Cylinder Pile Retaining Wall

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This paper discusses the conditions encountered during construction of the Seattle Freeway, I-5 through Seattle, Wash., necessitating a change in design of retaining walls, from the conventional reinforced concrete cantilever or counterfort wall to cast-in-place concrete cylinders with welded steel beam reinforcement.

Also discussed are the geological and geographical problems that combined to create massive foundation failures requiring the change in design concept, as well as their solution and the construction techniques involved. Although expensive, the cylinder pile retaining wall provides a method of controlling earth movements by placing the strength of the wall in the soil before excavation begins.

Also covered are the design of a 57-in. diameter concrete cylinder, the original assumptions used for load and resistive capacity of the clay materials, and the reasons for the various assumed values and their locations.

Analysis is also given for the jacking load tests run on the cylinders, as well as a discussion of the differences between the test and calculated values for such items as the load-carrying capacity of the large cylinders, the location of the neutral axis, the tension carrying capacity of concrete, the coefficient of horizontal subgrade reaction, and the behavior of preconsolidated clays during excavation operations.

Observations made for a one-year period on the cylinder wall are given, together with a discussion of the ability of the cylinder to function as a retaining wall under working loads.

• UNANTICIPATED soil conditions encountered during the construction of I-5 through the City of Seattle required a change in design and construction techniques. This change involved a bold approach to sidehill stability maintenance and elimination of damage to adjacent property (outside the right-of-way).

Traffic on I-5 required as many as 12 lanes. These lanes were benched into the sidehill to conserve right-of-way and to permit connection to existing surface streets. A typical section along Capitol Hill (as designed) is shown in Figure 1 and a plan showing a section of the Seattle Freeway is shown in Figure 5.

Basically, the hill is composed of horizontal varved clay preconsolidated through pressure of glacial ice load and an upper mantle of glacial till. Advance foundation explorations had shown that removal of the glacial loading caused vertical fractures in the clay. Trouble was anticipated because past history of the location had revealed the existence of numerous slide areas most of which were fairly shallow and apparently caused by supersaturation of the weathered mantle of exposed clays.

Based on observations and conventional analysis, it was felt that proper drainage during construction would control the excess water problem. In addition, contract specifications were written providing excavation extent and sequence limitations in an
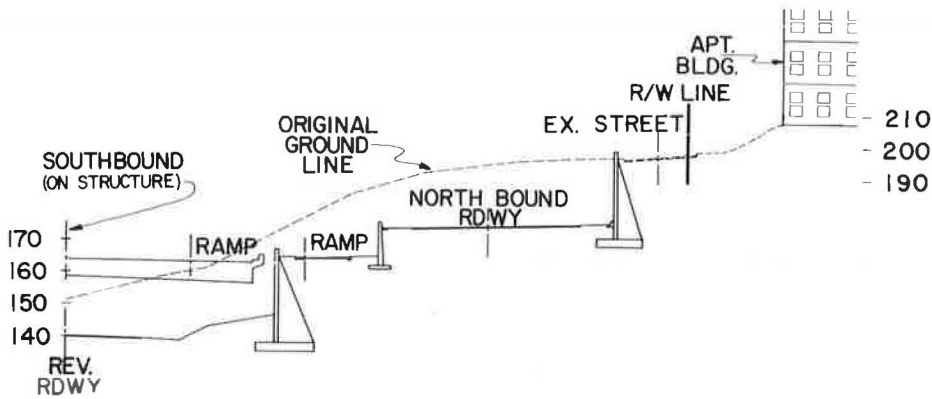
Figure 1. Design of Seattle Freeway, Capitol Hill section (scale: 1" = 60').

attempt to prevent the development of massive slides that might result from the presence of the vertical fracture planes and horizontal varves. The specifications provided a limit of 200 ft horizontally for any major excavation with at least 200 ft between excavations.

Retaining walls as originally designed were to be conventional reinforced concrete, either cantilever or counterfort, depending on height. Design loads applied to the walls varied from equivalent fluid pressures of 30 psf/ft of height for dry areas in granular cuts to 60 psf/ft of height for wet areas in clay cuts.

To insure that actual loads would not exceed the design loads, the backfill behind the walls was to be granular and free draining to reduce lateral pressures. Footings varied between spread and pile depending on available foundation support.

CONSTRUCTION

During construction of the section immediately north of the central business district, the first problem in the nature of a slide in a retaining wall and abutment excavation at Olive Way occurred. The construction area was within the 200-ft specified limits, and the back slope had been cut to approximately 3/4:1. The footings for several sections of the wall were poured on piling driven to a 35-ton load-bearing capacity and battered. Before the wall stems were formed, a slide movement of the backslope started to occur. It appeared to be a series of block slides moving outward on nearly horizontal planes at various elevations above the bottom of the footing.

To reduce the driving force, the top of the cut was lowered immediately and the slope laid back as much as possible. Heavy rock riprap was added behind the wall footing to buttress the slope. Neither had any effect. To more accurately pin-point the exact location and extent of the slip planes, slope indicators were installed that showed the lower limit of the slippage zone to be on a plane approximately 6 ft below the bottom of the footing.

The slope indicator, developed by Stanley D. Wilson of Seattle, used for this purpose consisted of a 3-in. plastic tube inserted in a prebored hole drilled to the required depth. An electronically activated pendulum that travels in preformed grooves was lowered into the plastic tube. This instrument registers the slope at any given elevation. From this information a deflection curve of the tube and an estimation of the forces involved could be determined. The wall was redesigned and constructed as a solid concrete block on the footings thereby adding weight and causing the batter piles to function. This brought good results and the slide movement was stopped. To complete the planned roadway facility in the area, it was necessary to excavate in front of the wall footing to a depth of approximately 20 ft in order to construct a two-lane vehicular tunnel for reversible traffic. To prevent reactivation of the slide some means of toe support for the wall appeared necessary while the tunnel was being built and back-
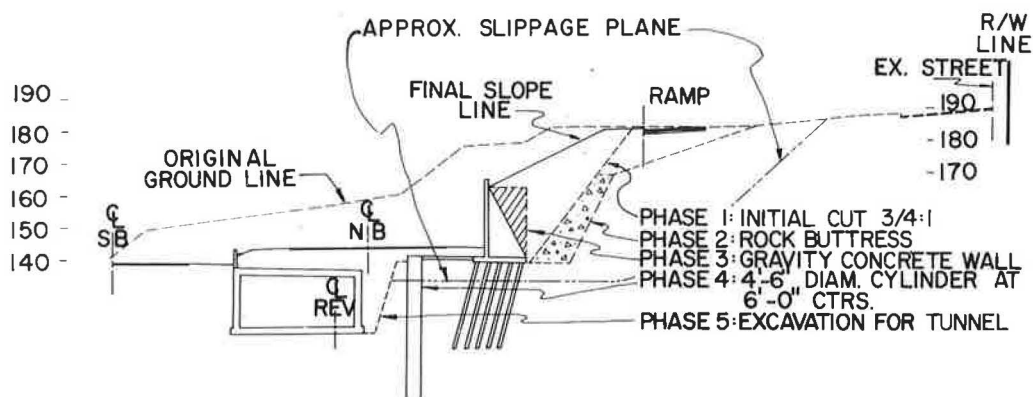
Figure 2. Design of Seattle Freeway, construction of concrete cylinders (scale: 1" = 60').

filled. Strutting across the excavation was impractical, and tie backs through the rock riprap were impossible. Investigation into the possible use of large concrete cylinders poured in place against the toe of the wall footing was carried out. Because there were no known design criteria for this type of construction, it was necessary to establish rational criteria as the design progressed. Reinforced concrete piles 4 ft 6 in. in diameter approximately 45 ft deep with a spacing at 6 ft centers were designed. These poured-in-place cylinders (Fig. 2) solved the problem.

