INNOVATIVE FOUNDATION SYSTEM HITS A HOME RUN AT MEMPHIS AUTOZONE PARK

By: Kord J. Wissmann, Ph.D., P.E., Chief Engineer, Geopier Foundation Company
Allen G. Minks, P.E., Principal Engineer, Shannon & Wilson, Inc.

INTRODUCTION

Blues City Baseball, Inc. is revitalizing downtown Memphis, Tennessee with the construction of a new ballpark. AutoZone Park will be the future home of the Memphis Redbirds, an affiliate of the St. Louis Cardinals. Like the Redbird players, the ballpark approaches major-league status with a 5-story stadium, a sunken playing field, a 10,000-person capacity cast-in-place seating bowl structure, dugouts connecting the playing field and basement-level concourse level, and outfield bullpens and light banks (Figure 1). Total construction costs are to be on the order of $46 million. Unsurprisingly, Blues City Baseball desires the most economical foundation system so that more money is available for other amenities such as improved seating and concession facilities.

The construction of AutoZone Park is unique and challenging because of its tall grandstand structure, because the stadium is being constructed near a powerful seismic source zone, and because the stadium is being constructed in an excavation within a deposit of soft loessial soil. Maximum column loads are estimated to be about 750 kips. Foundation soil conditions consist of soft silt and clay (loess) incapable of supporting the applied loads on spread footings without excessive settlements. Rather than designing the structure to be supported by relatively expensive deep foundations such as piles or drilled shafts, project engineers opted for a less expensive intermediate foundation consisting of *Rammed Aggregate Piers™*. Rammed aggregate piers are designed to improve subsurface soil conditions and allow the use of high-bearing pressure shallow spread footings for foundation support. This paper describes the geotechnical challenges, design considerations, results of full-scale modulus load tests, and construction of the innovative rammed aggregate pier foundation system implemented at the site.

GEOTECHNICAL CHALLENGES

The project design team was assembled in the late summer of 1997 when Blues City Baseball commissioned Looney Ricks Kiss Architects (Memphis) to formulate a concept for the Ballpark. Blues City Baseball selected Stanley D. Lindsey Associates (Nashville) to develop structural designs, Beers-Inman, Inc. (Memphis) to act as the project's construction manager, PDR Engineers (Memphis) to be the project civil engineers, and Shannon & Wilson, Inc. (St. Louis) to provide geotechnical services. With the team players selected, the first order of business was determining the team's talent at hitting project curveballs.

As depicted in Figure 2, the geotechnical explorations indicated a subsurface profile characterized in descending order by:

1. Approximately 35 feet of soft to medium stiff loessial silt and clay.
2. Approximately 10 feet of medium dense to very dense sand and gravel.
3. Very dense clayey sand and hard sandy clay (Claiborne formation).

Geologic maps indicate that bedrock is present on the order of 1,000 feet below the ground surface. Groundwater was encountered within installed piezometers at depths ranging between 9 and 16 feet below the average existing ground surface. The combination of the ballpark design and subsurface conditions led to a suite of geotechnical engineering challenges that were pitched at the project engineers. These included:

1. The high groundwater levels at the site and deep excavation grades caused the bottom of the excavation to be below the average measured phreatic surface. This condition, combined with the presence of soft loessial soil would cause difficulty in supporting construction traffic if the cut ground surface was left unimproved. To solve this problem, Beers-Inman installed a series of dewatering wells around the site and placed a filter fabric and a blanket of 4-inch rock over the cut subgrade. The wells were abandoned at the end of excavation. To minimize long-term basement moisture, PDR Engineers designed a permanent passive underdrain system comprised of gravel blankets hydraulically connected to drainage pipes.
2. The presence of deep cuts near adjacent roadways necessitated the construction of a shoring system along Union Avenue and the Third Street Alleyway (Figure 3). The use of tie-back anchors was considered and then abandoned by the project team because of a myriad of known and unknown utility lines below the Memphis roadways. The final design consisted cantilevered soldier piles installed along Union Avenue and a deadman-supported shoring system along the Third Street Alleyway. Both systems were designed to be sufficiently rigid to minimize lateral

deflections within the retained soils in an effort to reduce the potential for utility line interuptions.

3. The site is located near the New Madrid seismic source zone, the strongest seismic zone in the central and eastern United States. A site-specific response spectra analysis was performed to numerically account for the influences of the soil profile on representative design-level earthquakes. The results of this analysis allowed for a significant reduction in design-level spectral accelerations for structural components with natural periods greater than about 0.5 seconds (Figure 4).

Although the challenges listed above were of great concern, the single greatest geotechnical challenge faced by the project engineers was the selection of the stadium foundation system. The results of settlement analyses indicated that conventional shallow foundations would settle excessively when loaded by the relatively heavy structural columns. Deep foundations were considered to be an expensive option for the loading and subsurface conditions. After careful consideration, an intermediate foundation system consisting of rammed aggregate piers was selected for foundation support.

