Abridgment

Plate Load Tests for the West Papago/I-10 Inner Loop in Phoenix, Arizona

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ABSTRACT

The proposed West Papago/I-10 Inner Loop freeway will have extensive reaches of retaining wall structures. Many of the walls will have footings within the desiccated silty clay, sandy clay, and clayey sand overburden and will be exposed to wetting by normal runoff and automatic watering systems. The maximum presumptive bearing pressure for the overburden soils would rarely exceed 1.5 trillion ft^2 in accordance with local practice. The Arizona Department of Transportation and the FHWA authorized the conducting of plate load tests to investigate the feasibility of increasing the allowable bearing pressure. A small test fill was constructed with the overburden material compacted to 95 percent of maximum dry density as determined by the modified compaction test. Tests were then performed on the compacted fill as well as on the wetted fill. Additional tests were performed on natural ground in the existing and wetted conditions. Load-settlement curves were analyzed and the resulting ultimate loads were used as a basis for selecting design bearing values for shallow spread footings founded within the overburden. Significant increases in allowable bearing values will result in more walls being designed for shallow soil bearing spread footings with considerable savings in construction costs.

The proposed I-10 Inner Loop freeway in Phoenix is primarily a depressed roadway and will have extensive reaches of retaining wall structures along the freeway and at the I-10 and I-17 interchange. Many of the walls will have footings within the desiccated overburden where bearing areas will be subject to flooding or wetting from normal runoff and automatic watering systems. Locally, the maximum bearing value for walls in the overburden rarely exceeds 1.5 trillion ft^2. Plate-bearing tests were authorized by the Arizona Department of Transportation and the FHWA to determine the feasibility of utilizing higher bearing values. This paper contains a description of the: soil conditions, load test program, analysis of test results, and selection of allowable bearing values for use in retaining wall foundations.

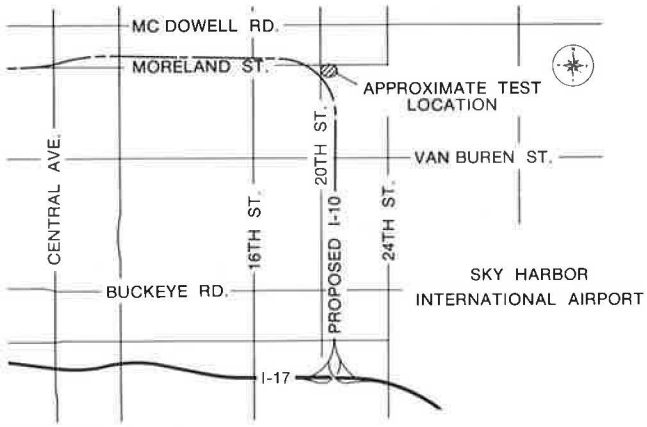

FIGURE 1 Location map.

SITE CONDITIONS

The test site, as shown in Figure 1, is underlain by 16 ft of desiccated, alluvial fan materials comprised mostly of clayey sand, sandy clay, and silty clay. These soils are characteristic of the surficial soils of downtown Phoenix, and are often highly stratified and moderately-to-strongly calcite-cemented, and contain scattered gravel and calcareous concretions. These materials have been arbitrarily grouped and termed "overburden" for this project. The overburden has particle sizes and Atterberg Limits characteristic of cohesive materials and have a Unified Classification of SC, CL, and ML. when undisturbed, segmental liner samples are subjected to consolidated, undrained direct shear tests, phi angle, ϕ, of 10 to 70 degrees and unit cohesion, c, of 0.06 to 0.7 trillion ft^2 are common. Directly beneath the overburden, there is a thick, dense layer of sand-gravel-cobbles (S-G-C) with random boulders up to 24 in. Ground water is generally 50 ft or more below ground surface.

PREPARATION OF SITE

The location for the bearing test on natural ground was prepared by removing the upper 2 ft of overburden to permit identification of buried footings, utility trenches, and to ensure that the bearing plate was in undisturbed soil.