During the excavation of a footing for a bridge structure on the adjacent project to the north, another slide similar to the Olive Way slide started to develop along Lakeview Boulevard. Immediately adjacent to the slide area stood a seven-story apartment building. A 35-ft high retaining wall was to be constructed between the bridge footing excavation and the building. It was evident that the wall excavation would almost certainly cause the slide to progress further into the hill and endanger the building.

Slope indicator tubes were immediately installed to determine the length, depth, and directional components of the earth movement. Surface cracks and the readings on the slope indicator tubes revealed that the movement was a block slide with an approximate depth of 45 ft. The back edge of the moving block angled downward through the wall location just westerly of the building. The base of the slippage was along a defined horizontal weak plane. Although movement (slippage) along this weak plane had not yet extended back under the proposed wall location, there was immediate danger that the slide would progress to the east and the proposed retaining wall location and building area. Because the horizontal layer of weak soil on which the slide was moving extended back under the proposed wall and the apartment house, the contractor was instructed not to proceed with further excavation and pile driving.

Because the back edge of the slide had not yet progressed into the footing area of the wall, consideration was given to lowering the footing below the horizontal slip plane and increasing the design to withstand the additional loading. This would have meant making deep excavations in front of the apartment building and using heavy and expensive shoring with no assurance that another slip plane would not develop below the new footing elevation. In addition, the time required for such conventional construction procedures would have caused considerable project delay.

It was decided to use the poured-in-place cylinder pile design. In this case, an active pressure had not yet developed at the cylinder location, and it was realized that if they could be installed before slippage occurred behind the wall location, the loading criteria could utilize the strength of the undisturbed natural soil formation. Also, in this case, the cylinders could be constructed to extend above the future subgrade of the adjacent roadway to the full wall height. This would allow substitution for the planned wall and would permit excavation in front of the wall without delay for shoring. A face wall was developed on the roadway side of the cylinders to simulate the original design.
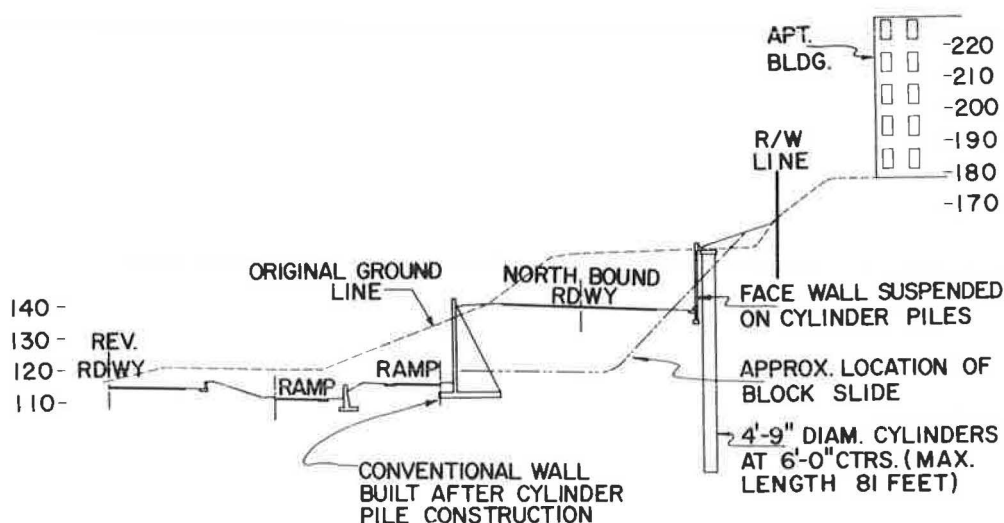
Figure 3. Final construction configuration (scale: 1" = 60').

To develop resistance to the assumed loads, the design called for 4-ft 9-in. cylinders at 6-ft centers. Cylinders up to 81 ft long were required to reach below the slip plane and other possible planes. The loading was too great to permit the use of reinforcing steel; therefore, welded steel beams were employed. The beams provided the necessary stiffness required for their placement into the predrilled holes.

Due to the lack of information available for this type of wall construction, provisions were made for instrumentation of the beams to study the magnitude of the loading and deflections during loading.

SR-4 strain gages and slope indicators were installed on the beams (Fig. 8). EPY-150 epoxy was used to affix these gages to the beam, and Colma Joint Sealer was used to provide a flexible watertight and airtight protection for the wiring connections to the junction box installed at the top of the beam. Also, a slope indicator tube was fastened with an epoxy sand grout mixture to the inside flange and web angle of the beams that were instrumented.

The initial cylinder pile wall construction along Lakeview Boulevard extended approximately 360 lineal ft to the south from the intersection of the Lakeview-Eastlake Frontage road structure and Lakeview Boulevard at Belmont Place (Fig. 5). This section of wall has 62 piling that vary in depth from 62 to 81 ft. The weight of the beams varies from 8 to 16 tons.

To avoid delays between concrete pouring and adjacent hole boring and to provide a minimum concrete curing period of three days, it was decided to maintain at least four spaces between open holes as construction progressed. This allowed the drilling crew to prepare a hole while the beam setting crew and concrete crew finished the cylinder pile in a previously drilled hole.

A Calweld truck-mounted earth borer, a 75-ton Loraine crane with a 100-ft boom, an arc welder, compressor, 10-ton fork lift, and various other smaller pieces of equipment were used in this operation. The Calweld drill rig was equipped with a 48-in. diameter bucket with 5-in. side teeth. Drilling started at pile 18 near the north end of the cylinder wall. Five-foot casing was ordered for installation in the upper 15 to 20 ft of boring to retain the sandy clays in that area. However, in the first hole it was found that a 10- to 12-ft thick layer of very wet sand was present from an approximate depth of 62 to 75 ft. This necessitated full depth casing to prevent sloughing. Excessive moisture from this layer was a constant problem in the borings of the north half of the cylinder pile section, and steady pumping was required before concrete placing. Slough into the holes from this wet zone was not always avoidable, and ultimately it was found

necessary in a few holes to drill and case to the upper limit of the sand zone then place the I-beam and drive it to its final elevation with an improvised mandrel and hammer. Because the concrete backfill was used basically to provide full bearing of the soils against the beam without allowing intrusion into the hole, no loss in stability was expected by driving the beams into the sand rather than boring and placing concrete.