RAMMED AGGREGATE PIERS

The selected rammed aggregate pier foundation system was designed and constructed using a proprietary process held by the *Geopier™* Foundation Company, Inc. The aggregate piers were constructed in February of 1999 by augering 30-inch diameter holes to depths ranging between 7 feet and 15 feet below the footing bottoms, placing controlled lifts of aggregate stone within the cavities, and compacting the stone using a specially-designed high-energy tamper. The first lift of stone was forced into the soil thus forming a bottom bulb of stone located below the bottom of the excavated shaft (Figure 5). The bottom bulb effectively extends the design length of the Geopier element by one pier diameter. The piers were completed by placing additional lifts of stone over the bottom bulb and compacting the stone with the Geopier tamper. During compaction, the beveled shape of the tamper forces stone laterally into the sidewall of the excavated cavity. This action increases the lateral stress in the matrix soil and provides additional stiffening effects (Figure 5). A sufficient number of piers were installed to cover 30 to 35 percent of the gross area of the overlying foundation elements. Spread footings, with an allowable composite bearing pressure of 5,500 psf, were then constructed over the rammed aggregate piers.

As shown on Figure 2, the tips of the majority of the installed aggregate piers do not extend completely through the soft loess and therefore are not considered to be end-bearing elements. Rather, the piers are designed to improve the overall modulus of the subsurface soils at depths in which footing-induced stresses are the highest, thereby limiting long-term foundation settlements to one inch or less. The design methodology used to estimate Geopier foundation settlements consists of summing the settlements computed within the upper aggregate pier-enhanced zone and the lower non-reinforced zone. Upper zone calculations are described in detail elsewhere (Lawton and Fox 1994 and Lawton et al. 1994) and are summarized herein for completeness:

1. The footing is assumed to be perfectly rigid relative to the foundation materials. Thus, the stresses applied to the composite foundation materials depend on their relative stiffnesses (R_s) and area coverage. The total downward force (Q) on the footing, which may be expressed as the product of composite stress (q) and footing area (A), is resisted by a total upward force in the rammed aggregate piers (Q_g) and soil (Q_s) materials:

$$Q = q\,A = Q_g + Qs = q_g\,A_g + q_s\,A_s \quad , \tag{1}$$

where q_g is the stress at the top of the rammed aggregate pier elements, A_g is the area of the pier elements below the footing, q_s is the vertical stress within the matrix soil below the footing, and A_s is the area of the matrix soil in contact with the bottom of the footing.

2. Because the footing is rigid compared to the bearing materials, the settlement of the pier will equal the settlement of the matrix soil. The settlement of the foundation (s) can be written in terms of aggregate pier stress and aggregate modulus of subgrade reaction (k_g) or, equivalently, in terms of the matrix soil stress and matrix soil modulus of subgrade reaction (k_s):

$$s = q_g / k_g = q_s / k_s \quad . \tag{2}$$

3. Equation 2 can be rewritten to express the matrix soil stress in terms of the aggregate pier stress and the ratio of the pier and matrix soil modulus values (R_s):

$$q_s = q_g\,(k_s / k_g) = q_g / (k_g / k_s) = q_g / R_s \,. \tag{3}$$

4. Combining Equations 1 and 3 and defining area ratio (R_a) as the ratio of A_g to A:

$$q \;= \{q_g\,A_g / A + q_g\,A_s /(A\,R_s)\} = [q_g\,R_a \;+ q_g\,(1 - R_a) / R_s\} \;=$$

$$= \{q_g\,[R_a \;+ 1/R_s - R_a/R_s\,\} = \{q_g\,[R_a\,R_s + 1 - R_a] / R_s\,\} \,. \tag{4}$$

5. Rewriting q_g in terms of q:

$$q_g = \{q R_s / [R_a R_s + 1 - R_a]\}. \qquad (5)$$

6. Upper-zone settlements are computed using Equations 2 and 5 which depend on the applied composite footing stress, the relative stiffness of the aggregate pier and soil materials, the area ratio of the aggregate pier elements, and the aggregate pier modulus of subgrade reaction.

Estimates of settlements in the lower zone material below the bottom of the aggregate pier bulb were computed for project footings using conventional consolidation analysis well described in the literature (Terzaghi and Peck 1967) combined with data interpreted from the results of oedometer tests. The analysis included the assumptions that the loess was normally consolidated at the time of the investigation, the loess was overconsolidated by the removal of the excavated soil at the start of foundation construction, and the lower zone footing-induced stress increases could be estimated using solutions for a footing supported by an elastic half-space. Design calculations for footing D4 are shown in Figure 6 to provide an example of the settlement calculations implemented for design.

RESULTS OF MODULUS LOAD TESTS

To verify the assumed modulus values used for the aggregate piers, two full-scale aggregate pier modulus tests were conducted prior to construction. The tests were performed by placing steel plates over the full cross-sectional area of an installed aggregate pier element and then applying pressure in gradual increments. The maximum applied stress corresponded to 150 percent of the design stress computed at the top of the aggregate pier elements. Test results, as shown on Figure 7, indicate that top of pier deflections ranged between 0.35 inches and 0.52 inches when subjected to 100 percent of the design stress (95 psi). These values were used to compute aggregate pier modulus values of 95 psi / 0.52 inches = 180 pci and 95 psi / 0.35 inches = 270 pci. The lower modulus value corresponded to an aggregate pier constructed with lift thicknesses exceeding specifications by as much as 50 percent. The measured values confirmed or exceeded the design-level modulus value of 180 pci.

