16. Nalezny, C. L., and Li, M. C., "Effect of Soil Structure and Thixotropic Hardening on the Swell Behavior of Compacted Clay Soils," *Highway Research Record, No. 209,* (Highway Research Board), Washington, D.C., 1967.
17. Seed, H. B., and Chan, C. K., "Thixotropic Characteristics of Compacted Clays," *Journal of the Soil Foundations and Mechanics Division,* ASCE, Vol. 85, No. SM4, Proc. Paper 1427, June, 1957, pp. 87–128.
18. Seed, H. B., Woodward, R. J., and Lundgren, R., "Prediction of Swelling Potential for Compacted Clays," *Journal of the Soil Mechanics and Foundations Division,* ASCE, Vol. 88, No. SM3, Proc. Paper 3169, June, 1962, pp. 87–128.
19. Smith, D. W., "Electro-chemical Bonding and Clay Compression," thesis presented to the Virginia Polytechnical Institute, at Blacksburg, Va., in 1965, in partial fulfillment of the requirements for the degree of Doctor of Philosophy.
20. Verwey, E. J. W., and Overbeel, J. Th.G., *Theory of the Stability of Lyophobic Colloids,* Elsevier Publishing Co., Amsterdam, 1948.
21. Warkentin, B. P., "Water Retention and Swelling Pressure of Clay Soils," *Canadian Journal of Soil Science,* Vol. 42, No. 1, 1962, pp. 189.

APPENDIX II.—NOTATION

The following symbols are used in this paper:

K_1, K_2, K_3 = expansion constants;
n_s = number of moles in solid phase;
n_w = number of moles in water phase;
$\underline{P}_s$ = swelling pressure for no volume change;
$\underline{S}_s$ = entropy of solids;
S_w = entropy of water;
u_0 = initial suction;
V_0 = initial volume;
ΔV = volume change, for no load; and
Ψ = swelling potential.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

POTRERO HILL SLIDE AND CORRECTION

By Travis Smith,[1] F. ASCE and Raymond Forsyth,[2] M. ASCE

INTRODUCTION

Southern Pacific Tunnel No. 1.—Southern Pacific Tunnel No. 1 near the San Francisco Bay coastline, San Francisco, was constructed in 1906 or 1907. As shown in Fig. 1, it consists of unreinforced concrete walls 25 in. ± in thickness to a height of 10 ft supporting a 15-ft radius arch consisting of six courses of mortared brick which was later gunited for protection from reactive fumes. The tunnel is a 1,830-ft long 2-track facility 30 ft wide at the base. The floor consists of unreinforced concrete 1 ft to 2 ft in thickness. Because all Southern Pacific Railroad train movements into San Francisco must go through it (140 per day) this facility is a vitally important San Francisco Bay Area Transportation link.

In 1928, as a result of evidence of tunnel movements, copper pins were installed at 50-ft stations to measure changes in distance across the tunnel with time. At that time the minimum width (29.325 ft) was found to be located 580 ft from the northern portal. Measurements continued until 1935 when movement had apparently ceased. During this 7-yr period, a maximum movement of 3/4 in. was reported.

Highway Design Considerations.—In August, 1960, a Project Report proposed a new highway facility to provide immediate relief for the James Lick Freeway (Route 101). In the report, three alternate routes were evaluated. As shown in Fig. 2, the easterly two lines (L and T) were projected through a heavily industrialized area adjacent to San Francisco Bay. The third route (Alternate P) was approximately 1/2 mile westerly lying at the dividing line of the residential and industrial section of this part of San Francisco and traversing

Note.—Discussion open until August 1, 1971. To extend the closing date one month, a written request must be filed with the Executive Director, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 97, No. SM3, March, 1971. Manuscript was submitted for review for possible publication on July 6, 1970.

[1] Asst. Materials and Research Engr., California Div. of Highways, Materials and Research Dept., Sacramento, Cal.

[2] Sen. Materials and Research Engr., California Div. of Highways, Materials and Research Dept., Sacramento, Cal.

the Potrero Hill area. Of primary concern in the route selection was removal of irreplaceable land from private use. Every effort was made to insure that the various alternatives required the least amount of property possible consistent with design standards. The latter alternate was recommended for the following two reasons: (1) It presented the greatest traffic service in terms of user benefits; and (2) it acted as a buffer between a residential and industrial area.

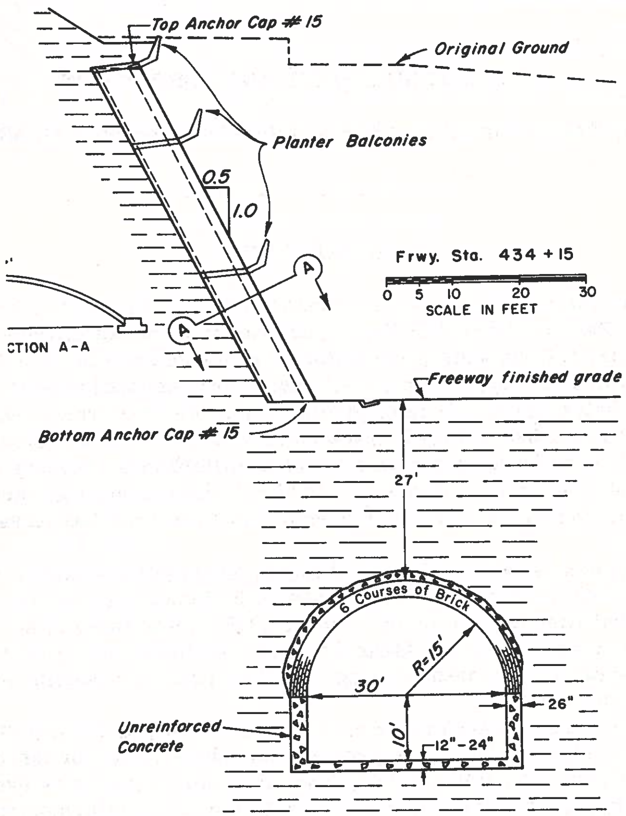

FIG. 1.—TUNNEL AND RETAINING WALL DETAILS

The sidehill cut through Potrero Hill area would provide a depressed freeway with reference to the residential properties to the west.

Retaining Wall Design.—The foundation investigation of the site of the proposed retaining wall showed interbedded fractured and sheared sandstone and shale of the Franciscan formation. This study consisted of six rotary sample borings, eight cone penetrometer tests and a number of 1-in. sample borings for identification purposes. The report recommended a cantilever or a counterfort retaining wall.

A retaining wall design for this particular location was governed by three primary considerations. These were: (1) Esthetics; (2) the possibility of overloading the railroad tunnel; and (3) loss of property. It was decided that the retaining wall should be of the rock bolted type for the following reasons: (1) The structure would be less massive than the conventional counterfort or cantilever walls, and thus more esthetically pleasing; and (2) as the load would be carried primarily by the hill mass, large footings would not be necessary. This would minimize loss of property and eliminate the necessity of applying a large vertical load component near the tunnel.

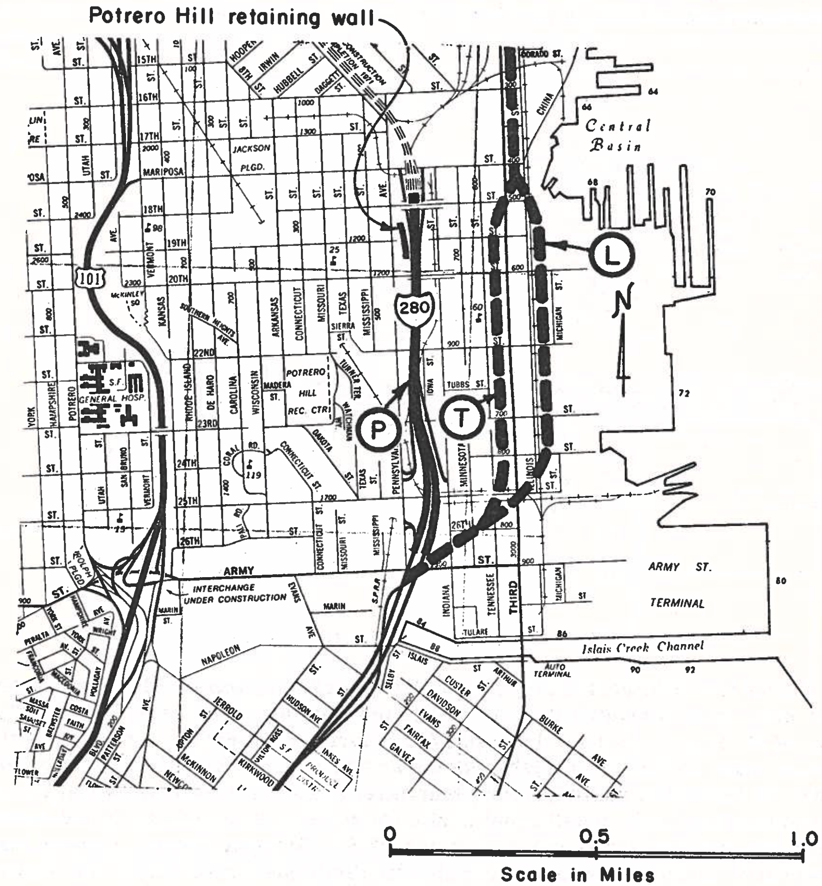

FIG. 2.—ALTERNATE ROUTE LOCATIONS

As finally designed, the retaining wall was 800 ft long and 20 ft to 60 ft high, with 1/2:1 batter. Its alignment was slightly skewed to the tunnel. Anchor caps 4 ft in width were spaced at 40-ft intervals along the walls. Anchor rods spaced vertically along each anchor cap varied in length from 30 ft at the bottom to 45 ft at the top. Between anchor caps, the wall consists of a concrete arch slab having a horizontal radius of 27 ft and a mid ordinate of approximately 7 ft (see Fig. 1).

