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• Journal of the Boston Society of Civil Engineers Section/ASCE

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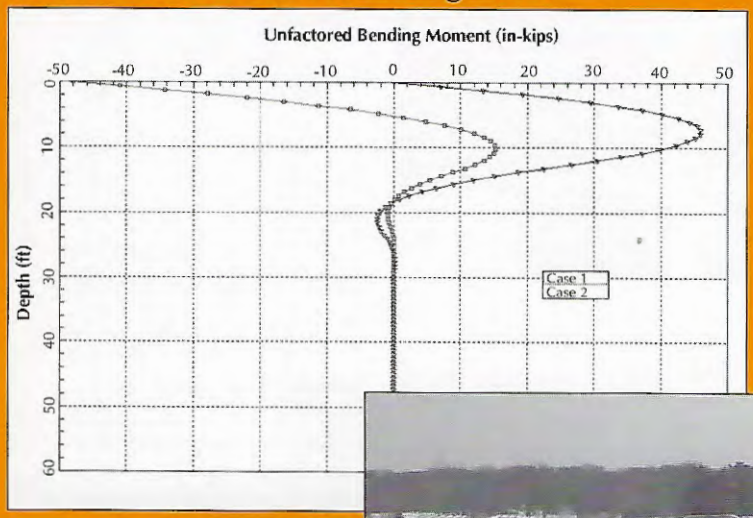
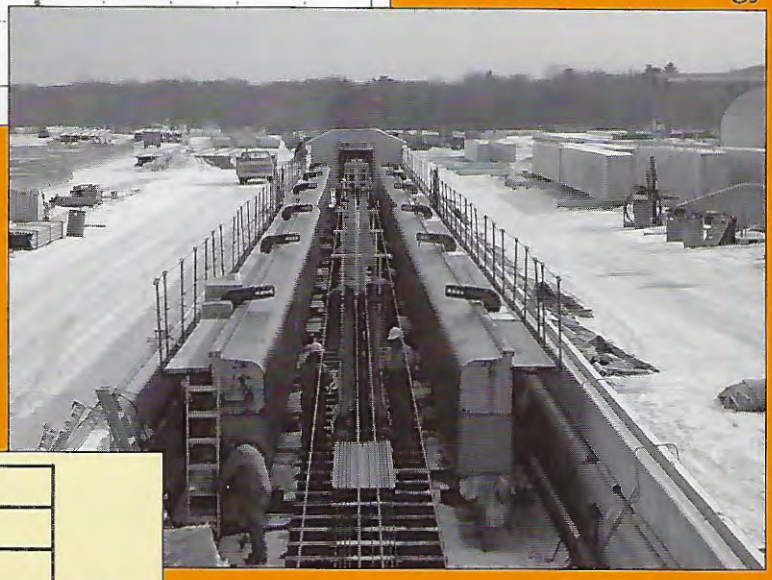
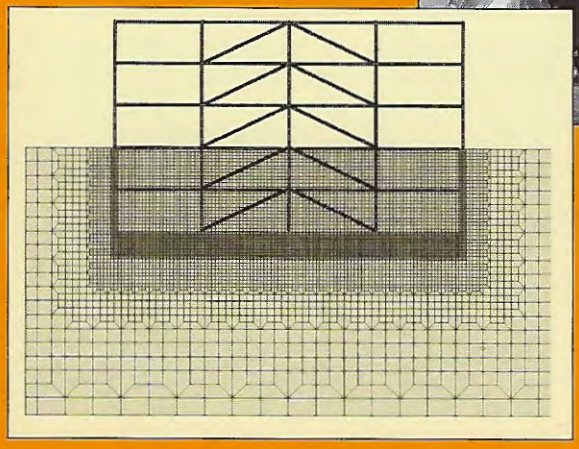
Soil-Structure Interaction for Geotechnical & Structural Engineers



Using Spliced Bulb-Tee Girder Technology



Modeling the Effects of Soil-Structure Interaction on a Tall Building



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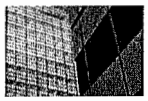
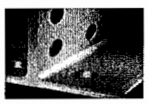