During aggregate pier modulus testing, one of the test piers was instrumented with two Geokon™ pressure plates to verify the assumed stress distribution within the aggregate pier elements. One pressure plate was installed near the top of the pier so that a stress-arching correction factor could be developed from the data (Shannon & Wilson, Inc. 1998). The other pressure plate was installed at a depth of 5.5 feet below the applied load. Test results indicate that a stress of approximately 47 psi was measured at the lower pressure plate during the 100 percent top-of-pier design load increment (Figure 8). Figure 9 presents the results of the stress readings plotted with depth below the ground surface. Based on a straight-line construction between the two stress measurements, the projected stress at the bottom of the pier is approximately 10 psi (1,440 psf). A straight line construction is justified for conditions in which the aggregate shaft / matrix soil adhesion is constant with depth. The test results suggest that only 10 percent of the top-of-pier applied stress is transferred to soil below the bottom of the pier. Similarly, only 26 percent of the composite footing applied stress (5,500 psf) is transferred to soil below the tip of the pier. Because this value correlates very well with a Westergard stress influence factor of approximately 0.25 used for a typical 11.25-foot square footing, the criterion of minimizing excessive bottom-of-pier stress levels was met.

AGGREGATE PIER CONSTRUCTION

A total of 1,207 Geopier elements were constructed for the Ballpark foundation in the month of February 1999. Aggregate pier elements were installed with two crews each averaging about 40 pier installations per working day.

QUALITY ASSURANCE

Shannon and Wilson engineers provided full-time quality assurance by inspecting each pier hole and monitoring aggregate stone compaction. Field engineers made adjustments to the aggregate pier shaft in areas of the site where undocumented fill soils were encountered.

THE REST OF THE LEAGUE

To-date, five structures in the Memphis area have been founded on rammed aggregate piers installed by Geopier Foundation Company, Inc. and monitored by project geotechnical consultants. Rammed aggregate pier-supported structures in the city of Memphis include: the 15-story Peabody Place Office Tower and associated parking deck, the Ronald McDonald House, and a 5-story Hampton Inn currently under construction. Rammed aggregate pier construction activities at these sites were monitored by Professional Services Industries, Inc. (Memphis).

SUMMARY AND CONCLUSIONS

AutoZone Park, the future home of the Memphis Redbirds, is being supported by an innovative intermediate foundation system consisting of rammed short aggregate piers. This intermediate foundation solution was selected by the project engineers because a conventional shallow foundation system was thought to be inadequate for controlling foundation settlement, while deep foundations, consisting of piles or drilled shafts, were thought to be overly expensive for this site.

The rammed aggregate piers are 30 inches in diameter and extend 7 feet to 15 feet below the footing bottoms. The piers do not extend to the underlying dense sand layer but, rather, are terminated within the compressible loess stratum. They are designed to limit foundation settlements to an inch or less. Spread and strip footings placed above the rammed aggregate piers are designed for a composite bearing pressure of 5,500 psf. Assumed design parameters were verified by field modulus tests. The implementation of the rammed aggregate pier system is estimated to have achieved cost savings of greater than 20 percent of the foundation costs relative to auger-cast piling. These savings are planned to be re-invested into other, more visible, features of the ballpark.

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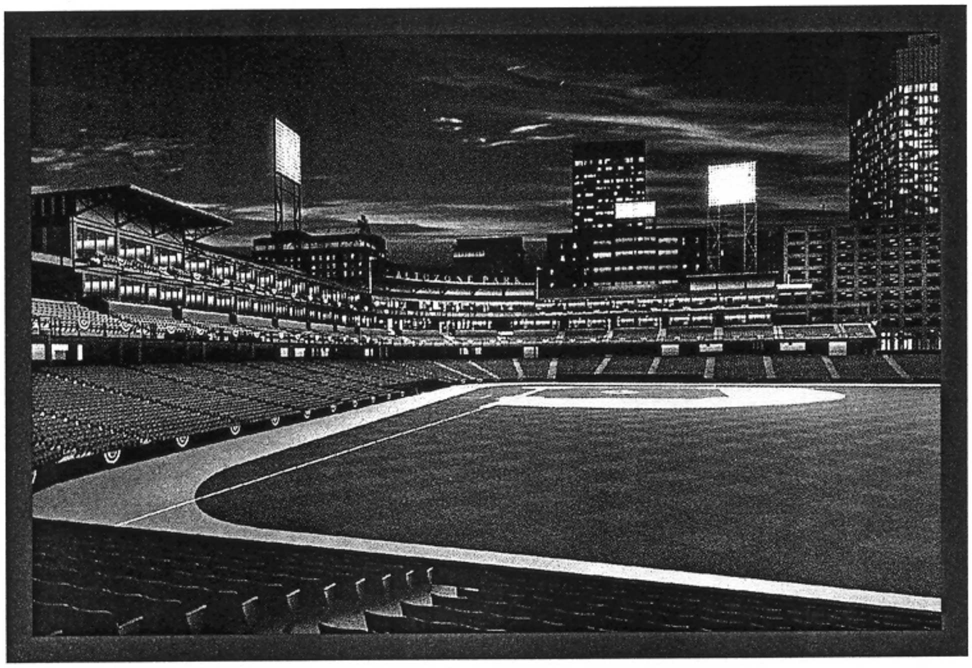
3. Shannon & Wilson, Inc., 1997, "Geotechnical Engineering Report, AAA Baseball Stadium & Parking Garage, Memphis, Tennessee," a report prepared for HBG Management dated October 15, 1997.