Problems developed between the sequencing of the boring and casing operation and the beam placement operation. Also, the concrete under pressure intruded through sand lenses into the area of adjacent holes. This intrusive concrete was well hardened by the time the construction sequence allowed drilling of the adjacent hole thereby hampering the effectiveness of the bucket drill. This problem led to spacing the beams at 7-ft centers where this construction was employed at other locations.

During the boring of the first set of holes, considerable difficulty was encountered in pulling the casing because the earth behind the wall tended to creep forward and exert its pressure on the cylinders. During subsequent operations, however, as the gaps were filled the casings became easier to pull, indicating that the soil loading was being transmitted onto completed piles.

Progress was slow at the beginning of the project, and some questions regarding the construction method were raised. To work out some of the construction problems, charts based on work experience were prepared to determine a better sequence of operations. It was determined that a second large crane used full time for beam and concrete placing would economically increase production. A 70-ton Manitowoc crane and an additional drill rig (a 5-ft diameter Williams earth auger mounted on a Bucyrus Erie 38B crawler with a 60-ft boom) were put on the project. The complete installation of the 62 cylinder piles was completed in 47 working days.

To make the cylinders work as a unit and to transfer shear between cylinders, a 60- by 12-in. cap beam was poured that engaged the uppermost 9 in. of each steel beam. This cap beam was reinforced with six No. 7 bars. To receive the face wall, blockouts were formed in the concrete backfill on the front face of the beams down to subgrade. On the first beams this was accomplished by bolting a 10- by 18-in. timber block to the steel beams, the front face of which was rounded to the contour of the 5-ft diameter casing. This block arrangement was installed in the field. To eliminate this extra step in the field construction pattern, the blockout was soon changed to 13-ga metal that was welded to the beams in the shop. After the beams were installed and roadway excavation completed, the blockouts were removed to expose approximately 18 in. of the steel beam face. Tension ties consisting of 24-in. bolts at 18- to 24-in. centers were welded to the beams and after conventional forming, the face wall was poured giving the final appearance of a standard retaining wall. The final construction features of the cylinder pile wall are shown in Figure 3.

Slide problems similar to those previously mentioned occurred in other areas of the project. In all cases, except one, the nature of the movement was the same, namely, block slides moving outward on nearly horizontal planes with surface cracks developing from 50 to 100 ft from the cut face. Slope indicators confirmed that the slides extended back horizontally and then angled upward to the surface. The one exception was a mantle slide approximately 300 ft wide that developed during heavy rains and was apparently caused by saturation, softening, and subsequent failure of the upper mantle.

Because the soils along the Freeway route adjacent to Capitol Hill did not respond as anticipated to conventional analysis and slide problems did develop, a solution was employed to provide construction continuation and adjacent property damage prevention. The solution has added many dollars to the construction costs; however, it has proven very successful in terms of effectiveness and speed of construction. Additional study and documentation are still required to prove the criteria developed for the walls constructed on this project.

A part of this studying process has been a search for an answer to the unorthodox behavior of the soils. The answer to date is centered around the theory that clays are preconsolidated under extreme pressure from the glacial periods. The extreme pressure, of course, is now dissipated at the ground surface; however, it is still "locked-in" at greater depths and as deep excavations are made, the release of these "locked-in" stresses combined with the lack of horizontal plane cohesion that have been unloaded
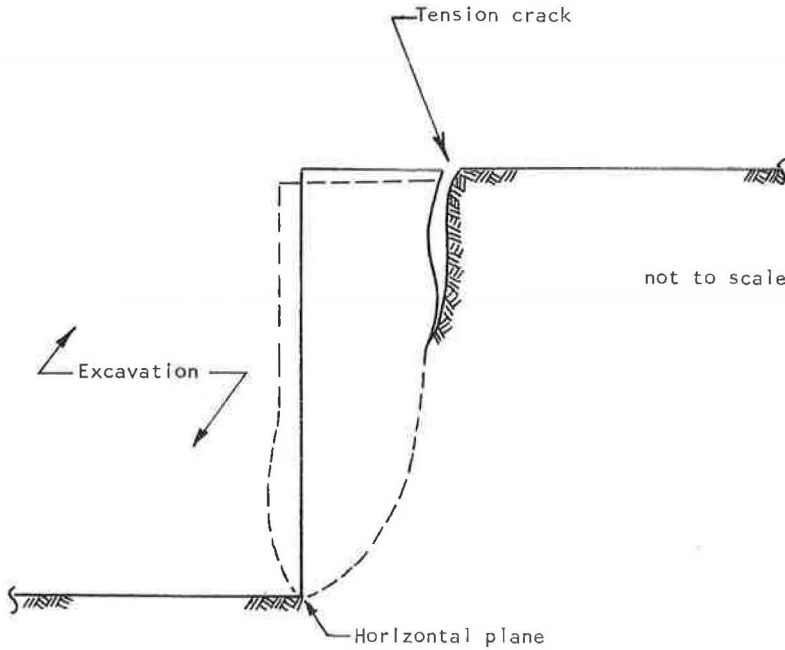
Figure 4. Development of tension cracks due to the release of "locked-in" stress.

may permit horizontal slippage near the toe of the excavation. Lateral movement at the toe along the well-defined horizontal planes quickly develops tension cracks in the slope along the top of excavation (Fig. 4). This theory, as set forth by the Soils Engineering Consultants, has been fairly well documented by slope indicator data. Once the vertical cracks have developed surface water enters and builds up hydrostatic pressure against the wedge and the clay disintegrates and softens. Further movement occurs and new cracks open up and the slide progresses up the slope.

Accepting this analysis, it then becomes very important to prevent the release of pressure or to control the allowable movement so that the strength of the clay is not exceeded. To accomplish this, the cylinder pile concept of retaining wall construction seems to be very well adapted because the strength of the wall is placed in the soil before excavation begins, and adequate stiffness can be designed into the beams to restrict the deflection to within the safe limits of the soil. Another very important benefit from the cylinder pile construction approach is the elimination of pile-driving shock waves under the footings of conventional walls. Taking all points into consideration, it is felt that cylinder pile wall construction has a definite place in areas where room and construction schedules are tight.