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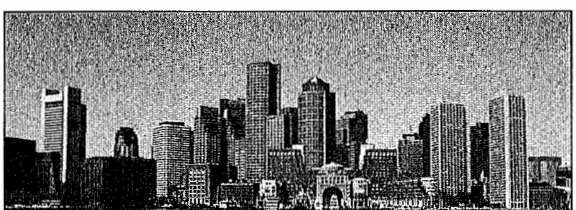

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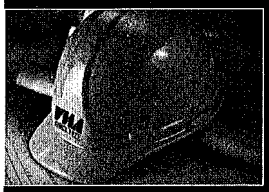

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The Role of Soil-Structure Interaction for Geotechnical & Structural Engineers

The application of soil-structure interaction has been fostered by the development of special analytical tools that will need further expansion in the future.

LYMON C. REESE

Soil-structure interaction (SSI) is the prediction of the response of soil to the loading of a structure as a function of deflection and the corresponding response of the structure. The soil may be fine-grained, coarse-grained or rock. The structure may be a mat, a footing, a caisson or a pile. The loading may be seismic, dynamic, short-term, sustained or cyclic.

As an example of SSI, the behavior of an axially loaded pile is considered. After a given time in a specified soil, under a given value of axial load, a computation can be made to show the soil resistance along the length of a given type of pile, the axial compression in the pile and, consequently, the downward movement of the pile, point by point. Figure 1 shows a model of this problem. Figure 1a shows an axially loaded pile with distribution of side resistance and end bearing. Figure 1b shows that pile replaced by a deformable member and with the soil replaced by non-linear mechanisms.

A typical set of load-transfer mechanisms is shown in Figure 1c. The soil is replaced by a set of springs and sliding blocks, multiple mechanisms for side resistance and a separate one for end bearing. Some have stated that the discrete springs (the Winkler assumption) violate the condition of continuity, but continuity can readily be satisfied by adjusting the springs if the interaction between points along the pile is known. The load-displacement functions for the soil have been developed from field tests of instrumented piles, where continuity was satisfied.

Integration of the unit values yields the load carried in skin friction, in end bearing, and the movement of the top of the pile. The