4. Shannon & Wilson, Inc., 1998, "Geopier Installations and Load Tests, AAA Baseball Stadium, Memphis, Tennessee," a report prepared for Blues City Baseball, LLC dated July 30, 1998.

5. Terzaghi, K., and R.B. Peck, 1967, Soil Mechanics in Engineering Practice, John Wiley and Sons, New York.



MEMPHIS BALLPARK *Photo Courtesy of Looney, Ricks, Kiss, Inc.*

FIGURE 1 ARTIST RENDERING OF AAA BALLPARK

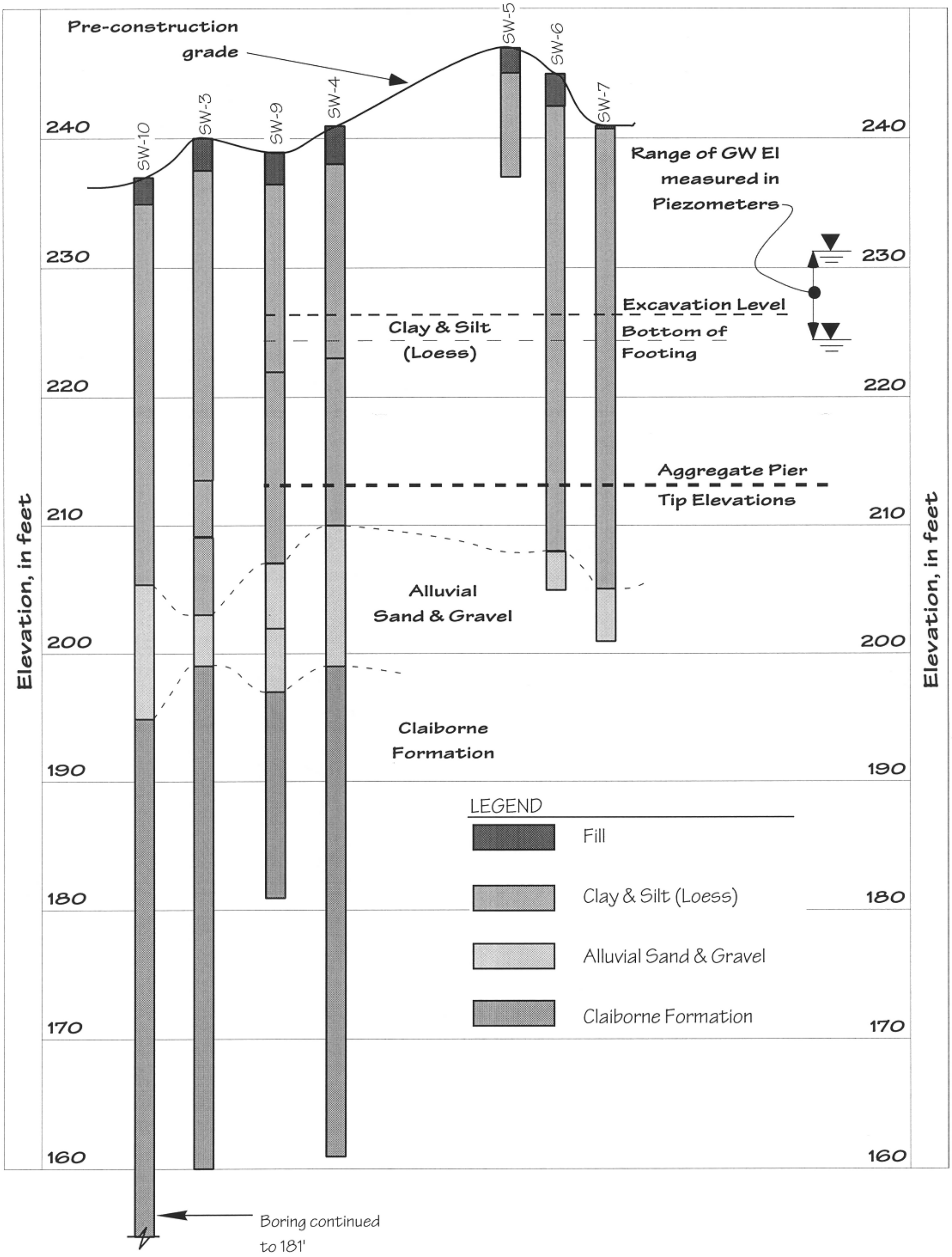