EARLY DISTRESS

Cut excavation in the retaining wall area was begun in August, 1966 and was virtually complete in mid-September, 1967, when signs of tunnel movement were noted. At this time, the tracks in the tunnel had shifted as much as 1/2 in. and an old crack in the tunnel appeared to be widening.

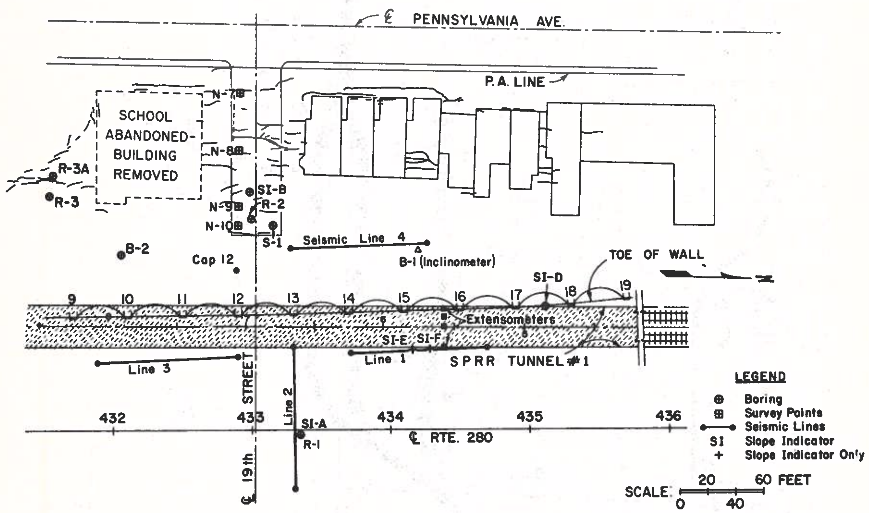

FIG. 3.—PLAN—POTRERO HILL AREA

During the latter part of September, severe cracking was noted in the Potrero Hill area from Pennsylvania Avenue to the retaining wall (see Fig. 3). The rear of St. Theresa School dropped 1/10 ft during this period. Stressing of the anchor caps continued through September and was completed by October 23, 1967. On November 4, 1967, Burton Marliave, who was retained by the California Division of Highways, noted a continuation of movement both easterly and downward and recommended the evacuation of St. Theresa School. Lateral and downward movement continued at a slightly diminished rate during the months of November and December so that extensive utility and foundation repair were required. By December 22, it had been necessary to place jacks under five houses to preclude extensive damage as result of movement. It was also necessary to repair a number of utility house connections, window sills, and repair several broken sidewalks. By November 24, 1967, the school had been evacuated. On December 18, 1967, an earthquake registering 5.3 on the Richter scale was recorded in the area with no apparent effect on the school or the structures in the immediate vicinity. By December 26, most of the emergency

repair work on the houses was complete. Movement in the Potrero Hill area continued, though at a diminished rate, until May, 1968.

INSTRUMENTATION, EXPLORATION AND TESTING

Instrumentation.—On September 15, 1967, a decision was made by the Division of Highways to provide the equipment and personnel to gather the information necessary for an evaluation of the Potrero Hill problem. Reference points had been established on the retaining wall for measurement of changes in alignment and elevation. A survey grid was established by highway personnel in the streets behind the retaining wall. During this period, the Southern Pacific Railroad established a horizontal offset line within the tunnel along the

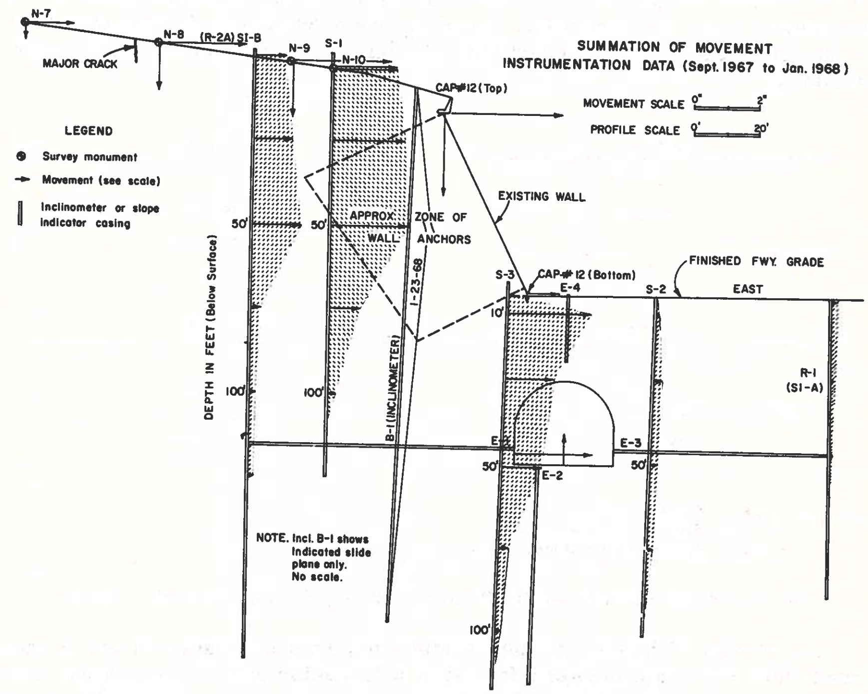

FIG. 4.—CROSS SECTION—HIGHWAY STATION 433±, RAILROAD STATION 690±

right curb. Elevations were taken along the center drain at both the right and left curbs. The firm of Shannon and Wilson, Inc., was retained by the Southern Pacific Railroad as consultants. Their investigation included extensometer and slope indicator installations, one boring and several types of geophysical measurement.

In November, 1967, five holes were drilled by highway personnel to obtain cores for classification and strength testing and for the installation of inclinometer and slope indicator casings. The borings varied in depth from 80 ft to

200 ft. In addition, rock noise and seismic measurements were made in the hill area. A layout and cross section of the tunnel, retaining wall, and surrounding area showing instrumentation is presented in Figs. 3 and 4.

EXPLORATION

Seismic Refraction Survey.—Several seismic refraction lines were run in the vicinity of the Potrero Hill retaining wall on November 15, 1967, to determine the velocities of the material and, if possible, to determine the depth to bedrock. Their locations are shown in Fig. 3.

These data were obtained with a 12-channel seismic instrument using 10-lb sledge hammer blows as the energy source. As shown in Fig. 5, a straight line interpretation of the data was made. Considerable difficulty was encountered due to electro-magnetic radiation and heavy train and vehicular traffic in the immediate area. Interpretation, therefore, required a considerable degree of judgment because of the questionable quality of much of the information obtained.