DESIGN

The cylinder pile is designed using the loading data shown in Figure 7. The most difficult design problem is determining the magnitude and location of the loads. Cylinder 5 used in Test 1 is designed to carry 30 pcf equivalent fluid pressure (EFP) from the top of the wall down to the slip plane. Cylinder 83, shown in Section BB of Figure 6 and used in Test 2 is designed to carry 45 pcf EFP from the top of the wall down to a point 5 ft below the subgrade. The passive soil pressure, Pp, and the active soil pressure, Pa, were found by using the formula $Pp = Po + \delta Z + Qu$ and $Pa = Po + \delta Z - Qu$ (3). The passive and active soil pressures determined by the formula were used to find the supporting capacity of the soil in the cylinder design. The equations take into account the surcharge of soil, Po, that is overlaying the stratum at which Pa and Pp are being sought by the use of the term Po. The term $Po = (\delta h)$ is the
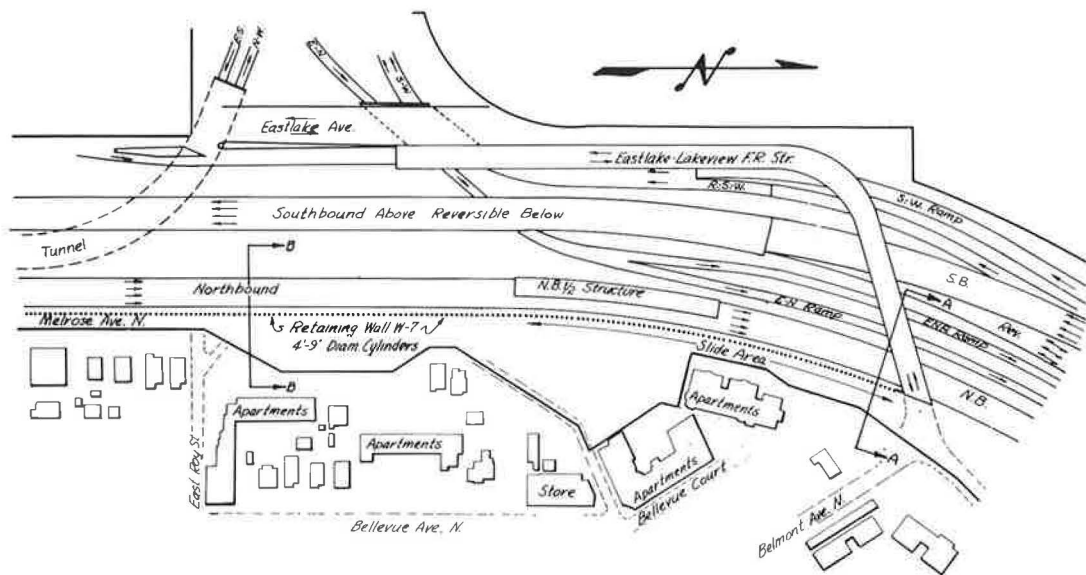
Figure 5. Plan of Seattle Freeway in vicinity of cylinder Wall W-7.

weight of the soil above the soil stratum being investigated, h is the height, δ is the unit weight of soil, and Z equals the thickness of the soil stratum being investigated. The term Qu is the unconfined compressive strength of the soil. From laboratory test data a value of 5 kips/sq ft (ksf) was assumed for Qu in the hard clay material encountered in the area of the cylinders. Values of Qu are shown to range from 4 to 8 ksf for very stiff clay (2). These values of Qu are valid when the contact pressure is less than one-half the ultimate unit bearing capacity of clay (1).

This assumption of 5 ksf is within these limits, as the blow count from a standard penetration test as well as laboratory test results in the area of Wall W-7 (Fig. 5) shows an ultimate bearing capacity greater than 10 ksf. Using an assumed earth pressure and resistive capacity of the soil below subgrade and with no knowledge of Kh (Coefficient of Horizontal Subgrade Reaction) it was felt that the deflection of the cylinders would have to be kept at a minimum in order for the cylinder design to function successfully as a retaining wall. The maximum deflection at the top of the cylinder was set at 6 in. and the maximum deflection at the slip plane or 5 ft below subgrade was set between 1 and ½ in. After the loads and dimensions of the cylinder were determined, a deflection calculation was made to determine whether the cylinder deflection was within this limit assuming the tip of the cylinder to be stationary.

There is an inconsistency between the term "equivalent fluid pressure" and the term "Qu." In the following discussion the definition of the term EFP shall be taken as the pressure exerted on a vertical surface, such as a retaining wall, by a cohesionless free-draining gravel backfill material for which the angle of internal friction, θ, is either known or can be easily determined.

The term "EFP" implies that the soil has a θ angle and little or no cohesion, C, whereas the term Qu, as used in the equation (2), refers to clays with zero or small θ angles and a cohesive value. From observations and tests of similar soils conditions, it is now believed that based on a total stress analysis the hard clays have a high apparent C value when they are undisturbed. When the material is unloaded or disturbed the value of C is greatly reduced and the chunky hard clay exhibits the properties of a soil aggregate having an angle of θ and a reduced value of C. There is a pronounced reduction in the strength of the clay when it is unloaded.

The true in-place strength of the clay soil in this area is not known because it has not been possible to extract a completely undisturbed sample for testing. If this value
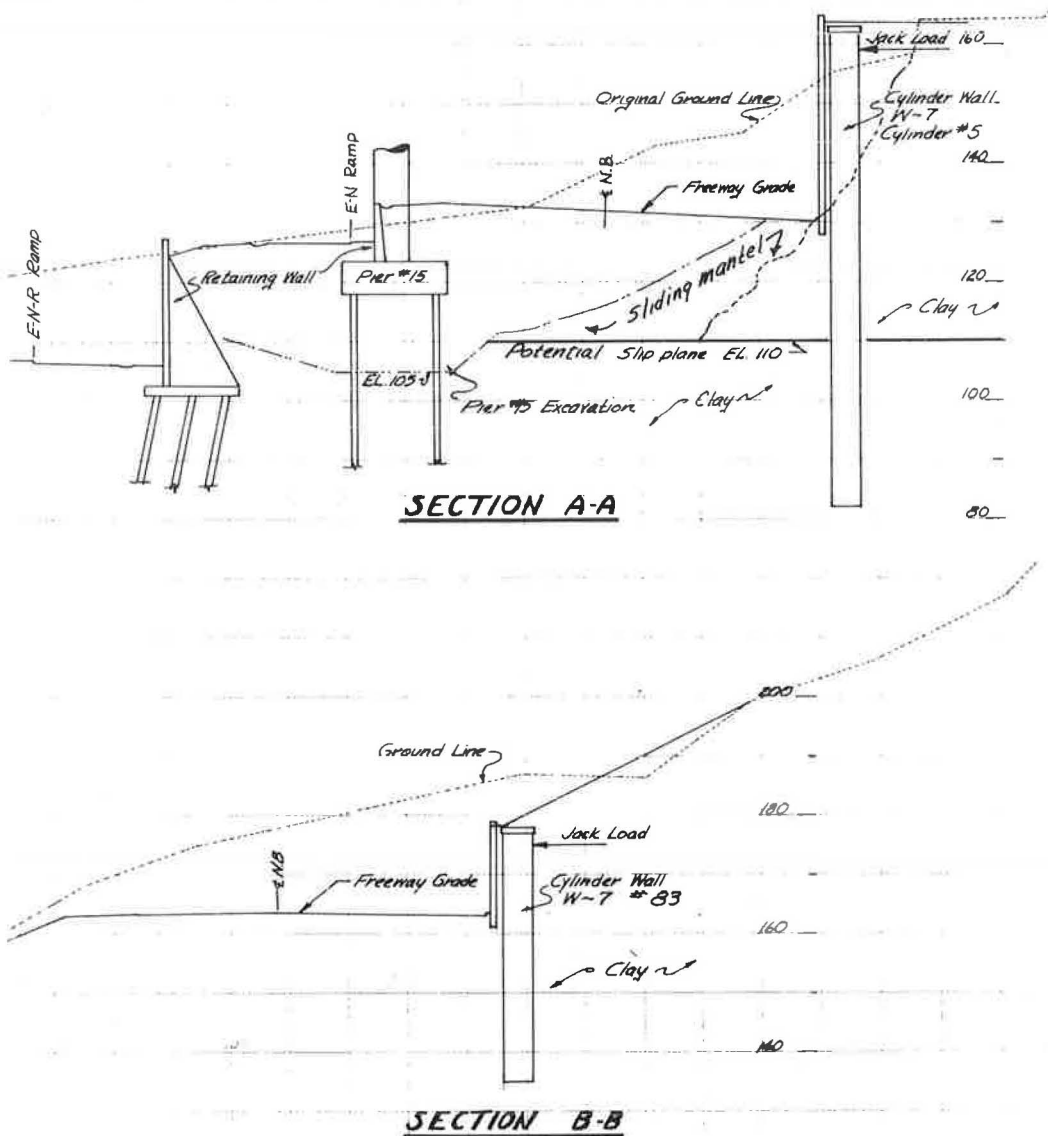
Figure 6. Design and test configuration of cylinder pile Wall W-7.