FIGURE 2 GENERALIZED SUBSURFACE PROFILE

FIGURE 3 PHOTOGRAPH OF SHORING SYSTEM

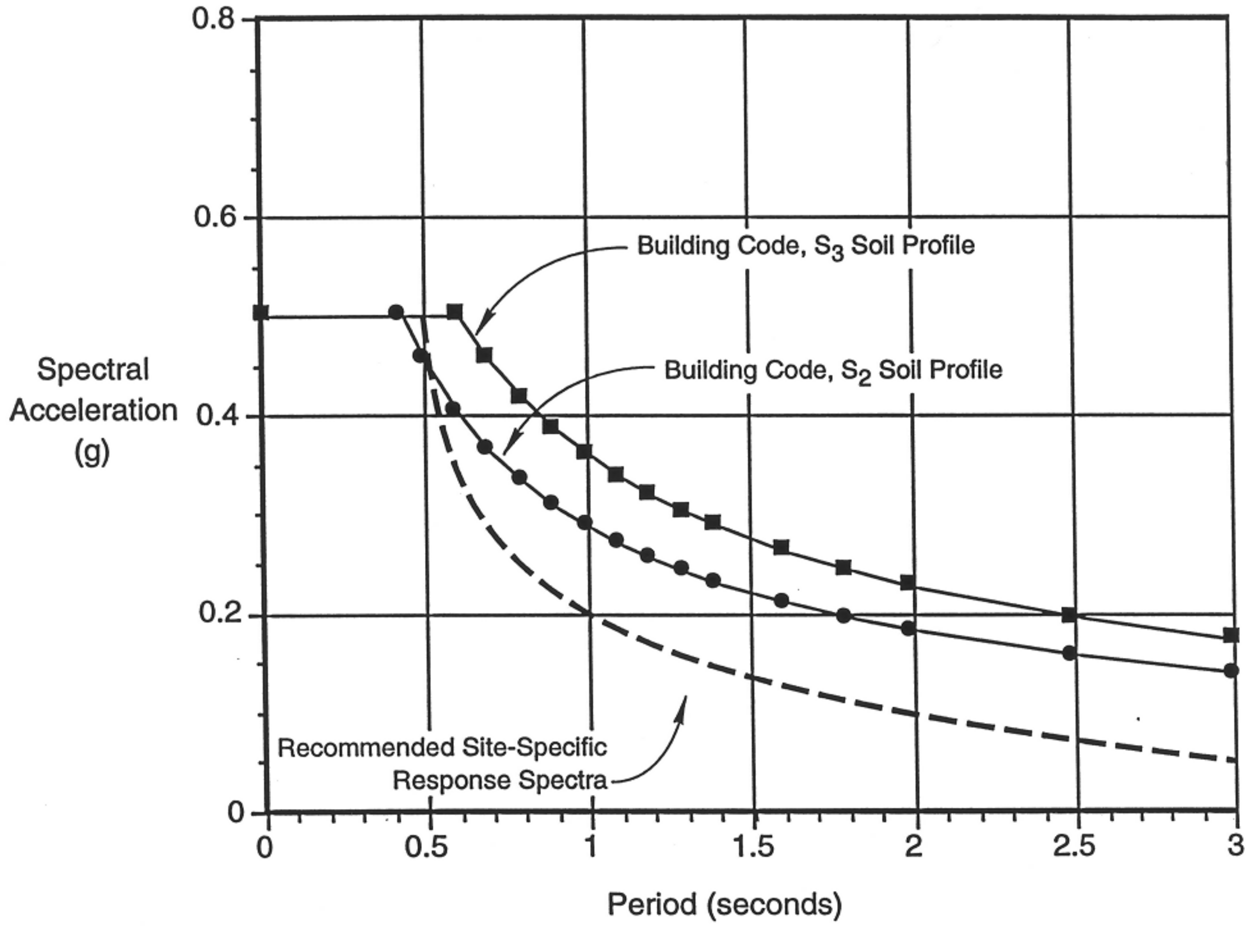


FIGURE 4 COMPARISONS OF RESPONSE SPECTRA

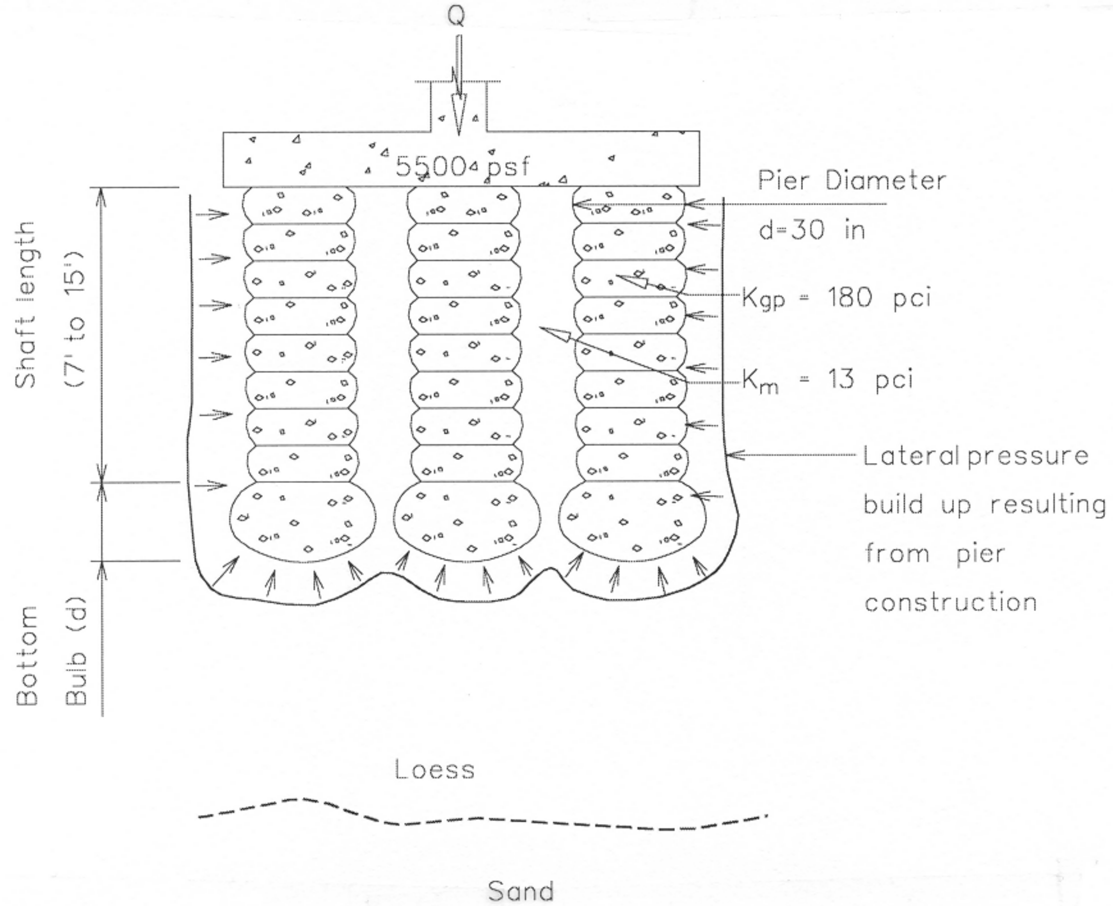


FIGURE 5 RAMMED AGGREGATE PIERS

Settlement Calculations

Design composite bearing pressure, $q = 5{,}500$ psf.

Aggregate pier area ratio, $R_a = A_g / A = 0.35$.

Assumed aggregate pier subgrade modulus, $k_g = 180$ pci.

Assumed matrix soil subgrade modulus, $k_s = 2{,}000$ psf / 1 inch settlement = 13.9 pci.

Stiffness ratio, $R_s = k_g / k_s = 180/13.9 = 12.9$.

Aggregate pier stress, $q_g = q\,R_s / (R_s\,R_a - R_a + 1) = 13{,}737$ psf.

Upper-zone settlement = q_g / k_g = 13,737 psf / 180 pci = 0.53 inch.

Lower-zone settlement (from one-dimensional consolidation analysis) = 0.32 inch.

Total settlement = 0.53 inch + 0.32 inch = 0.85 inch.