In the fill test area, the natural ground was stripped of vegetation and deleterious materials, the subgrade scarified, moisture conditioned, and compacted to 95 percent of maximum density as determined in accordance with the ASTM standard on modified compaction (ASTM D 1557). The adjacent overburden borrow area was "disced" and moisture-conditioned; borrow was picked up and placed with a self-loading scraper. The fill was leveled into uniform loose lifts of 8 in. and compacted with a self-propelled sheepsfoot roller. Placement of the fill was visually monitored and field moisture-density testing performed to confirm compaction to

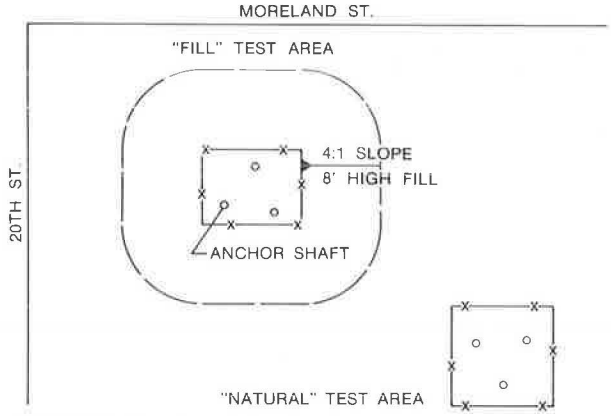

FIGURE 2 Site plan of plate load tests.

95 percent of maximum dry density as determined by the Modified Compaction Test. The completed fill was approximately 8 ft high, with a 40 x 40-ft top and side slopes of 4:1. The plate load test site plan is shown in Figure 2.

LOAD-SETTLEMENT MEASUREMENTS

A reaction system capable of applying a 40-t test load was selected, and consisted of a W24 x 104 steel beam. A triangular anchor scheme provided for efficient handling of the beam for adjacent testing (see Figure 3). Anchors consisted of 24-in. straight-sided, drilled shafts.

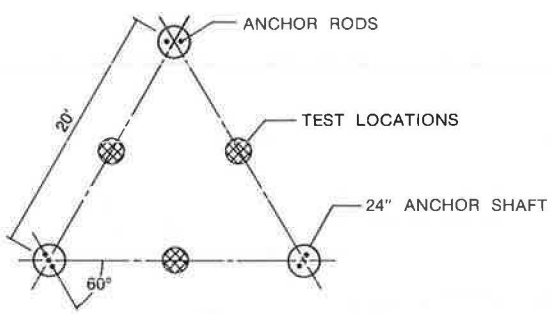

FIGURE 3 Anchor shaft layout.

Immediately before seating the bearing plate, the bearing surfaces were hand-trimmed, with final leveling accomplished by a dusting of silica sand. The steel bearing plate was 1-in. thick and 24-in. in diameter, and was reinforced with from one to three 12- and 18-in. diameter plates. Loads were applied by a manually operated 60-t hydraulic jack fitted with a calibrated pressure gauge. A ball-and-socket joint was positioned between the jack and reaction beam to provide a uniform load application.

Plate settlement was measured by three, 2-in. travel dial gauges positioned 120 degrees around the plate and 1-in. from the edge. Three additional dial gauges were set at 1-ft centers outside the edge of plate to measure movement of the ground surface during loading. All gauges were fixed to a 2 x 3-in. steel tubing beam supported on both ends at a distance of 11 ft from the center of the bearing plate. The test setup is shown in Figure 4.

For the inundated tests, flooding of the test area was used to wet the soil beneath the plate. A

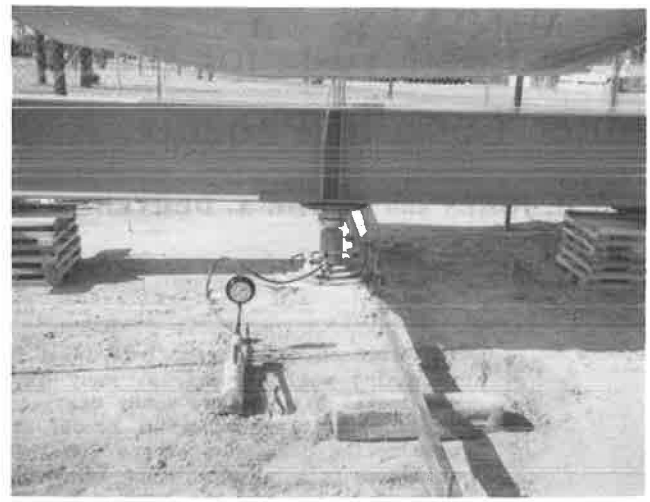

FIGURE 4 Plate load test setup.

6-in. high soil berm was constructed around the plate in a 6 x 6-ft area and filled with water. Wetting was aided by four, 4-in. diameter borings, 5 ft in depth, drilled radially 4 ft from the center of the plate. The borings were filled with silica sand. Flooding was maintained for 24 hr and ponded water removed just before commencing loading.