could be determined, the original assumptions could be changed to give a more precise approach to the determination of the lateral pressure and passive resistance to be used.

The material in the mantle slide behind Cylinder 5 consisted of soft, disturbed back-fill materials which when saturated would produce an EFP of about 84 lb/ft (1). By removing this material and replacing it with crushed stone, the EFP was reduced and assumed to be 30 lb/ft (1).

If the coefficient or horizontal subgrade reaction, Kh, were known, the dashed line shown in Figure 7A would represent the resistive soil pressure. The high soil pressure that is found by using a value of Kh occurring at point X in Figure 7A probably would exceed the strength of the soil at this point and the load would tend to be redistributed as shown in Figure 7B; however, the load diagram shown in Figure 7C was used to simplify the calculations, and this was based on the assumption

Cylinder Pile
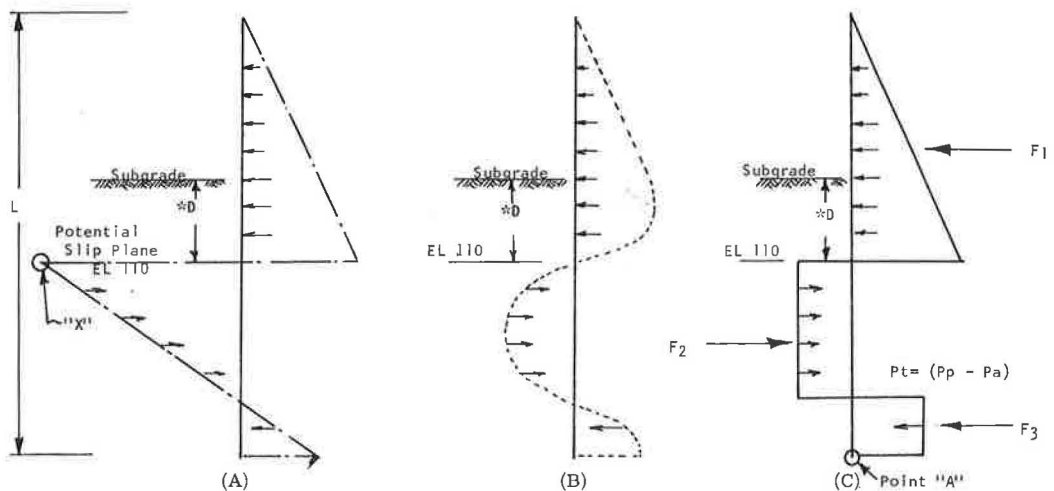
In cases where a slip plane or weak zone do not exist "D" shall be a minimum of 5' - 0" below the subgrade.

Figure 7. Design diagrams: (A) resistive soil pressure, (B) load distribution, and (C) assumed loading.

that the active pressure acted only on the unsupported portion of the cylinder and Qu is 5 kips maximum as outlined elsewhere in this paper.

With an equivalent fluid pressure of 30 pcf and the assumed loading as shown in Figure 7C, the length of the pile was determined by trial and error and by satisfying the two equations, $\Sigma M = 0$ and $\Sigma Fh = 0$.

Knowing the length of the cylinder and the loads acting on it, a moment and shear diagram can be drawn. The size of the beam can then be determined.

The steel beam core was used instead of the reinforcing steel cage because of construction handling problems. A high-strength, low-alloy steel (ASTM 441) was used in the beams to reduce weight and size.

The maximum and minimum center-to-center spacing was set for the 4-ft 9-in. diameter cylinder piles at $7\frac{1}{2}$ ft maximum to 6 ft and 7 ft minimum. The maximum spacing was set by the size of the steel beams; the minimum spacing was set by construction conditions as previously discussed.

The design of the steel beam was based on the assumption that the beam would carry the entire load and the concrete would fill the remaining area in the 4-ft 9-in. diameter drilled shafts. It was contemplated that there would be some interaction between the steel and concrete; but because the amount of interaction was so indeterminate, it was not considered.

Baldwin Lima Hamilton bonded wire gages AB-1 (SR-4) strain gages were placed at ten locations on Test Beam 1, Cylinder 5, and at seven locations on Test Beam 2, Cylinder 83, as shown in Figure 8. Two gages were placed at each location and waterproofed with a sealer compound. A slope indicator tube was also placed on each beam that had the SR-4 strain gages.

The strain gage and slope indicator tubes on Cylinders 2, 5, 11, and 14, which were the first cylinders to be constructed, were originally installed to determine the actual load carried by the beams. With this information the original loading diagram could be verified or adjusted. It was determined by the gage readings that the entire steel beam went into tension for the first seven weeks.