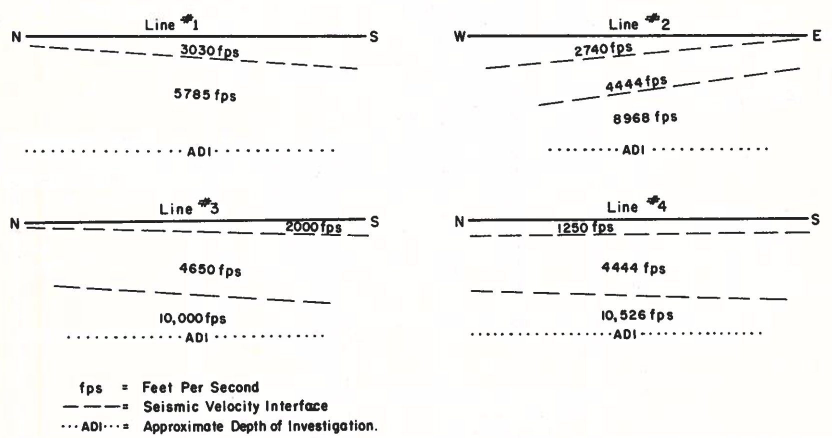

FIG. 5.—POTRERO HILL—SEISMIC LINE PROFILES—SCALE 1 IN. = 50 FT

The results of this study show surface material of weathered and broken rock which has a thickness of 5 ft to 20 ft with a seismic velocity ranging from 3,500 fps to 4,400 fps. The underlying velocity zone ranges from 6,900 fps to 9,750 fps. These velocities probably represent hard sandstone boulders or lenses.

Coring.—Between November 11 and December 14, 1967, seven borings were made by the Division of Highways and one by Shannon and Wilson, Inc., at the locations shown in Fig. 3.

Drive sampling employing the California sampler was attempted with little success due to hard rock fragments scattered throughout the material. Most of the coring was done using 2-in. or 3-in. diamond core barrels.

In Boring R-1, in the median area, new Station 433+35, 18 core samples were taken with an average recovery of 76 %. The material was soft, highly sheared

gray shale with many fragments, lenses, and thin beds of hard dark gray shale and hard, light gray, fine grained sandstone. This pattern was repeated with

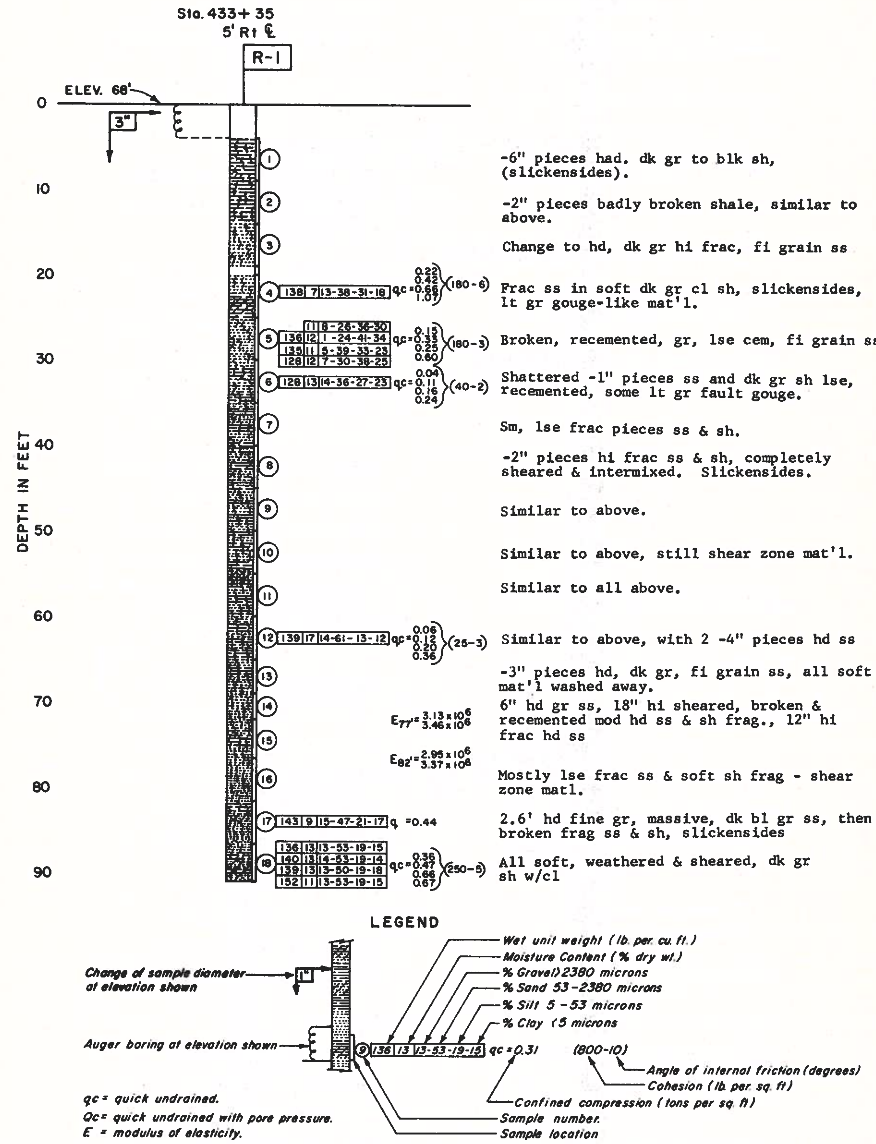

FIG. 6.—LOG OF BORING R-1

little variation at Borings R-2, R-2A, R-3 and R-3A.