PLATE LOADING PROCEDURE

Plate loading was in general accordance with the provisions outlined in ASTM Standard D1194, Bearing Capacity of Soil For Static Load on Spread Footings. Selected load increments were applied to the plate and maintained for a minimum of 15 min. Loads were added until the 40-t capacity of the load frame was reached or until the total settlement of the plate exceeded 10 percent of the plate diameter, or 2.4 in. Following the last load increment, the load was released and rebound measurements taken. Intermediate load-unload cycles were necessary in some instances to adjust equipment and provide additional jack extension.

ANALYSIS

The plate settlements recorded for the four tests were used in developing load-settlement plots. Final settlement for each load increment was selected by averaging the final settlement readings for the three dial gauges. The plots of load versus total settlement are shown in Figure 5 for the two tests on compacted overburden fill and in Figure 6 for the two tests on natural ground.

Interpretation of the load-settlement curves involves identification of the failure point, or ultimate load at which the loaded plate causes a bearing capacity failure of the supporting soil. In the case of a general shear failure, the failure point is theoretically clearly definable as a peak or high point in the curve. However, for punching or local shear conditions, there is seldom a sharp break in the curve and the failure point is usually difficult to determine (1).

A peak load point was not clearly identifiable on any of the four load-settlement curves. Among the methods reviewed for interpreting plate load test plots, the Semi-Log Plot-Intersecting Tangents method

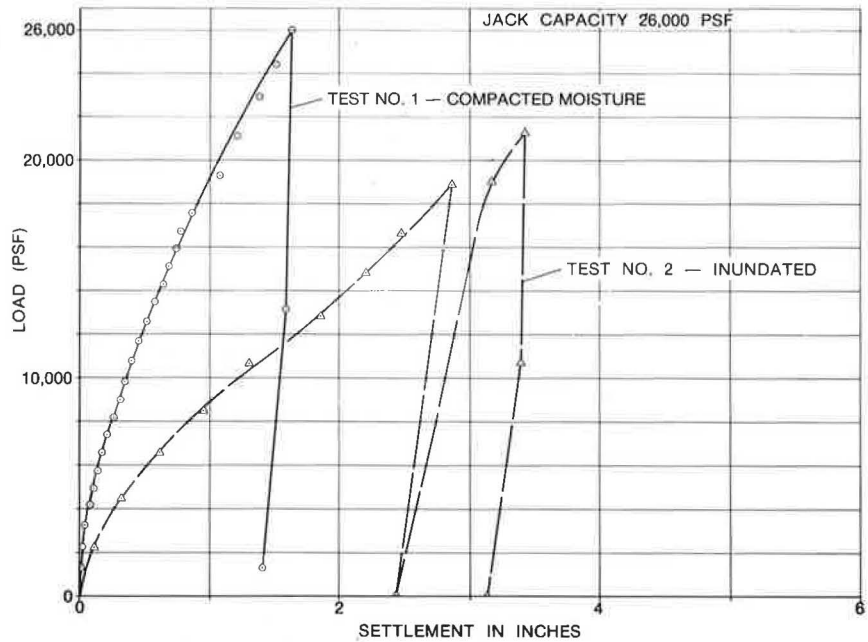

FIGURE 5 Load-settlement curves—compacted overburden fill.

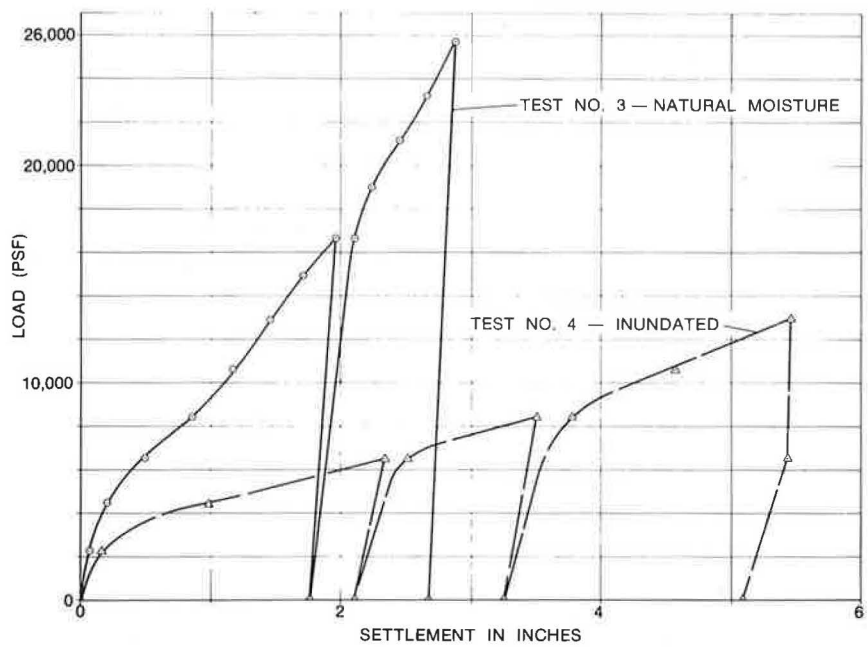

FIGURE 6 Load-settlement curves—natural ground.