The material in front of Cylinder 5, Wall W-7, was then excavated causing the earth to exert a load on the cylinder wall. When Cylinder 5 was loaded by the excavation, the strain gages on the beam showed tension and compression as anticipated. With tension in all the gages before the initial load, it is difficult to determine the neutral axis, NA,
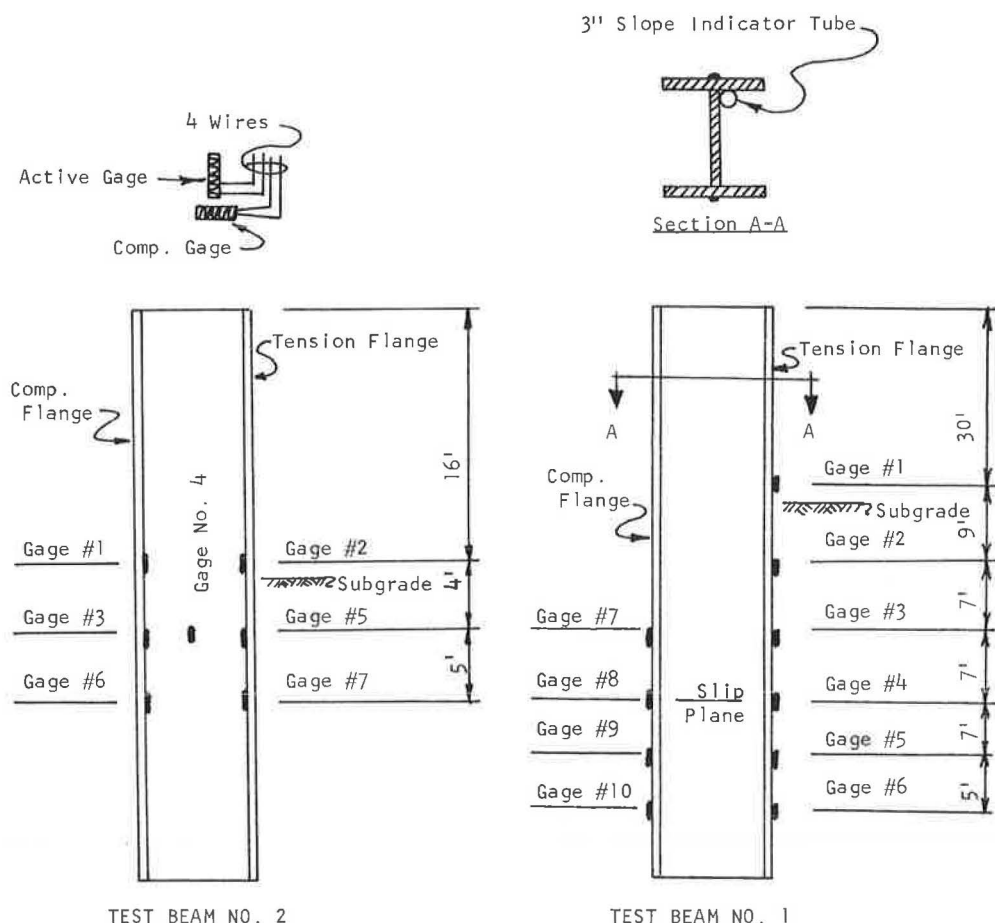
Figure 8.   Strain gage placement and spacing.

of the cylinder. In Cylinder 82, a strain gage was placed on the web so that the tension could be determined and the other gage readings corrected.

It was noted from the first load test on Cylinder 5 that the NA was near the center of the beam section; therefore, it was felt that a gage in the center of the web would only detect the initial tension effects with the loading effects being negligible.

After five weeks of loading on Cylinder 5, the gages and slope indicators showed the beam to be stressed to about 13 percent of the original assumed load. The stress did not appreciably increase beyond this initial load limit and because of the small load on the beam, a load test was necessary to get more information on the structural characteristics of the beam and the supporting characteristics of the soil.

The equipment used for Test 1 and later for Test 2 is shown in Figure 9. The load was applied by a 50-ton jack and the results of Test 1 are given in Table 1. The small stress readings on SR-4 strain gages 4, 5, 8 and 9 (Table 1) and the small slope indicator readings shown on the curve in Figure 11 are all within instrument error.

The moment of inertia, Ixx, of Cylinder 5 was calculated by the formula $\Delta = PL^3/3EI$ assuming the Cylinder to be fixed at 34 ft below the top. The point at which Cylinder 5 was assumed fixed was determined by the test dial gage deflection curve shown in Figure 11. The Ixx of the transformed section was calculated assuming the cylinder was an uncracked section. The two values of Ixx agree fairly well with Ixx transformed section equaling 8.8×10^5, in.^{4} and Ixx (by deflection) equaling 10×10^5, in.^{4}. The compressive strength of concrete as shown by the curve in Figure 10 is 6,000 psi, and
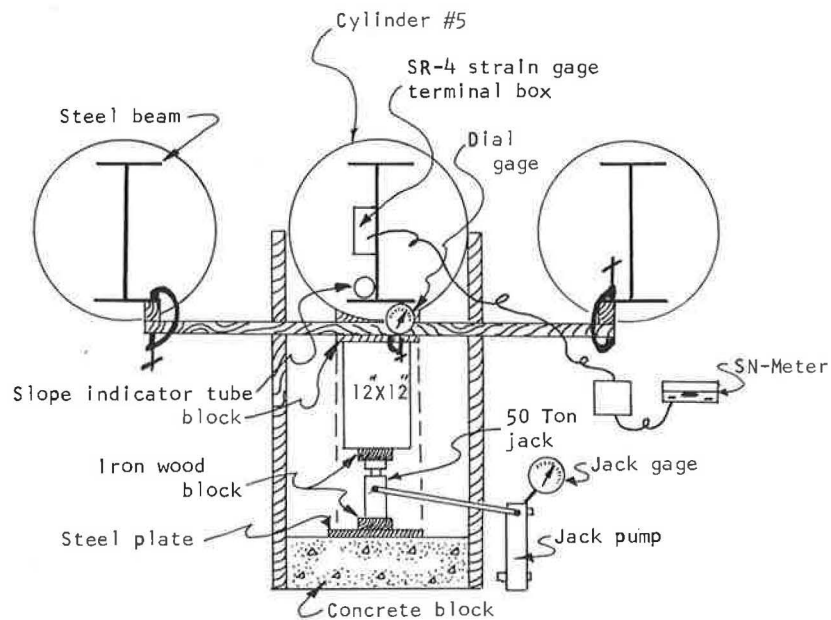
Figure 9. Cylinder test plan configuration.

the modulus of elasticity of concrete, Ec, was taken to be as tentatively recommended by ACI-ASCF for prestressed concrete, approximately 5,000 ksi while the modulus of elasticity of steel, Es, was taken to be 30,000 ksi giving an n of approximately 6.

The maximum stress in gage 1 as given in Table 1 is 2.4 ksi, whereas the calculated stress in the cylinder at that point using the uncracked section and using an n = 6 is 3.0 ksi. This shows fair correlation between test and calculated values.

The added stiffness realized in Cylinder 5 (not anticipated by the initial assumptions) is believed to be mainly caused by the ability of the concrete in the cylinders to carry tension. The added stiffness may also be due, in part, to the initial tension in the steel that was indicated by the strain gages and may have been caused by the shrinkage of the concrete onto the steel beam which in turn would set up tension stresses in the steel through the effects of Poisson's ratio.

By using the formula for beams on elastic foundation (3) and assuming an infinite beam, a value for the coefficient of horizontal subgrade reaction, Kh, was investigated. Using the dial gage deflection curve, Ixx, and the loading data from Test 1, the value of Kh was calculated to be 2.16 k/cu in. This value of Kh is much higher than any of the recommended values in hard clay.

Cylinder 5 was reanalyzed as a beam on an elastic foundation (4) using an assumed value of 850 pci for Kh, Ixx for an uncracked section, and the loads as shown in Figure 7C. The calculations showed that there was very little difference in the maximum moment between this method and the original trial and error method used.