Boring B-1 above the wall, 10 ft north and 20 ft west of the centerline of

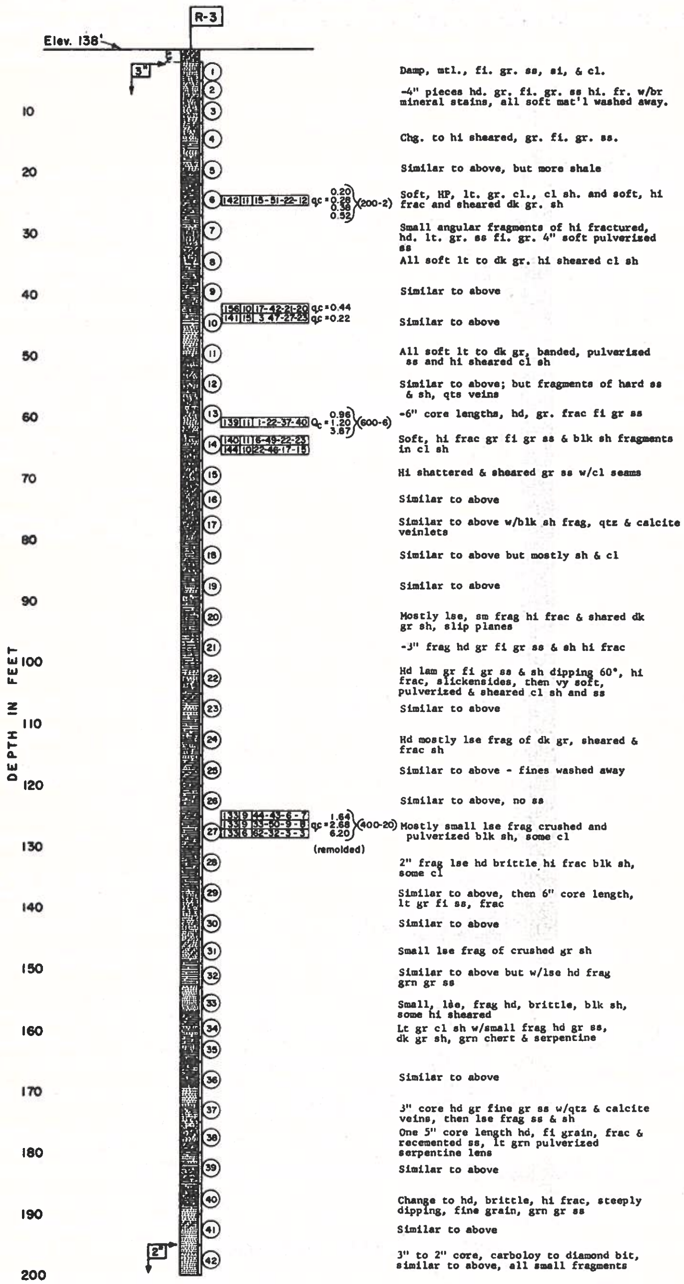

FIG. 7.—LOGS OF BORINGS R-2 AND R-2-A

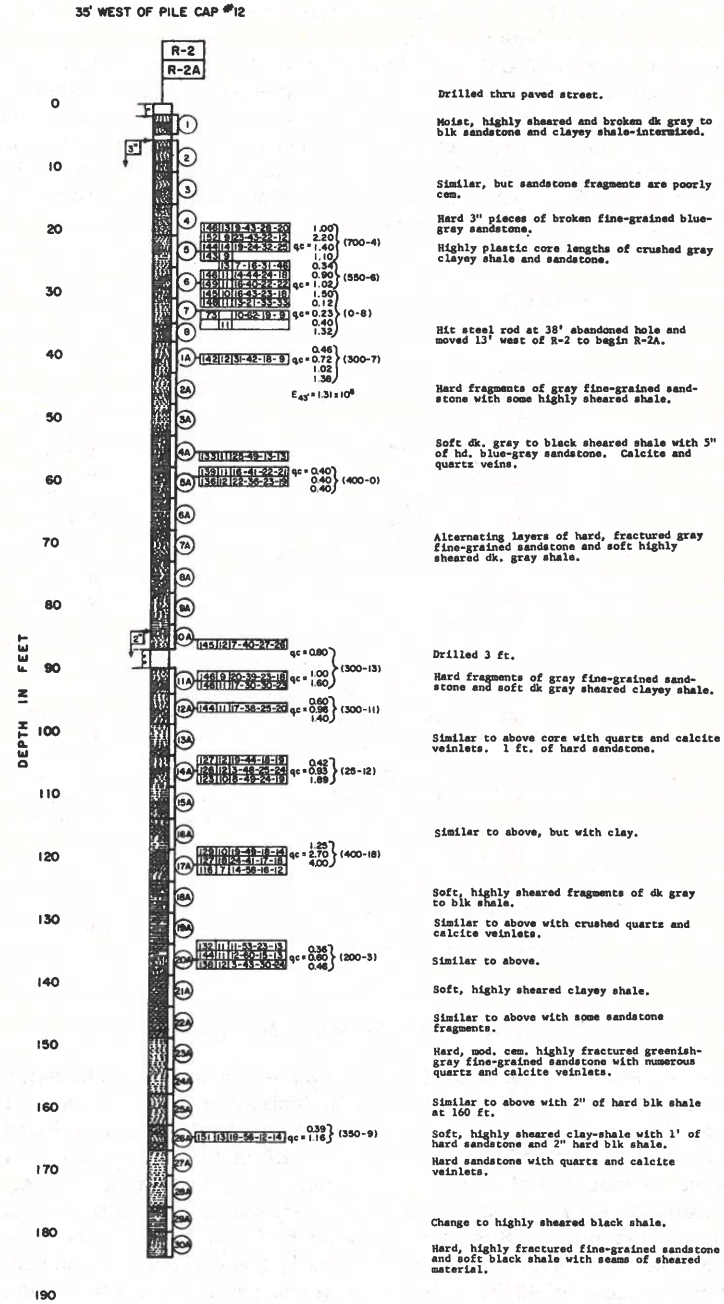

FIG. 8.—LOG OF BORING R-3

Cap 15, was cored to 180 ft. The hole was then reamed to 4-1/2 in. for installation of 154 ft of inclinometer tubing.

As shown in Figs. 6, 7 and 8, the borings revealed a wide variation in quality with the predominant materials being weathered to hard sandstones and clay shales. Examination of boring logs and the materials encountered, revealed no clearly definable pattern with respect to location or depth, although there was tendency for more competent materials at greater depths. The average quality of material encountered in all borings might be defined as a fine fractured sandstone or shale intermixed with clay.

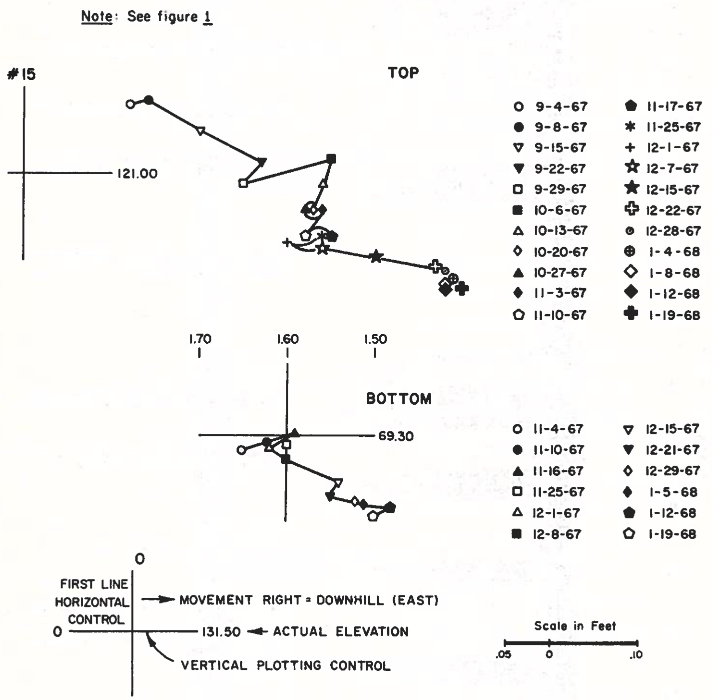

FIG. 9.—ANCHOR CAP NO. 15 MOVEMENT

Movement Patterns Prior to Construction.—The survey grid established subsequent to the beginning of movement in September, 1967, is shown in Fig. 3. The PA line down the east edge of Pennsylvania Avenue was found to be virtually out of the area of influence. The profile of 19th Street, shown in Fig. 4, revealed movements of a much larger and more consistent nature. From Pennsylvania Street west both lateral and vertical movements were of relatively minor magnitude. However, between N-7 and N-8, a very significant change in magnitude of movement both laterally and vertically occurred within a horizontal distance of 43 ft. This was in keeping with the visible surface distress shown in Fig. 3 which shows a large and continuous transverse crack all the way across 19th Street between grid points N-7 and N-8. It is probable that the westerly limits of the slide activity or the westerly scarp can be defined by this single large crack.

Horizontal and vertical movement data resulting from survey measurements on the tops and bottoms of the retaining wall anchor caps indicated that movement was largely confined to the zone between anchor caps No. 5 and 18. The lateral and vertical movement patterns of one of the most active caps (No. 15) is shown in Fig. 9. Here as was the case at all other anchor caps, there was a much greater magnitude of movement at the top of the anchor cap. Movements at the bottom of the anchor cap were found to be similar in direction but of much smaller magnitude, indicating the certain amount of distortion of the individual anchor caps and probably the wall as the result of the slide activity. Fig. 9 also reveals an interesting trend of relatively uniform eastward and downward movement until October 5, after which movement was almost entirely downward until December 7. This may be the result of the anchor tensioning which occurred until mid-October and could have temporarily inhibited lateral movement.

Tunnel Measurements.—To detect movement in the immediate vicinity of the tunnel with a higher degree of accuracy than would be possible by conventional surveying methods, four extensometers (E1-E4) were installed as shown in Fig. 3 in order to measure the amount, rate and location of deformation. These devices which were installed by the firm of Shannon and Wilson, Inc., have a sensitivity of ± 0.002 in.

A plot of measurement movement versus time for Extensometer E-1 through E-4 is presented in Fig. 10. At E-1 and E-2, almost all the measured movement occurring outside the 80-ft length of E-1 is indicated by the fact that the total amount of movement as of January 19 (0.5 in.) was substantially less than translation to the east of the west tunnel wall, as determined by internal tunnel measurements at Station 5+70 (1.6 in.). It is also entirely possible that the soil movements indicated by E-2 may be substantially less than indicated due to buckling of the tunnel floor slab. A negligible amount of movement was noted at E-3 indicating that the east tunnel wall remained stationary during the period of slide activity. As shown by Fig. 10, measurements of distance across the wall by the Southern Pacific Railroad Co. for the period in question, ranged from approximately 1 in. per month in the early fall to 5/8 in. per month from mid-December to mid-January. These data also indicated that almost all tunnel movement occurred between Railroad Stationing 4 and 7 and for the most part, between Stations 5 and 6. Maximum vertical movement within the tunnel occurred at survey Station 5+50.