FIGURE 6 EXAMPLE DESIGN CALCULATIONS

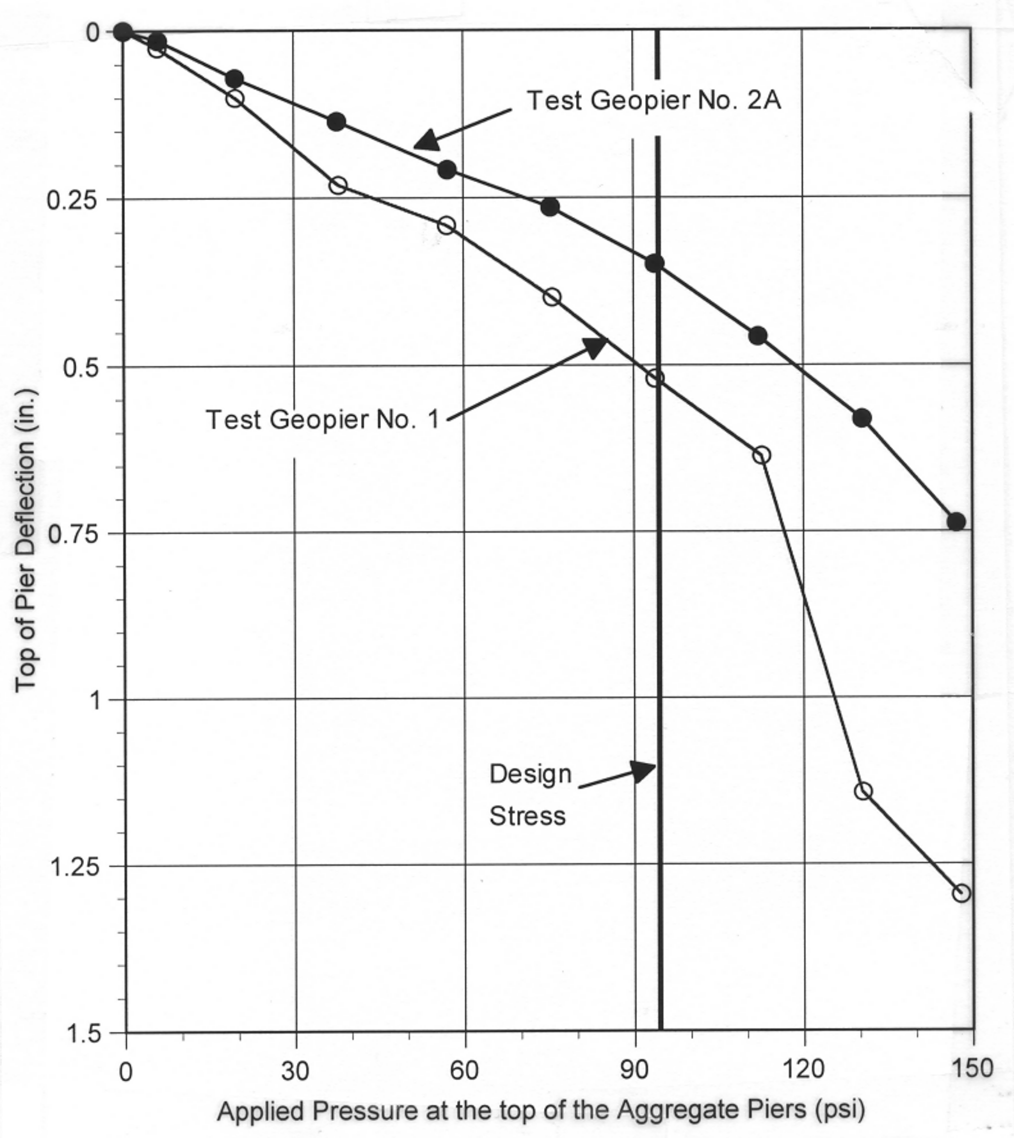


FIGURE 7 MEASURED DEFLECTIONS DURING LOAD TESTING

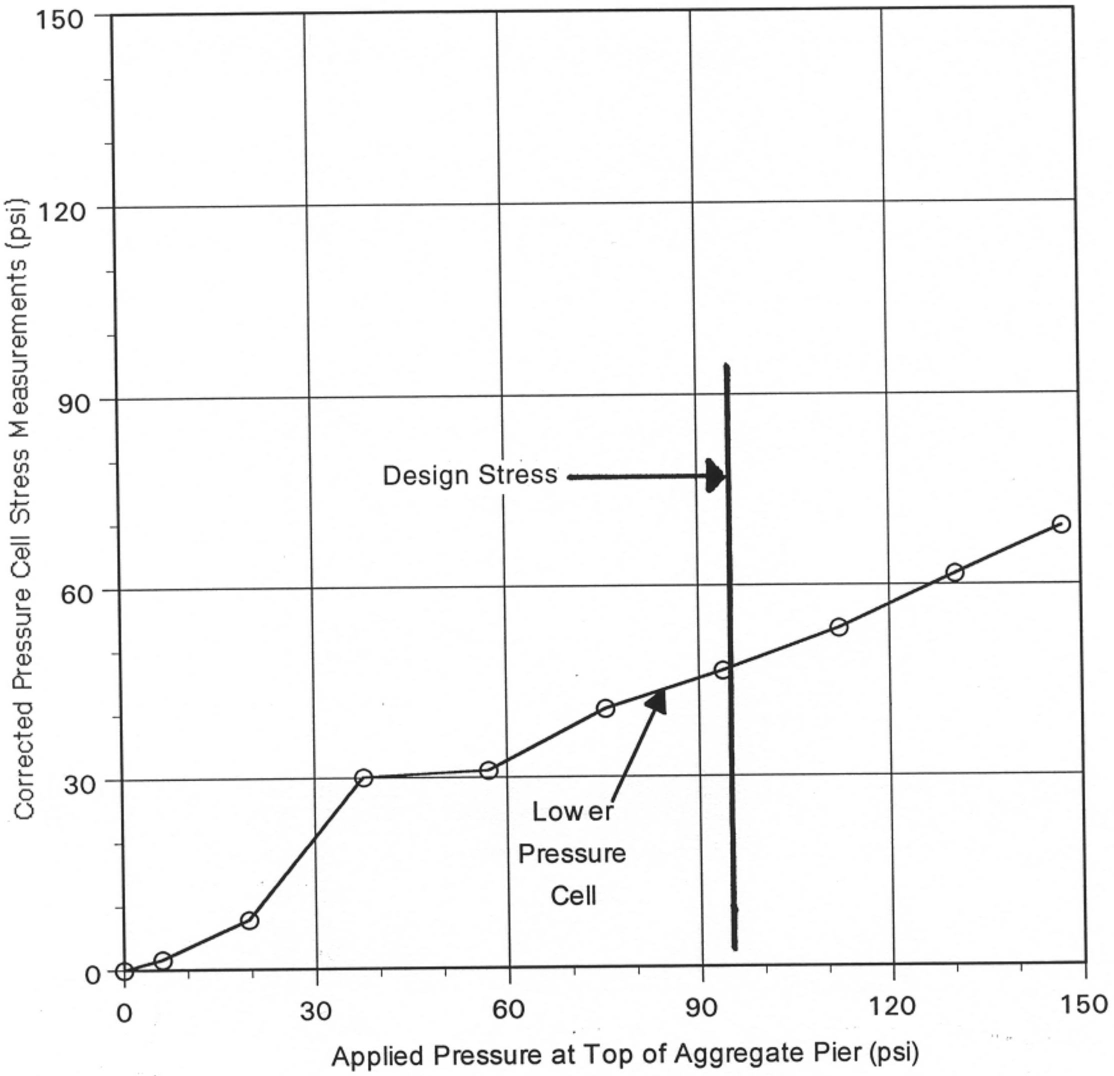