TABLE 1

TEST DATA FOR TEST 1, CYLINDER 5, WALL W-7

Load (ton)	Upper Dial (in.)	1 ksi	2 ksi	3 ksi	4 ksi	5 ksi	7 ksi	8 ksi	9 ksi
10	0.02	--	--	--	--	--	--	--	--
20	0.074	+0.84	+0.450	+0.30	--	--	-0.15	--	--
25	0.109	+1.050	+0.450	+0.45	--	--	-0.15	--	--
30	0.147	+1.50	+0.600	+0.60	+0.09	--	-0.15	-0.15	--
40	0.250	+2.4	+1.05	+0.90	+0.15	--	-0.30	-0.15	--
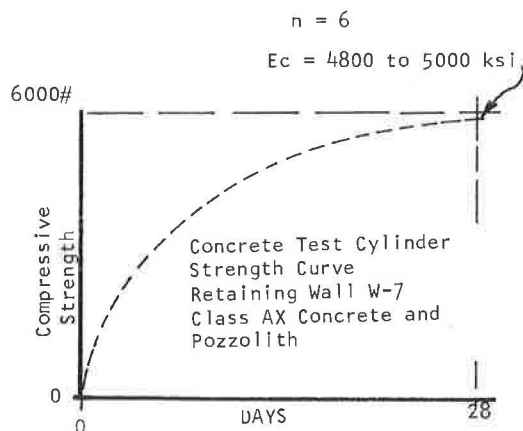
Figure 10. Concrete compressive strength curve.

Figure 11. Deflection curve for Test 1 showing dial gage and slope indicator data.

A total deflection of approximately 2 in. was calculated and was less than initially anticipated; therefore, the simplified trial and error solution using the loading as shown in Figure 7C and a limiting value of 5 ksf for Qu gives results that are very close to those found by the elastic analysis.

The calculations of Kh show that a small change in deflection will cause a large change in the value of Kh; therefore, the value Kh within a large range has little effect on the deflection of the cylinders when the value of EI is large.

After evaluating the information in Test 1 and finding that the results were not as anticipated another test was set up on Cylinder 83, shown in Section BB Figure 6, to verify some of the data from the first test and further investigate the location of the NA, tension effect, and Kh.

The same equipment and method used in Test 1 were used in Test 2. The location of the SR-4 strain gages are shown in Figure 8 with gage 4 being placed in the center of the web so that the tension effect and NA could be evaluated.

Tension showed on all the gages in Test 2 before the cylinder was excavated. The actual tension cannot be completely evaluated because of a malfunction of the strain gage meter a few weeks before the test was to be made.

The SR-4 strain gage readings for Test 2 are given in Table 2, and the deflection curve is shown in Figure 12. Strain gage 4 that is on the web of the steel beam showed negligible change during the test loading; therefore, it was assumed that the cylinder acted as an uncracked beam during the entire test.

The property of the cylinder pile to act as an uncracked beam during loading was also observed in Test 1 and with the added information showing the NA to remain in the center as shown in Test 2, it is felt that the concrete does take tension that is normally not assumed in design.

The maximum stress shown by the strain gages and those calculated using an Ixx of the transformed section seem to agree because the calculated stress at gage 3 is 2.2 ksi and Table 2 shows a stress of 2.1 ksi for gage 3. The calculated deflection 0.0926 in., assuming the beam to be fixed 23 ft from the top (determined from the Test 2 dial gage deflection curve) using the transformed Ixx and the observed total deflection 0.104 in., also agrees fairly well.

Using the data found in Test 2, the value of Kh was calculated to be approximately 200 kips/cu. in. This is larger than the value found in Test 1 and much larger than
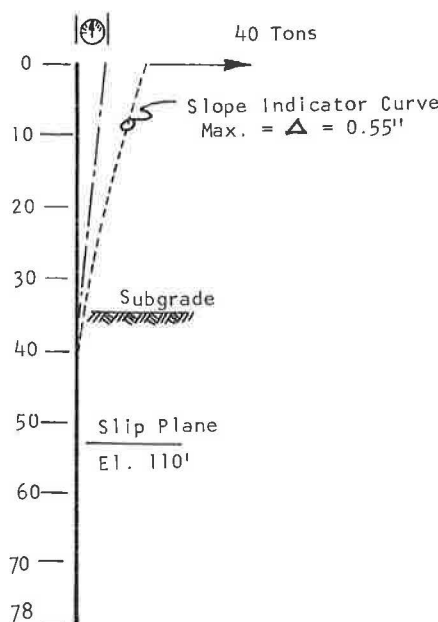
TABLE 2
TEST DATA FOR TEST 2, CYLINDER 83, WALL W-7

| Load	Upper Dial	Lower Dial	1 ksi	2 ksi	3 ksi	4 ksi	5 ksi	6 ksi	7 ksi
10k	0.006	0.001	-0.03	+0.21	-0.18	0.0	+0.18	-0.03	+0.06
20k	0.007	0.003	-0.09	+0.12	-0.15	0.0	+0.15	-0.03	+0.06
30k	0.013	0.003	-0.18	+0.18	-0.21	0.0	+0.24	-0.18	+0.09
40k	0.012	0.003	-0.18	+0.18	-0.24	0.0	+0.09	-0.15	+0.12
50k	0.020	0.004	-0.18	+0.18	-0.30	0.0	+0.24	-0.21	+0.15
Subtotal	0.058	0.014	-0.66	+0.87	-1.08	0.0	+0.90	-0.60	+0.48
50k	0.031	0.008	-0.48	+0.45	-0.48	0.0	+0.36	-0.30	+0.21
60k	0.009	0.002	-0.12	+0.09	-0.12	0.0	+0.15	-0.12	+0.06
70k	0.013	0.003	-0.15	+0.15	-0.30	0.0	+0.27	-0.15	+0.15
80k	0.015	0.006	-0.18	+0.18	-0.30	0.0	+0.30	-0.24	+0.18
90k	0.009	0.005	-0.18	+0.24	-0.30	0.0	+0.27	-0.24	+0.18
Total	0.104	0.030	-1.29	+1.53	-2.10	0.0	+1.89	-1.35	+1.05
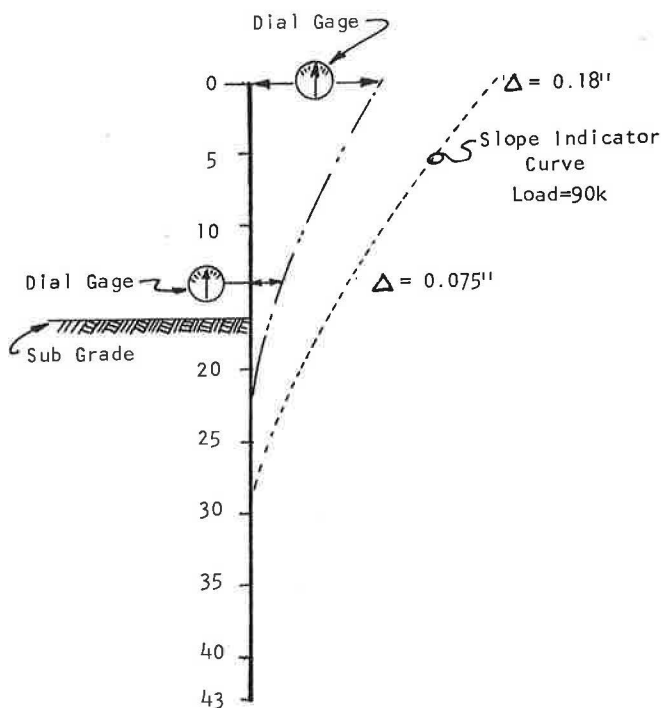
Figure 12. Deflection curve for Test 2.

any recommended value. (The values for Kh in this paper were calculated using the full diameter of the cylinder.)