Slope Indicators.—Slope indicators are precision instruments designed and manufactured by the Slope Indicator Company of Seattle, Washington, capable of measuring inclination from the vertical to approximately 3 min of arc. With this degree of sensitivity lateral movements of 1/16 in. can easily be detected. A total of seven of these devices were installed in support of the investigation, three of these by the firm of Shannon and Wilson, Inc., and the remaining four by the California Division of Highways. A plot of indicated lateral movements through mid-January for three of the five devices revealing consistent and significant lateral movements (easterly) is shown in Fig. 4. At location S-1, the slide plane area is relatively well defined. Here lateral movements ranging from 2 in. at ground surface to 2.5 in. at 45 ft were recorded as of December 14, 1967. Movement diminished relatively rapidly below 45 ft to approximately 1/2 in. at 80 ft. At slope indicator location S-3, the slide plane area was again

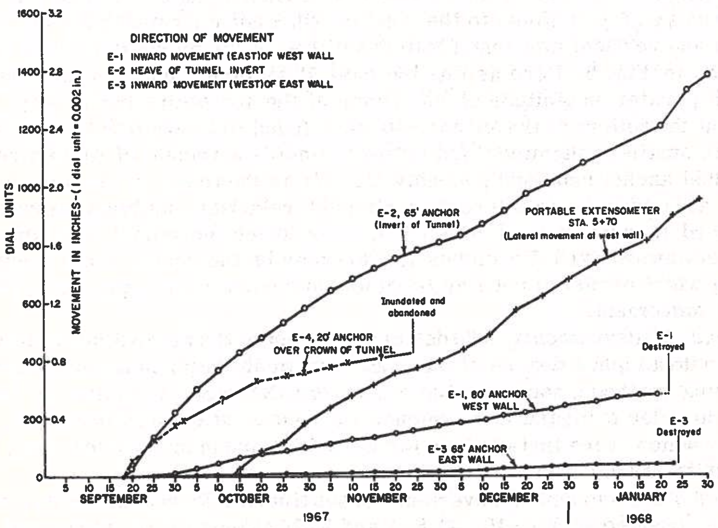

FIG. 10.—DIRECTION OF MOVEMENT VERSUS TIME

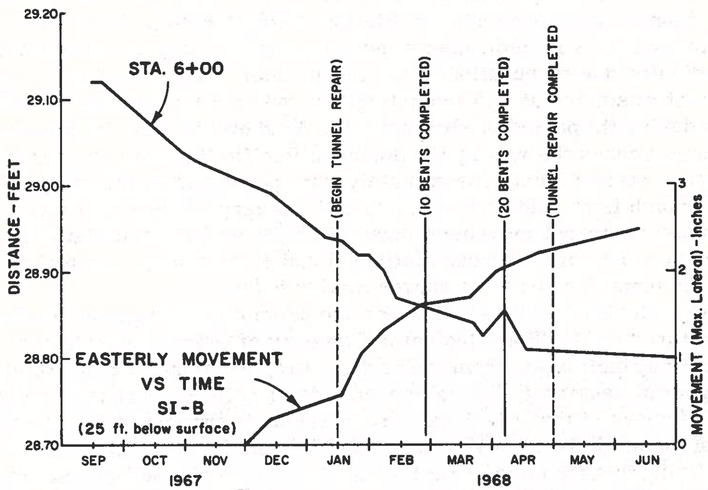

FIG. 11.—DISTANCE ACROSS WALL SP TUNNEL NO. 1

well defined with easterly movements of over 3 in. at the 5-ft level diminishing to 1-3/4 in. at the 45-ft level and then reducing abruptly to 1/2 in. at the 65-ft level. At SI-A, a relatively small easterly movement of 1/2 in. occurred at the surface which diminished to approximately 1/4 in. at the 30-ft level as of January 23, 1968. No movement was recorded below 50 ft. At slope indicator SI-B, a slide plane area is again fairly well defined with approximately 1 in. of movement from the surface to a depth of 40 ft diminishing to 1/2 in. at 50 ft and 1/4 in. at 130 ft as of January 23, 1968. Slope indicator location SI-C indicated inconsistent, i.e., westerly movements of very small magnitudes with a maximum of 1/4 in. At location SI-D, which would correspond in cross section approximately to location S-3, easterly movements of 3/4 in. from the surface to 40 ft were recorded. The magnitude of movement then reduced to 1/4 in. at 60 ft with no significant movement below the 75-ft level. Thus, the location of the slide plane zone at approximately a 40-ft depth is in general agreement with the movements shown for location S-3.

Inclinometers.—An inclinometer is read by lowering probes of varying lengths (3 ft, 1.0 ft and 0.5-ft lengths) into a cased boring. Inclinometer Boring B-1 was cased to a depth of 150 ft on November 28, 1967. Seven days after installation the 3-ft probe was stopped at a depth of 143 ft. Sixteen days after installation the 3-ft probe was stopped at a depth of 62 ft. Twenty-nine days after installation the 3-ft and 1.0-ft probes were stopped at a depth of 37 ft to 40 ft, which tends to corroborate the slope indicator data.

A graphic summarization of movement from September to mid-January as indicated by instrumentation and surveys of critical Station 5+70 (railroad) is presented in Fig. 4.

RESULTS OF PHYSICAL TESTS

The primary requirement for a valid stability analysis is a knowledge of the strength characteristics of the individual layers comprising the soil mass under consideration. If these strength characteristics can be defined by laboratory testing, and a representative soil profile developed, a rational stability analysis based on current soil mechanics, theory, and practice is possible. These conditions did not exist at Potrero Hill, however, due to its very complex soil profile. The materials encountered consisted of weathered, interbedded sandstones and shales with a clay matrix to reasonably competent rock. The presence of rocks throughout the soil profile made a continuous coring operation utilizing a Longyear 3-in. diamond core barrel necessary. A 2-in. Christiansen Diamond Core Barrel was used when harder rock formations were encountered. The samples available for test, therefore, were badly disturbed by the coring operation and the application of drilling water at high pressure. Examination of individual core samples did not reveal a well defined fracture plane, therefore residual strength direct shear tests on a consistent slickensided surface were not considered.

The initial testing program consisted of a series of unconsolidated, undrained triaxial tests (UU) on the partially saturated gray clay shale samples considered representative of the poorer quality materials encountered at three boring locations (R-1, R-2 and R-3). Total stress parameters were found to range from 0° to 18° in angle of internal friction with cohesion values from 25 psf to 700 psf. In a subsequent series of tests, selected samples were saturated under a vacuum for 24 hr after which UU tests with pore pressure measurements were made utilizing a Borden gage and mercury manometer arrangement. Under these conditions, ϕ' ranged from 6° to 18° and C' from 25 psf to 550 psf.

A final series of tests were conducted on remolded samples at field moisture content from Boring R-3. Based both on visual evaluation of all the samples recovered in the investigation, it is believed that these samples approximated an average condition of the earth mass under consideration. Here, UU tests on the partially saturated samples resulted in a total angle of friction (ϕ) of 20° and a cohesion value of 400 psf. Examination of shear strength versus displacement plots for all triaxial test series did not reveal a consistent and well-defined residual strength level. This is very probably due to the fact that the clay shales subject to test were not overconsolidated but rather residual or weathered in place.

ANALYSIS OF DATA

Even though the classical methods of stability analyses were not applicable to the Potrero Hill area, due to the extremely complex and heterogeneous nature of the soils, several types of analyses were made primarily for the purpose of comparing different repair proposals and to gain further insight into the probable mechanics of failure.

Coulomb (Wedge) Analysis.—This procedure assumes a planar failure surface. The forces considered were the weight of the soil mass in the wedge defined by the assumed failure plane, cohesion and friction acting along the failure plane, and a reaction acting at an angle of 90° + ϕ to the failure plane. The passive pressure mobilized by the existing soil wedge was assumed to act in a horizontal direction. A number of failure planes and assumed shear strength parameters were utilized to calculate a safety factor. Along the plane which most nearly conforms to the probable failure surface as indicated by instrumentation and surface cracking, a safety factor of unity was obtained utilizing a 10° angle of internal friction and a cohesion value of approximately 750 psf. As the assumed failure plane moved back or more deeply into the strength mass, the strength parameters required for a unity safety factor diminished.

Fig. 12 shows the results of an analysis along the most likely failure surface extending from the zone of the most severe transverse cracking to a point beneath the bottom of the tunnel. A water filled crack along the failure surface to a depth of 30 ft was assumed. Calculation of frictional resistance along the failure surface was based upon soil density in the bouyant state below the water table. The strength parameters utilized (ϕ = 20°, c = 400 psf) were derived from the aforementioned third series of triaxial tests (UU) on remolded samples of shale and sandstone with a clay matrix. Based on these conditions, the magnitude of passive pressure mobilized in order to resist sliding is shown acting on the easterly side of the tunnel, ranging from 2,500 psf at the top to 6,600 psf at the bottom. This admittedly direct and simple analysis permitted the first estimate of the magnitude of pressure in the vicinity of the tunnel as the result of slide activity.