was determined to provide the clearest definition of interpretation and thus selected as the preferred method. In this method, the loads are plotted to a log scale and the settlement to arithmetic scale. The ultimate load is defined by intersection of two tangents, one to the initial portion of the curve and one to the outer straight-line portion. The semi-log plots and interpretation are shown in Figures 7 and 8. Evaluation of ultimate load values and their relationship to the anticipated behavior of full-size footings is dependent on several factors, including soil type and conditions, plate size, and confinement of the plate, which are directly related to the individual load tests (2).

The scattering of soil shear strength test data and the plate load test procedure resulted in the assumption that the soil be considered to have only cohesive strength. Local shear failure was assumed to occur as the plate was observed to punch into the soil without displacement of the adjacent soil surface as would normally be expected in a general shear failure condition (3).

In analyzing the ultimate load test values and solving for soil shear strength, or cohesion, the following basic bearing capacity equation was considered as best representing the shear failure conditions for the test performed (4):

$$Q_{ult} = cN_c + \gamma D_f N_q + 1/2\gamma BN_\gamma \quad (1)$$

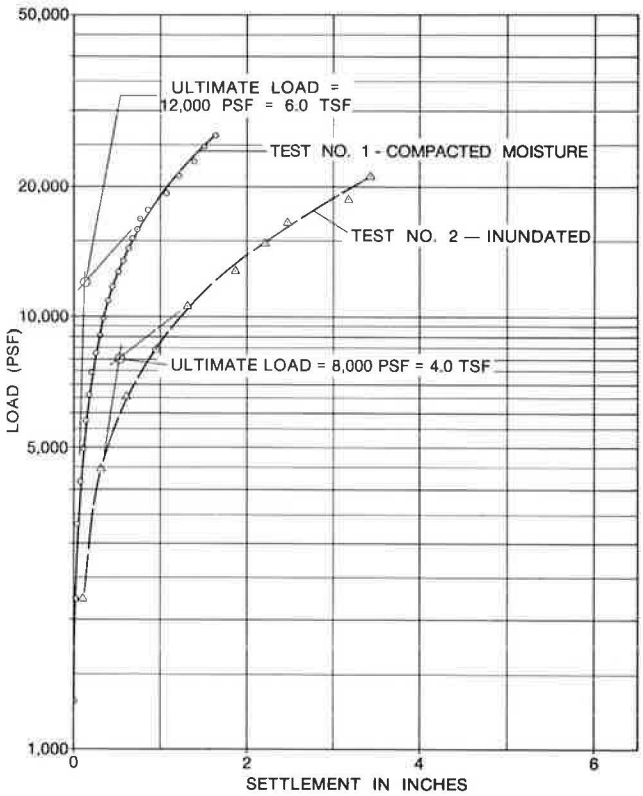

FIGURE 7 Semi-log plot, intersecting tangents method of interpretation—compacted overburden fill.

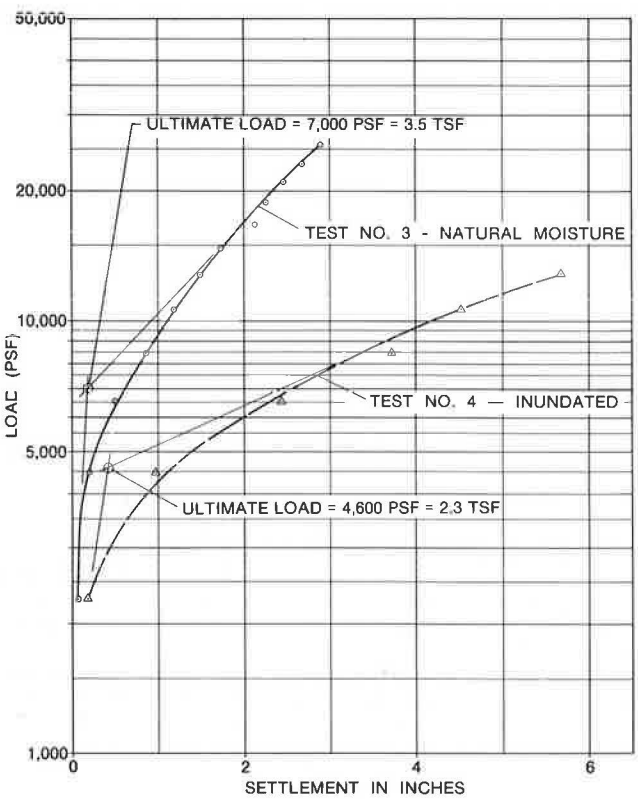

FIGURE 8 Semi-log plot, intersecting tangents method of interpretation—natural ground.

where for the case of a plate loaded at the surface on cohesive soil for local shear conditions

$$N_c = 2/3\ (5.7) = 3.8, \qquad (2)$$

$D_f = 0$, and

$N_\gamma = 0$.