FIGURE 8 CORRECTED LOWER PRESSURE CELL MEASUREMENTS

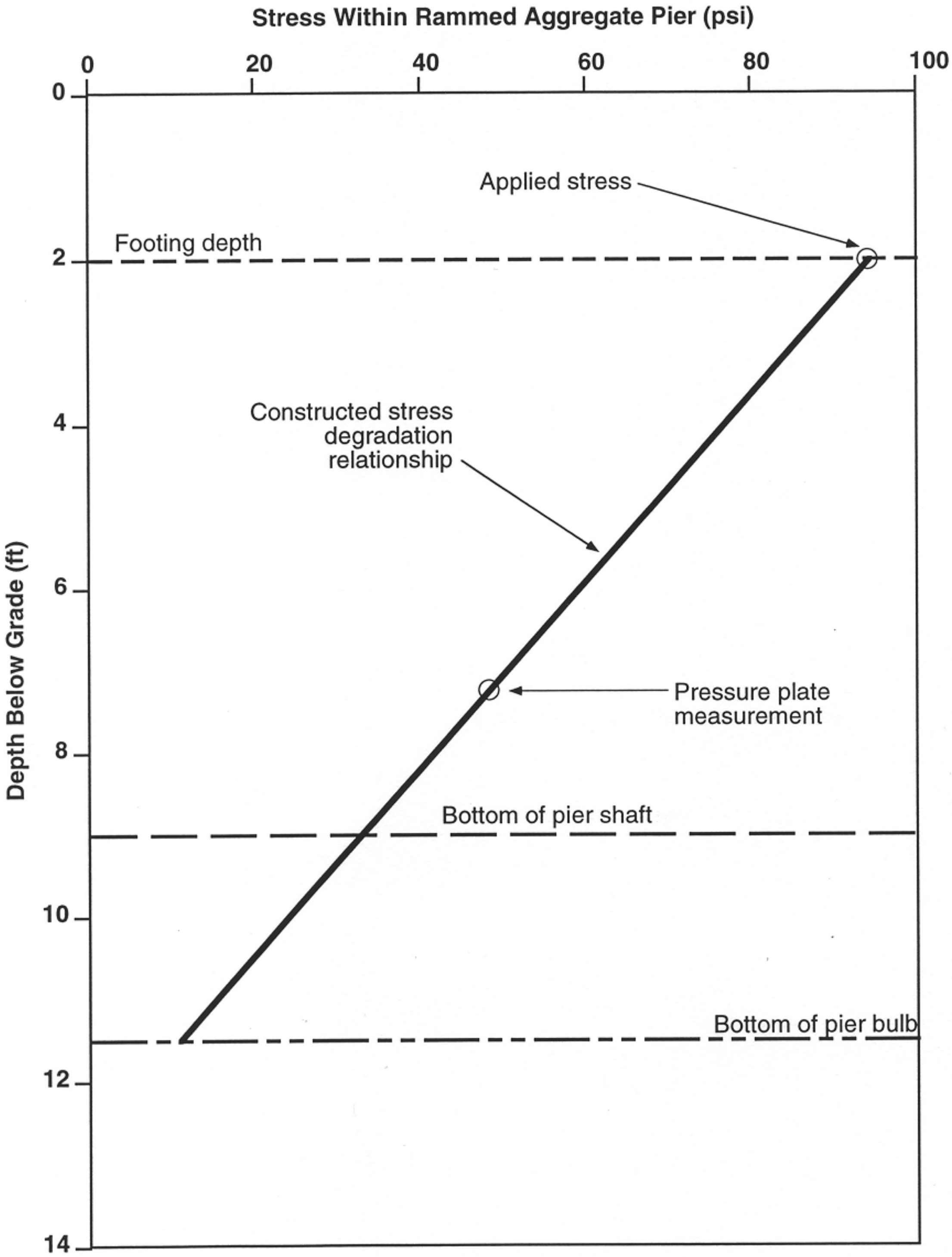


FIGURE 9 AGGREGATE PIER STRESS DEGRADATION