The strain gages in Cylinder 83 were read before, during, and after the unloading or excavation in front of Wall W-7. After evaluating these readings using the load-to-stress data found in Test 2, it was determined that an EFP load of approximately 100 pcf was impressed on the cylinder during the unloading operation. The large load is believed to be caused by the release of the high "locked-in" lateral stresses in the overconsolidated clays.

The results of Test 2 have confirmed most of the data found in the first test, but added testing is necessary to resolve all the questions that this type of design has brought up.

The two load tests that have been conducted on the cylinder pile are only short-time load tests at best. The final test of this type of design would have to be determined over a longer period of time.

The SR-4 strain gages and slope indicators on cylinders 2, 5, 11, and 14 have been observed for approximately one year. Slope indicator and strain gage curves from July 1962 to July 1963 are shown in Figures 13 and 14. The curves show the loading on the cylinders to be quite constant between August 1962 and April 1963. In April 1963 added load was brought onto the cylinders by a malfunction of horizontal drilling in the area. During the drilling operation for horizontal drains, air was being used at 100 psi to blow out the tailings in the drill hole. The drill hole became plugged and the air pressure pushed the water through the fissures and cracks in the hill behind Wall W-7 causing the release of the old slide. The added load came on the cylinder wall in April 1963 as shown in the curves. The loading on the cylinder wall again became quite constant from April 1963 to July 1963.
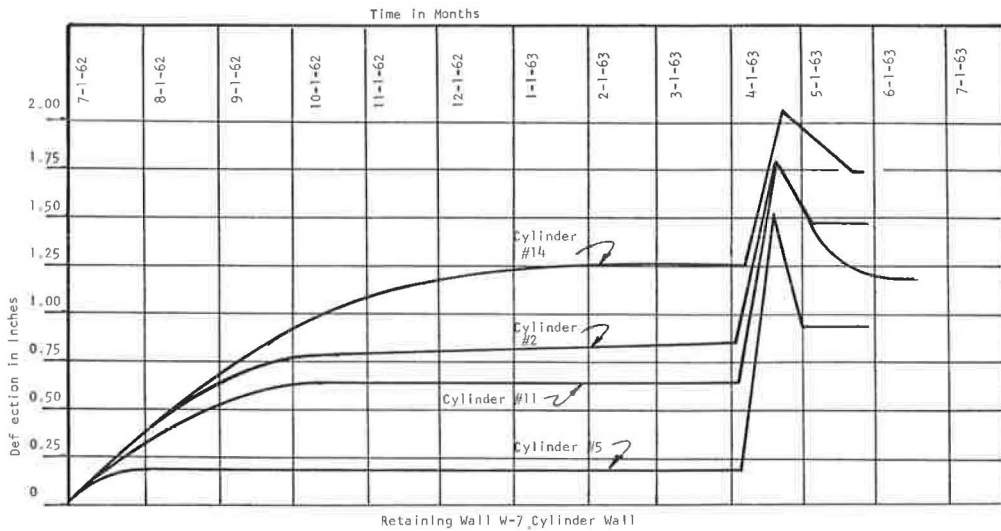
Figure 13. Curves showing total deflection at top of cylinders 2, 5, 11, and 14 vs time.
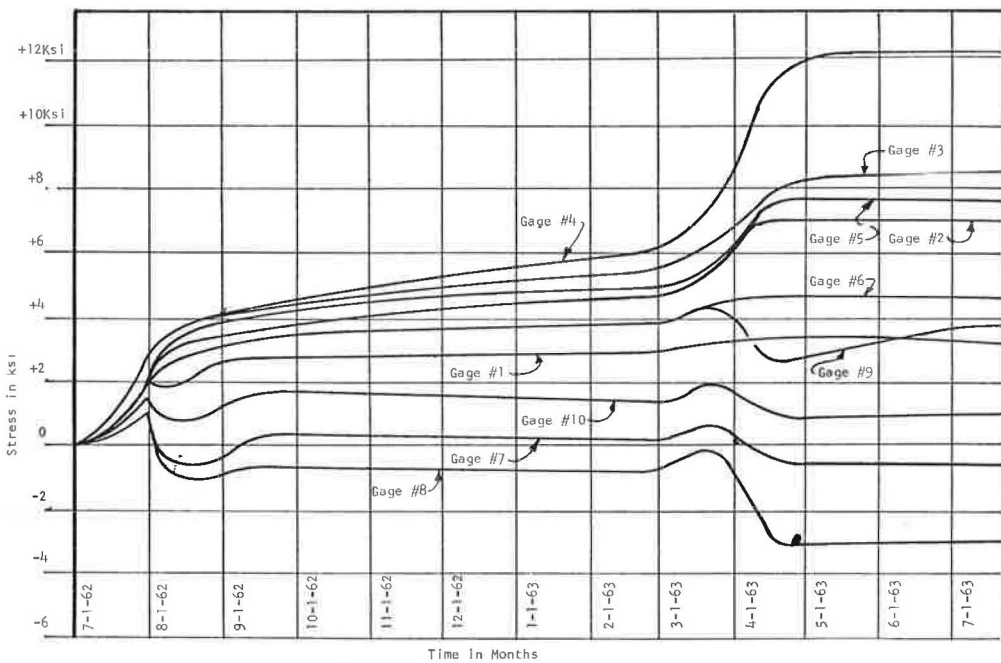

Figure 14. Curves showing stress vs time of cylinder in Wall W-7.

It has been observed for the short-time test that the cylinders are much stiffer than originally anticipated, and this is believed to be due to the capacity of the concrete to carry tension. As the tests are by no means conclusive as to the tension-carrying capacity of the concrete, it is believed that only a conservative amount of tension should be considered in future designs. Both the tension-carrying capacity of the concrete and the effect of tension in the steel of this type of section should be further in-

vestigated before any large value is assigned to the tension-carrying capacities of the concrete.

The type of load test conducted on the cylinders is not considered to be suitable for determining the Kh value of the clay because the inherent error in the testing equipment is excessive for such use, and the Kh value was found to be very sensitive to deflection in the cylinder design. A more suitable test for determining the Kh value has been proposed and is being carried out at this time.

The long-term test shown by the curves in Figures 13 and 14 shows the cylinder pile to be stable under normal working loads. The slope indicators, giving very little information in the short-time load test because of their error, have proven to be valuable in this type of testing because the error over a longer period of time tends to balance out.

Cylinder pile design has given rise to many questions regarding loading, capacity of the soil, and the structural properties of the cylinder pile. The tests that have been conducted on the cylinders have given some insight as to the properties of the cylinders, but further testing is required to fully answer the questions that have arisen.

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