Therefore,

$Q_{ult} - cN_c$ or $3.8c$.

By using the cohesion values determined from the load tests, the allowable bearing pressures were computed, with the following assumptions:

1. Noninundated conditions apply to all footings embedded 4 ft or more below final constructed grade.
2. Inundated conditions apply to all footings embedded less than 4 ft below the final constructed grade.
3. A factor of safety of 3 should be used in developing bearing pressures in order to limit settlement of full-size footings to tolerable amounts, and to provide adequate Factor of Safety against a bearing capacity failure. Evaluation of settlement for expected full-size footings when considering settlement proportional to footing width indicates settlements within reasonable limits.
4. For cohesive soil, since $N_\gamma = 0$, the width of footing is not considered to affect bearing capacity.

The cohesion values were utilized in the basic bearing capacity equation assuming general shear failure for actual spread footings. Allowable bearing pressures were calculated for the predetermined conditions. Average footing depths were used because allowable bearing pressures did not vary significantly for the normal footing depth range expected. The resulting allowable bearing pressures are given in the following table.

Condition	Allowable Bearing Pressure (trillion ft²)
Natural Ground, Noninundated ($D_f \geq 4$ ft)	2.0
Natural Ground, Inundated ($D_f < 4$ ft)	1.3
Compacted Fill, Noninundated ($D_f \geq 4$ ft)	3.0
Compacted Fill, Inundated ($D_f < 4$ ft)	1.9

These values were recommended as maximum allowable bearing pressures for use in the design of soil-bearing spread footings in overburden.

CONCLUSIONS

Based on the performance of 24-in. diameter plate load tests on compacted overburden fill and natural ground overburden, the following conclusions may be drawn:

1. A peak load point was not clearly identifiable on any of the four load settlement curves that would easily define an ultimate load. The Semi-Log, Intersecting Tangents Method of Interpretation was selected as the preferred method for determining ultimate load values.
2. A simplified analysis procedure based on the test conditions was used to evaluate the ultimate

load values and develop conservative allowable bearing pressures for compacted fill and natural overburden materials.

3. The consideration for inundation of shallow spread footings was addressed in the program with the establishment of noninundated conditions to apply to all footings embedded 4 ft or more below final constructed grade. Inundated conditions apply to all footings embedded less than 4 ft below final constructed grade.

4. The proposed allowable bearing pressures developed from the load test program were higher than the locally accepted values. Providing these allowable bearing pressures as design criteria to the project design consultants is expected to result in more uniformity in foundation design and considerable cost savings to this project.

ACKNOWLEDGMENTS

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Prediction of Axial Capacity of Single Piles in Clay Using Effective Stress Analyses

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ABSTRACT

This paper contains a description of research conducted on piles in clay. A finite element program was used to compute the state of stress in the soil around piles. The formulation accounts for the effects of pile installation and soil consolidation, updating the effective stresses continuously in a step-wise manner. The results of an approximate elastic solution were compared with the finite element solution. The two solutions were used to derive expressions for the effective radial stress after consolidation. A predictive procedure was developed that uses the effective radial stress to calculate the side resistance of piles in clay.

Investigators have attempted to determine the state of stress around piles in clay in order to estimate side frictional capacity (1-11). To this end, the state of stress in the soil is updated from the in situ condition to the conditions immediately following pile installation and soil consolidation, and at pile failure. The purpose of this paper is to describe the results of two computer solutions for the effects of pile installation, and to propose a design procedure for axially loaded piles in saturated clays for bridge foundations and other structures.

CAMFE is an acronym for Cambridge finite element, a program developed at the University of Cambridge (12). It uses a one-dimensional finite element formulation to determine the pore pressure and stress changes that occur during pile installation and soil consolidation. For this investigation, an elasto-plastic model was used to represent the soil. The results obtained from CAMFE for the pore pressure and stress changes resulting from installation are similar to those obtained from a cylindrical cavity expansion theory (13) for an elastic, perfectly plastic material. The consolidation phase of the formulation is completed assuming that water flows outward only radially. Thus, as with the cavity expansion procedure, plane stress conditions exist.