Swedish Circle Analyses.—The results of slip circle analysis for a number of assumed failure circles and shear strength parameters are presented graphically by Fig. 13. The circle most closely approximating the zone of movement (center No. 19) resulted in a near unity safety factor with a ϕ of 10° and cohe-

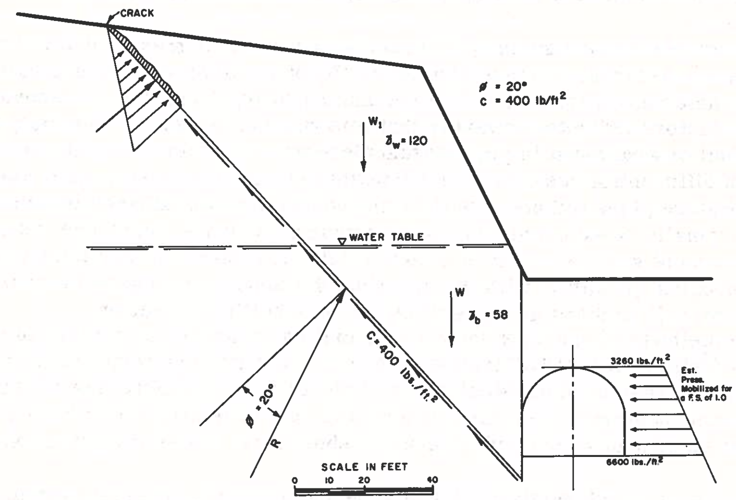

FIG. 12.—COULOMB WEDGE ANALYSIS

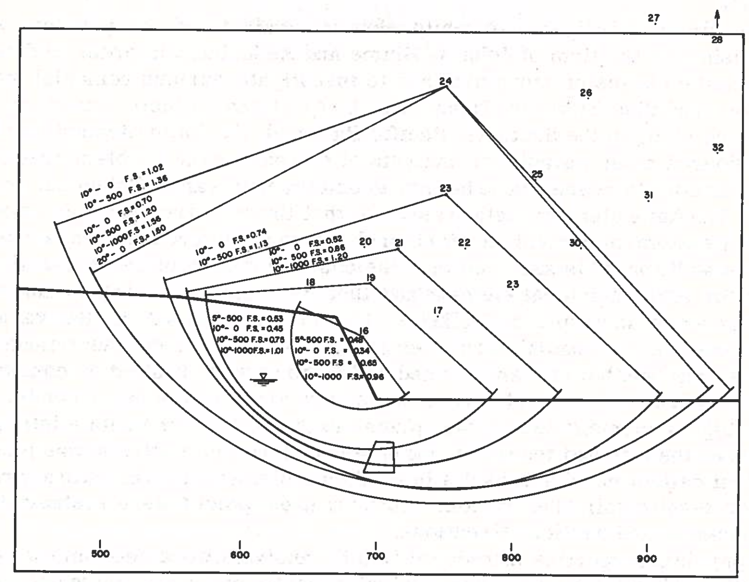

FIG. 13.—SLIP CIRCLE ANALYSES

sion value of 1,000 psf. For both the Coulomb wedge and the Swedish slip circle analysis, the most critical condition occurred through an arch or plane beginning approximately 30 ft up the hill from the retaining wall and extending to its base.

The fact that failure actually occurred at a somewhat greater depth, thereby affecting the tunnel, suggests the possibility of a progressive failure common to clay shales as a result of stress release resulting from the cut excavation. Several factors indicate, however, that this was not the case. The clay shale is residual or weathered in place so that there was no discernible peak-residual strength differential resulting from triaxial compression tests. Also, the apparent failure plane did not extend to the weathered-unweathered interface as would normally be expected. Available physical evidence, therefore, suggests that movement was the result of a lack of lateral support in the vicinity of the tunnel coupled, possibly, with the creation of a zone of weakness immediately behind the wall anchors as a result of the rock bolting operation.

Both methods of analysis show a less critical condition as the arc of plane extends further into the hill indicating a $\phi = 0$ behavior pattern which is contrary to the results of physical tests. This is very possibly due to sample disturbance, specimen size, anisotrophy and rate of testing which have a particularly significant effect with respect to short term slides involving fissured clays.

In any event, both methods of analyses and available physical evidence indicate that bolting to substantially greater depths into the hillside very possibly could have prevented the slide condition or greatly inhibited slide movement.

Finite Element Analysis.—A finite element analysis of the problem was accomplished by the firm of John A. Blume and Associates in order to determine possible causes of movement and to investigate various remedial measures for providing relief to Tunnel No. 1. As stated in their report on the engineering study to the Southern Pacific Railroad, the finite element method was considered most suitable for analysis of this particular problem "because of its flexibility to represent arbitrary geometry with various physical properties." The finite element method assumes that the soil structure may be considered a system of individual structural elements interconnected at nodal points. In addition, it is assumed that the characteristics of the material are uniform for each individual element and that the material subject to analysis is homogeneous, isotropic and elastic. The meshes utilized for the various models consisted of quadrilateral elements constructed from four triangular elements. The number of elements and nodal points were limited by computer capacity and units of computer time within practical limits as is usually the case. A higher element density was placed in the zone of immediate interest, in this case, the railroad tunnel. Along side boundaries no restraint was placed on vertical deflection which results in minimum distortion of the lateral pressures for elastic soil. The bottom boundary nodal points were restrained in both horizontal and vertical directions.

The physical properties introduced into the analysis were determined by in situ compressional and shear wave velocity measurements, from which were calculated the elastic modulus and Poisson's ratio. These measured values averaged approximately 810,000 psi for the elastic modulus and 0.4 for Poisson's ratio. Recognizing these values as probably being an upper limit, an elastic modulus of 140,000 psi and a Poisson's ratio of 0.4 were used in the analysis.

Three basic finite element models were analyzed as part of this program. These were: (1) The hillside subsequent to the construction of the railroad tunnel but prior to the freeway; (2) the hillside after construction of the freeway and retaining wall; and (3) the protective structure in place.

The results indicated that movement resulted from shear yield in an overstress zone extending from the west tunnel wall and extending in a generally circular arc to an area from approximately 50 ft to 90 ft behind the wall at ground surface. Model No. 3 clearly indicated a beneficial effect of a protective structure and provided the basis for the design of the individual bents. It was found that an average value of $33° \pm$ for yield point angle of internal friction outlined a shear yield zone comparable to that indicated by field instrumentation.

CORRECTIVE TREATMENT

The movement data obtained from September 1967 to January, 1968 were of a steady and continuous nature and of sufficient magnitude to indicate a real

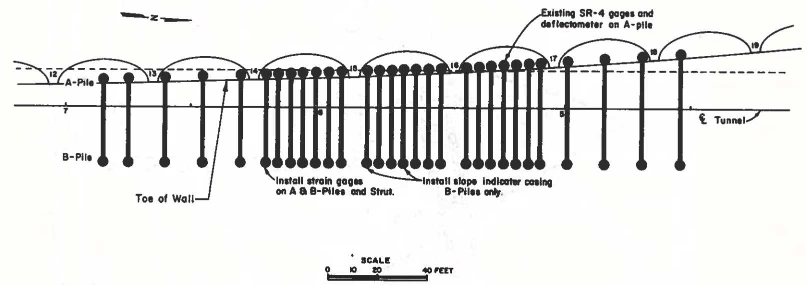

FIG. 14.—LOCATION PLAN OF PROTECTIVE STRUCTURE

threat to the integrity of the tunnel. The fact that the tunnel was unreinforced introduced the possibility of very sudden collapse after prolonged movement which conceivably could be followed by slide of major proportion in the hill area. The consequences of such a slide could have been catastrophic.

Serious consideration was given initially to the construction of steel reinforcing sets within the tunnel. This alternative was eventually abandoned, however, in view of the necessary disruption of train traffic which it would have entailed and the possible further weakening of the tunnel structure during the repair. Consideration was also given to removal of the retaining wall and the flattening of the cut slope to Pennsylvania Avenue. This alternative was not found feasible. Construction of a new tunnel facility along the east freeway right of way was also rejected in view of the time required to complete construction and the extremely high costs involved.

The solution which was ultimately adopted was the construction of a protective structure over the tunnel exterior. The final design of the protective portal system was accomplished by the firm John A. Blume and Associates based upon loading estimates developed from a finite element analysis of the problem. These are presented in graphic form for the design or typical bent in Fig. 18. The final design provided for the installation of 30 exterior frames or portals with from 5-ft to 15-ft centers. Each frame consisted of two drilled holes 48 in. in diameter in which a 36 WF 230 steel beam was placed and back-filled with concrete. The westerly or uphill battered piles were 90 ft long while the easterly vertical piles were 70 ft long. Load transfer between the battered uphill piles and the vertical piles were accomplished by the 14 WF 127 steel beams welded between the two piles 10 ft above the tunnel and enclosed in concrete. A portal layout plan and section are shown in Figs. 14 and 15.

