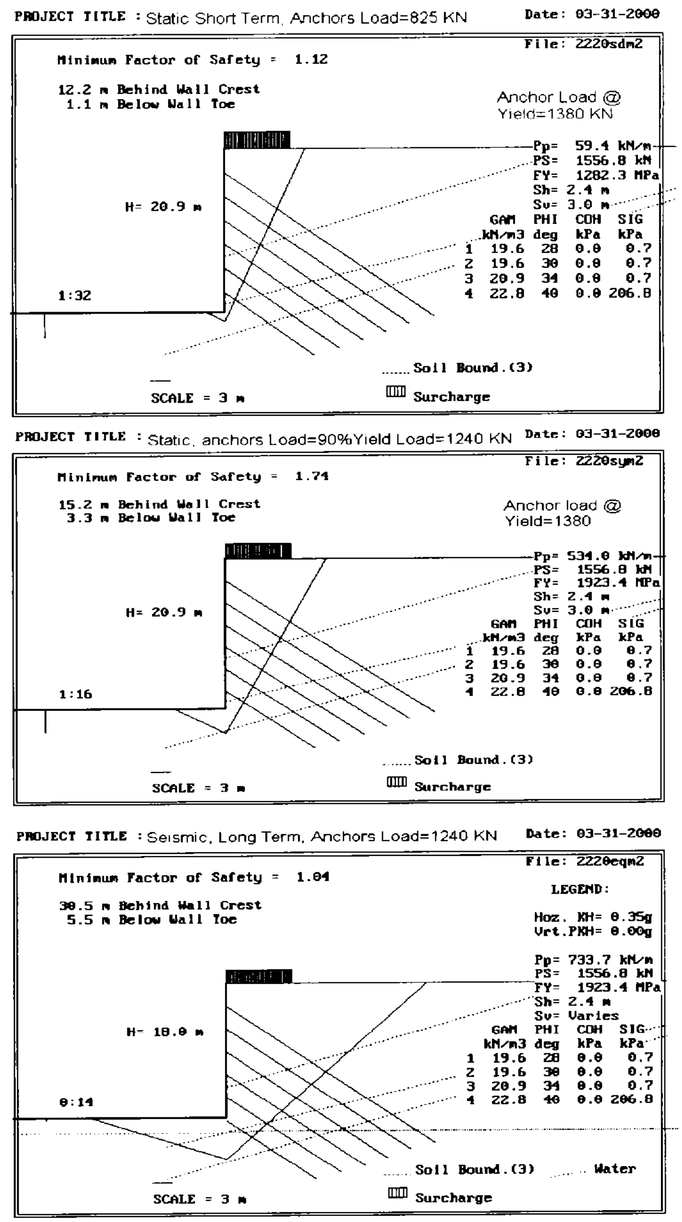

Figure 3. Results of Stability Analyses for Cases a, b, & c

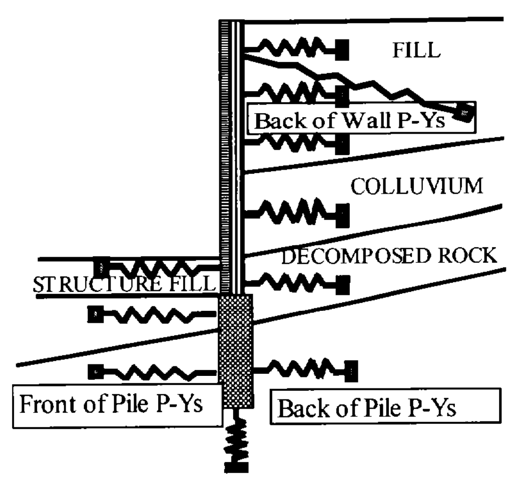

Figure 4. Model Used in Soil/rock-Wall Interaction Analysis

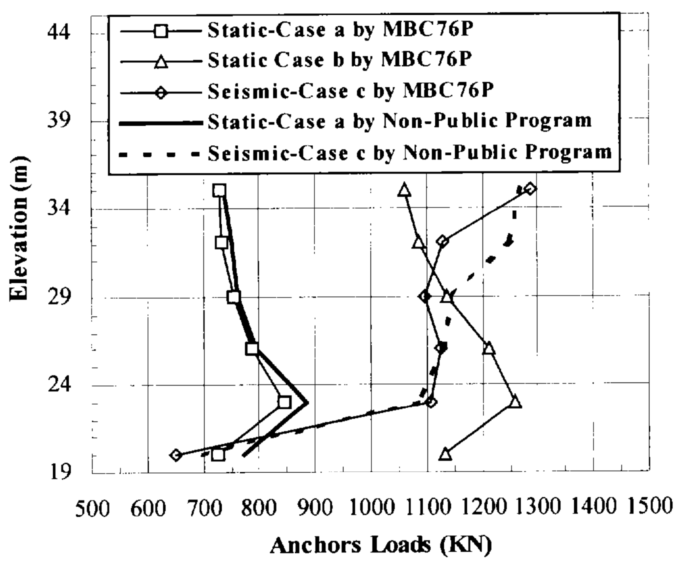

Figure 5. Anchor Loads for Cases a, b, and c

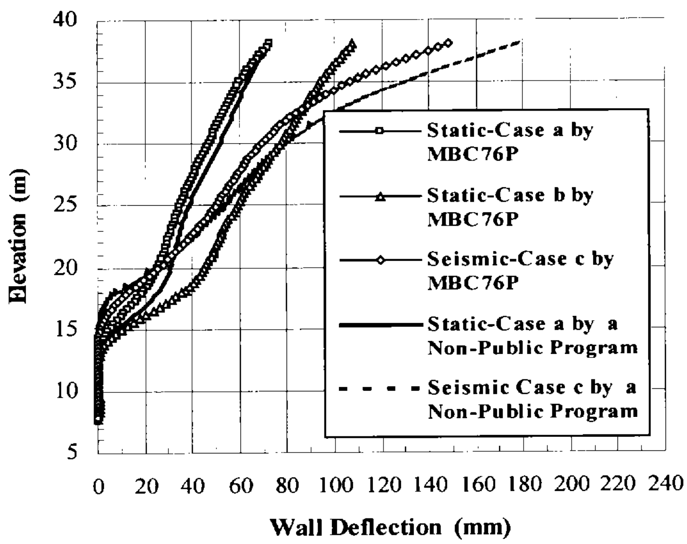

Figure 6. Wall Deflections for Cases a, b, and c