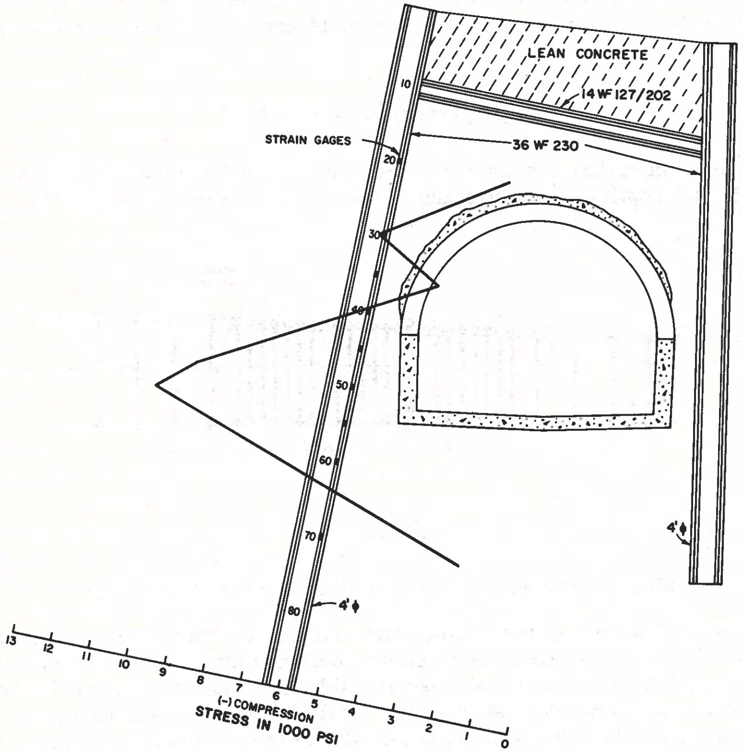

FIG. 15.—FLEXURAL STRESS—A PILE NO. 16.4, SR-4 STRAIN GAGES

The external portal solution had several decided advantages, the most important of which were: (1) Correction could be accomplished quickly; (2) it eliminated the need for disruption of train traffic; (3) it would serve not only to strengthen the tunnel but also to stabilize the slide by imbedment of battered piles into competent rock; and (4) incremental relief would be realized as the individual portals were completed.

Construction of the protective structure began on January 18, 1968. Within a period of 2 weeks after beginning of construction, three individual pieces of drilling equipment were in operation, only one of which had the capability of drilling the battered holes. Drilling was accomplished wherever possible with

augers and buckets. Under conditions of hard drilling, core barrels were used. In several instances where extremely hard drilling was encountered, holes were cased and miners were required. The extreme variability of the soil profile was further indicated by the range in drilling times required for the 90-ft battered piles (13 hr to 196 hr). The operation was carried out on a 20-hr day, 6-day per week time schedule. After drilling and positioning of the 36 WF 230 steel members, the hole was backfilled with a 5-sack concrete with a rapid setting agent. As each pair of piles was completed, excavation for the strut was accomplished with a back hoe. Strut members were welded into place followed by encasement of the standard 5-sack concrete. For final trench backfill a 2-sack concrete mix was used to insure a solid backfill and to permit excavation of adjacent trenches without the necessity of shoring.

MOVEMENT AND STRAIN DATA DURING AND SUBSEQUENT TO REPAIR

Portal Instrumentation.—During the drilling operation for the initial or experimental bent (16-4) a decision was made to instrument the battered A pile

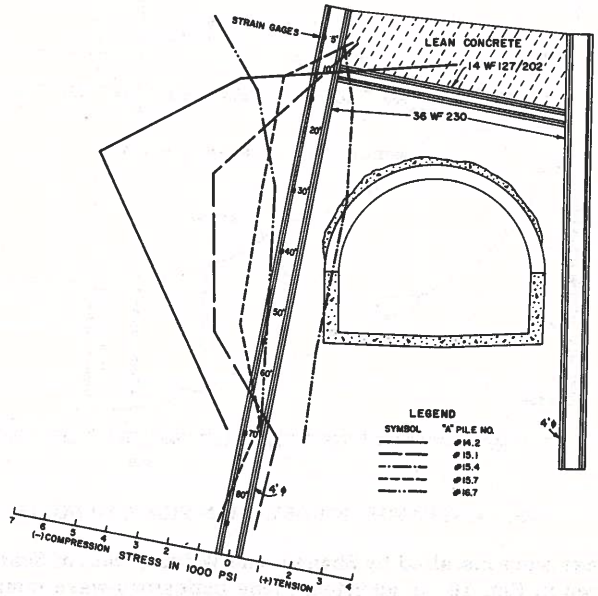

FIG. 16.—FLEXURAL STRESS A PILE, VARIOUS BENTS—UPHILL FLANGE

in order to obtain information on the magnitude and distribution of loads and deflection pattern. It was believed that these data would be necessary to fully evaluate the effectiveness for the portal design and could possibly indicate where design modifications for the rest of the protective structure would be in order. Accordingly, a series of 9 SR-4 strain gages were installed on the inside surface of the outer or easterly flange of the 36 WF 230 steel beam. The location of strain gages at bent 16-4 is shown in Fig. 15. The pile was completed on January 12 and initial readings obtained on January 16, 1968.

On January 2, 1968, representatives of the Southern Pacific Railroad Company and the Division of Highways agreed upon the desirability of additional instrumentation on five of the remaining portals.

In addition, strain gages were also designated for the strut and B pile. Vibrating-wire strain gages were utilized for the instrumentation, based upon the anticipation of faster and simpler installation and greater life expectancy.

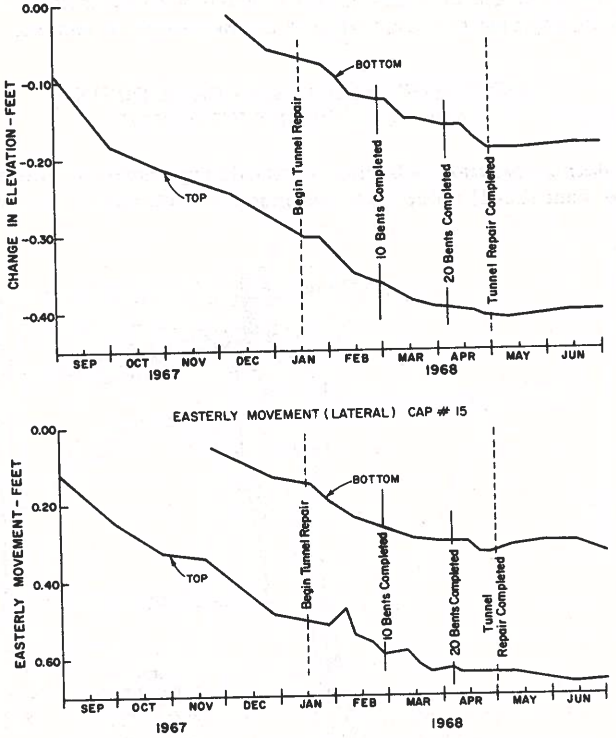

FIG. 17.—CHANGE IN ELEVATION PILE CAP NO. 15

These devices were installed by Shannon and Wilson, Inc., of Seattle, Washington, as shown in Fig. 16. In addition, slope indicators were installed on the B piles of Bents 15-1 and 15-4.

The strain readings taken on the instrumented piles during and subsequent to construction of the protective structure revealed wide variations in magnitude, direction and location on the main load carrying A pile members. By mid-June, 1968, the strain on all instrumented piles had stabilized. The state of stress as of this date is presented in Figs. 15 and 16. Examination of these plots revealed no well defined stress pattern with respect to time of completion, portal location or magnitude and direction of strain, which again demon-

strates the heterogeneous nature of the soils in the area because all instrument piles were located within a 100-ft length along the wall.

As would normally be expected, piles constructed early in the program tended to develop the higher stress levels. In contradiction to this, however, very low stress values were recorded on Pile 16-7 which was the second instrumented A pile in place. With the exception of 16-4, the data revealed fixity at approximately the 70-ft level. Four of five piles instrumented on the uphill flange were in compression, three of which have maximum values at the 30-ft level.

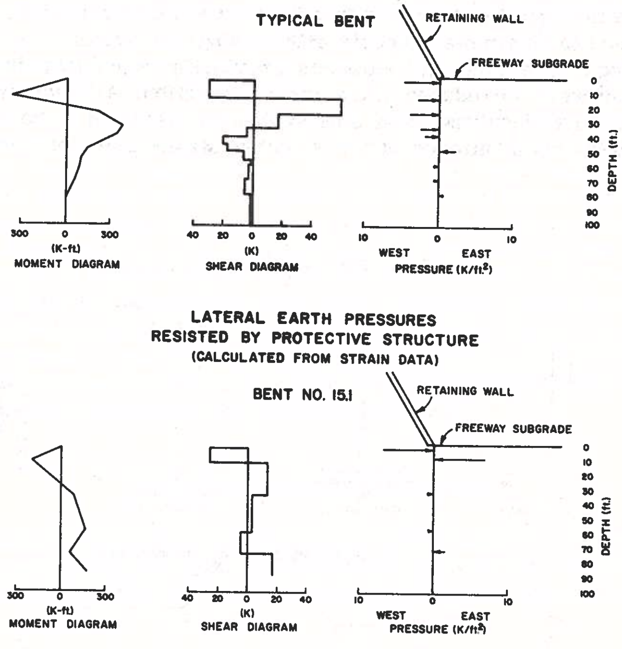

FIG. 18.—LATERAL EARTH PRESSURES—TYPICAL BENT AND BENT NO. 15.1

Instrumented piles 15-4 and 16-4 deviated from this pattern by revealing tension in the uphill flange and thus possibly a higher effective point of load application. B pile 15-1 which was instrumented on the downhill or easterly flange revealed stress reversal at the 30-ft level with maximum tensile value occurring at the strut connection (+ 6,500 psi). The data corroborate that determined from the slope indicator installations on B piles 15-1 and 15-4, which revealed easterly movements of 1/4 in. to 1/2 in. at the strut connection with a fixity condition at approximately the 50-ft level. The immediate beneficial effects of the tunnel repair on hill and tunnel movement were readily detectable by available instrumentation. Plots of tunnel wall distance, slope indicator and anchor cap movement with time, shown in Figs. 11 and 17, show significant movement reductions with only 10 portals complete at the end of February, 1968. For all practical purposes, all types of movement ceased with the completion of the repair in late April, 1968.

In order to fully evaluate the effectiveness of the protective structure and to compare the loadings predicted by the finite element analysis with those indicated by field instrumentation, strain gage data from three of the instrumented A piles (15-1, 15-4 and 15-7) were used to calculate that portion of the loading resisted by the structure. To do this a 2-step solution with a number of simplifying assumptions was necessary. For the first step, it was assumed that the portal was a rigid frame. Fixed end moments due to side sway were calculated based upon the lateral displacements measured with the slope indicators on B piles 15-1 and 15-4. For the second step, it was assumed that the A pile was a fixed end beam, and that all loads applied to the B pile were transmitted through the strut, and that the struts were horizontal. The struts were assumed to be simple supports with loadings as indicated by strut strain gage readings. The fixed end moments previously calculated due to lateral displacement were introduced at the strut connection. All moments in the A pile except those resulting from side sway were assumed to be the result of simple bending as monitored at the selected strain gage locations on the A piles.

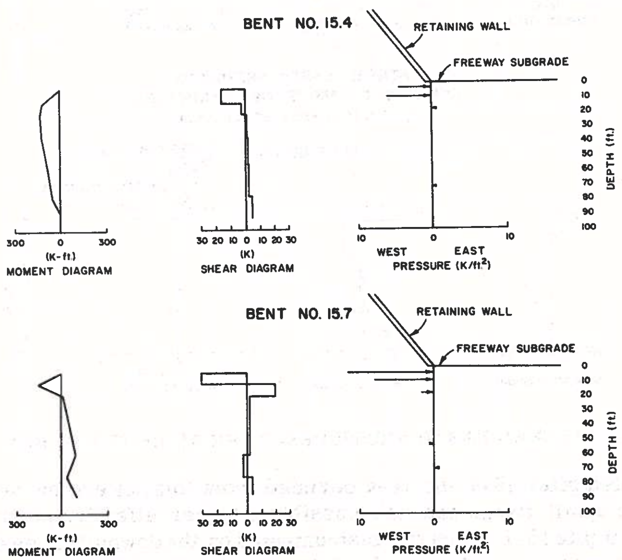

FIG. 19.—LATERAL EARTH PRESSURES—BENT NOS. 15.4 AND 15.7

Strains were converted to moments by application of Hookes' Law. Loadings were determined by successive integration with the following assumptions: (1) A straight line relationship between points of known moment; (2) applicability of the differential equation of the elastic curve. Thus

$$\frac{d^2y}{dx^2} = \frac{M}{EI} \qquad (1)$$

in which: M = moment; E = modulus of elasticity; and I = moment of inertia. Moments calculated from strain gage data on the A pile were added algebraically to the previous calculated fixed end moments due to side sway. From the resulting composite moment diagram and the calculated strut reaction the loading diagram was developed.

A comparison of point loads resisted by the protective structure as determined by the finite element analysis to those actually resisted as calculated from strain and deflection data is presented in Figs. 18 and 19. In making this comparison, a number of complicating factors should be considered before arriving at the two solutions. The vertical section developed from the strain data required modification of the results determined from sloping bents. The number of nodal points far exceeded available strain gage locations. Also, the pressure shown calculated from field strain data represents only that portion of load resisted by the steel members, whereas the finite element analysis predicted the response of the entire structure. A comparison of loadings resisted by the protective structure as determined from strain gage data for A pile 15-4 and 15-7 are reasonably close to those predicted by the finite element method with respect to magnitude and direction. The loads resisted by the top of the pile by the finite element analyses are somewhat higher than those calculated from strain gage data. For Bent 15-1, the easterly pressure on the A pile due to tension in the strut suggests the possibility of response of the portal to loadings on adjacent bents transmitted by the concrete interconnection above the struts. In general, the plots show reasonable qualitative agreement with the finite element analysis for the determination of load to be resisted by protective structure particularly taking into consideration the heterogeneous nature of the soil mass and the necessary simplifying assumptions required to convert available strain data into a loading diagram.

SUMMARY AND CONCLUSIONS

The results of observations, field measurement, analysis, and physical tests made prior and subsequent to the construction of the protective structure around Southern Pacific Tunnel No. 1 in the Potrero Hill area appear to justify the following conclusions:

1. Movement was confined primarily to a wedge of soil beginning 100 ft westerly of the retaining wall and extending to approximately 10 ft below the tunnel. Surface and subsurface movements within this soil mass and on the retaining wall were in an easterly and downward direction. The west wall of the tunnel (an unreinforced masonry structure) moved eastward while the east wall remained stationary. The ceiling and floor of the tunnel tended to move upward.

2. The causes of distress are somewhat complex but are relatable to the excavation for the freeway which removed sufficient material to cause appreciable movement. The tension of the retaining wall anchor rods closed fissures in the shattered and folded sandstone and shale formation which probably resulted in sufficient strain to open cracks at the back end of the anchor rods. These cracks probably freed a wedge of material which acted directly against the west tunnel wall. The structural inadequacy of the tunnel and the low strength of the soil structure permitted the soil mass to creep eastward and downward.

3. The protective structure, consisting of 30 individual bents surrounding

the tunnel has proven successful in halting the movement within the tunnel and in the hill area.

4. The wide variation in the response of the individual protective structure portals as determined from the instrumentation provides further indication of the complexity and heterogeneous character of the soil mass. At all instrumented piles, however, stresses are well below the working level of the steel members.

5. Deeper anchorage of the retaining wall into the hill would probably have prevented or greatly inhibited movement.

6. Even though complicated by the heterogeneous nature of the soil mass, the finite element method of analysis has proven to be an extremely useful tool in predicting the response of the structure to earth loading as indicated by strain measurements.

Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

STRUCTURES IN SOIL UNDER HIGH LOADS[a]

By Jay R. Allgood,[1] M. ASCE

INTRODUCTION

Presented herein is an overview of the design and analysis of structures in soil for resisting blasts from nuclear or chemical explosions, and the spin-off applications of this technology to civil construction. Relations for designing cylinder structure systems are presented and their dependence on soil properties is emphasized. Means are also outlined for achieving efficient designs. To this end, it is shown that at least 10-fold increases in load resistance are achievable by proper use of low modulus materials in the backfill.

BASIC SOIL-STRUCTURE INTERACTION RELATIONS

The prime goal of this and the section, entitled Reducing Interface Pressure, is to provide a set of relations and criteria for the design of buried cylinders. Interest in this goal stems from such diverse requirements as shelters for resisting nuclear effects, block houses for missile and satellite launch vehicles and culverts under deep fills for highway and railroad beds. These requirements may be met by essentially the same design methodology. The reasons are: (1) Soil provides essentially critical damping for fully-buried structures; and (2) attenuation factors and dynamic magnification factors may be applied to account for blast loading. By applying appropriate attenuation factors and, in certain instances dynamic magnification factors, the same relations are usable for static and dynamic cases.

Historically, three general analytical techniques have been pursued in an effort to achieve an understanding of soil-structure interaction, namely, so-

Note.—Discussion open until August 1, 1971. To extend the closing date one month, a written request must be filed with the Executive Director, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 97, No. SM3, March, 1971. Manuscript was submitted for review for possible publication on August 6, 1970.

[a] Presented at the April 6-10, 1970, ASCE National Structural Engineering Meeting, held at Portland, Ore.

[1] Senior Research Engr. and Project Coordinator, U.S. Naval Civ. Engrg. Lab., Port Hueneme, Calif.