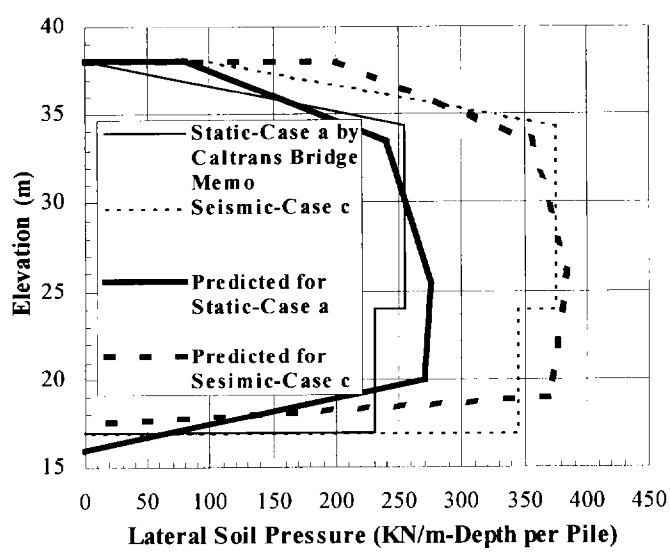

Figure 7. Comparison of Recommended & Predicted Pressures

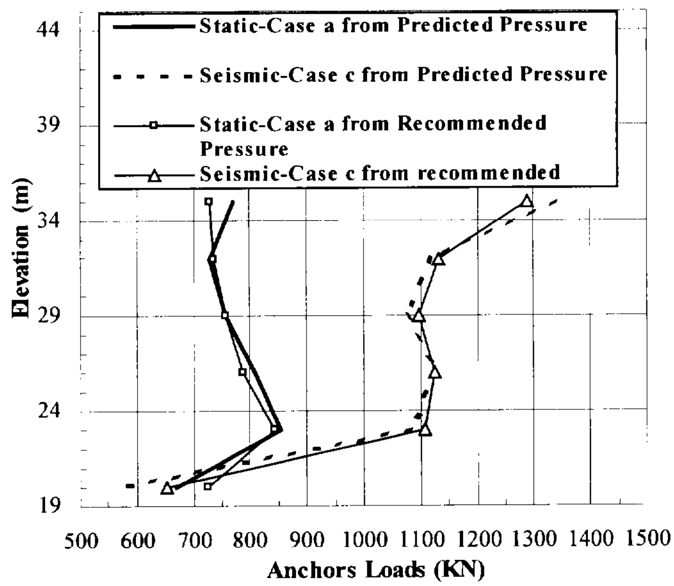

Figure 8. Comparison of Anchor Loads

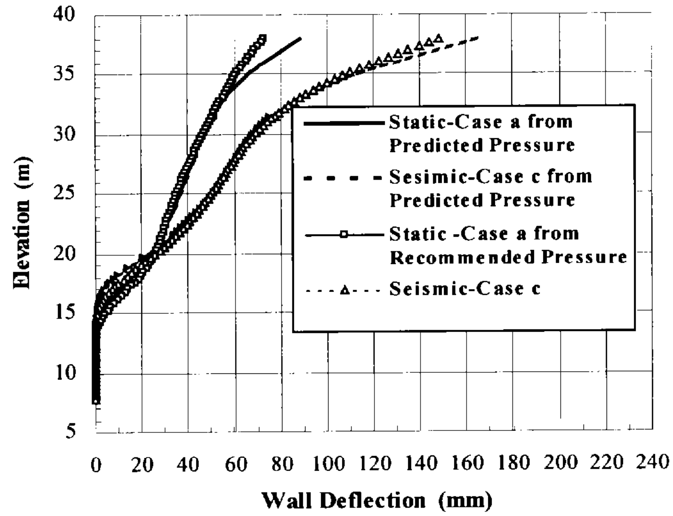

Figure 9. Comparison of Wall Deflections

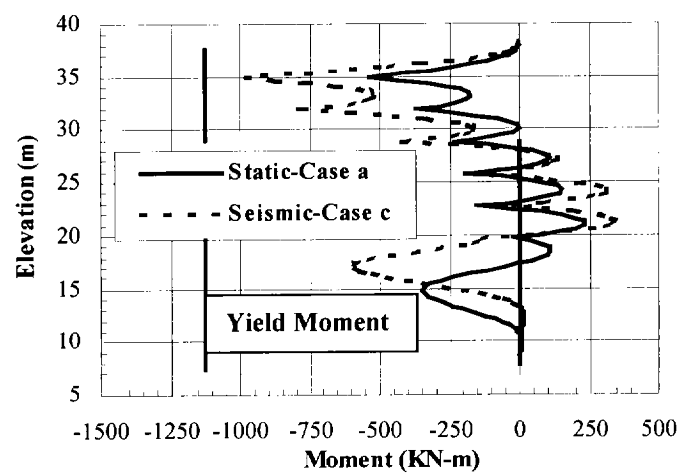

Figure 10. Moments in Pile from Predicted Pressures