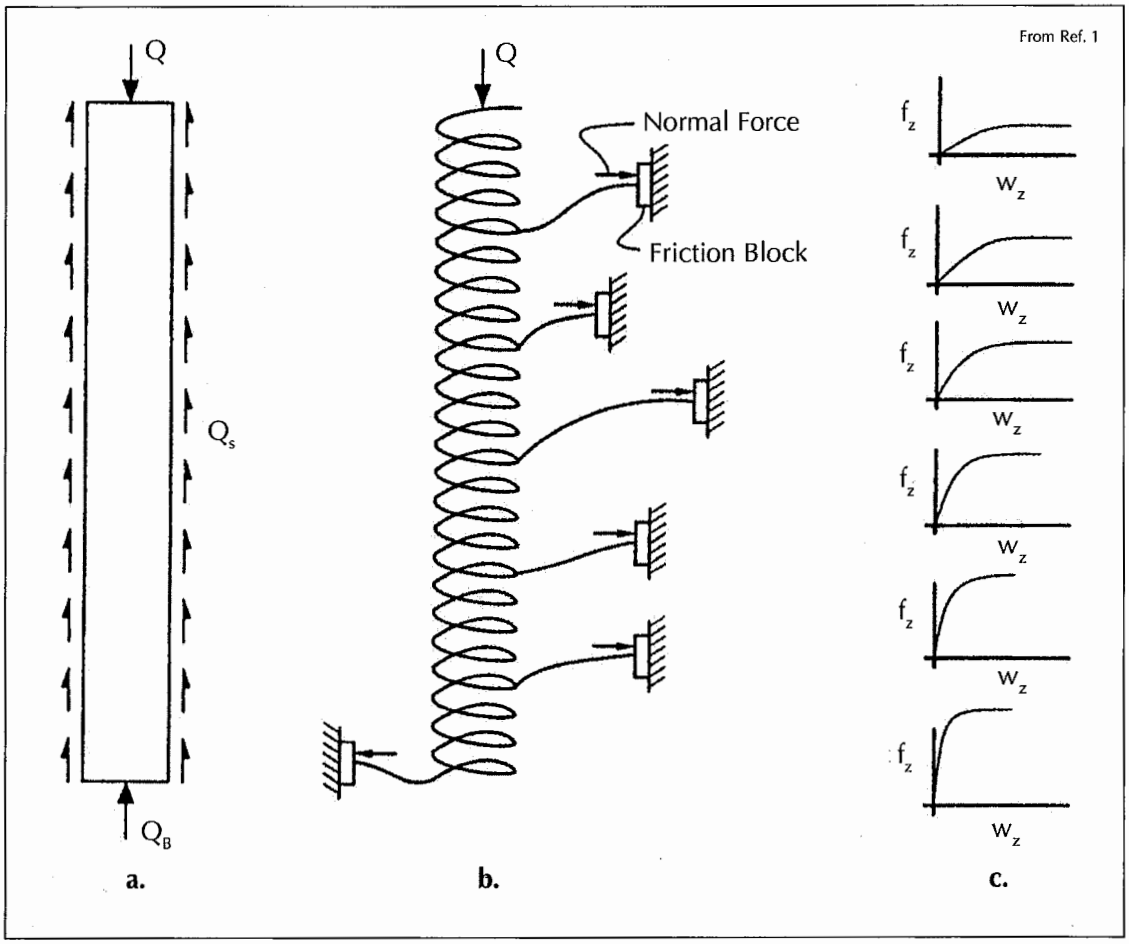


FIGURE 1. A model of a pile under axial loading.

same information can readily be computed as the compressive load is augmented. The "failure load" can be defined as the "plunging" of the pile or the load at some specified downward axial movement of the top of the pile. At any stage in the loading, the axial stress in the pile, steel or reinforced concrete can be computed, and if the pile extends above the ground, the load to cause buckling can be found. Soil-structure interaction can provide even more information than is obtained when a fully instrumented pile is tested under axial load, and current research can improve the reliability of the method.

Referring to the model in Figure 1, the differential equation can be obtained by considering an element from the pile as shown in Figure 2. The unit strain is:

$$\frac{dw_z}{dz} = \frac{Q_z}{EA} \tag{1}$$

where:

E = modulus of elasticity of the pile material;
A = cross-sectional area of the pile;
Q_z = load in the shaft at point z; and
w_z = movement of the pile at point z.

Differentiating Equation 1 with respect to Q_z, and noting that Q_z is equal to the stress distributed at the differential section f_z times the area of the section Cdz where C is equal to the circumference, the differential equation to be solved becomes:

$$EA\frac{d^2w_z}{dz^2} = f_zC \tag{2}$$

Referring to Figure 3, Equation 2, in finite-difference form, is written as Equation 3 for point m along the pile:

$$(w_z)_{m-1}-2(w_z)_m+(w_z)_{m+1} = h^2\eta_m\beta_m w_m \quad (3)$$

Load transfer f_z is expressed as some function β_m times the pile movement w_m, with C/EA equal to η_m, which will be a constant in many cases. However, the parameter β_m will usually vary with depth.

If load-transfer curves are at hand (such as those shown in Figure 1c), then Equation 3 may be solved to obtain movements at every designated point along the pile, given the applied load. If w_m is inconsistent with the value assumed in estimating β_m, a revision is made and the solution repeated. By varying the load and repeating the solution, a curve can be found to show the load versus settlement (or uplift) at the top of the pile. Other approaches can be developed for solving the differential equation, but iteration is always required because load-transfer depends on the movement of the elastic piles and movement depends on load transfer.

Whitaker and Cooke reported the results of an axial-load test of a bored pile with the geometry shown in Figure 4a.[2] The pile was installed in clay soil by the dry method of construction. The soil was an overconsolidated clay with an undrained shear strength of 76.6 kPa at the groundline (see Figure 4b), and with a value of 153.2 kPa at a depth of 15.2 meters; the water table was well below the ground surface and the unit weight of the soil was assumed to be 19.6 kN/m^3.

A computation was made for the load-settlement curve using the computer program SHAFT. The t-z curves that were employed were developed by a study of a number of load tests where details were reported as shown above.[3] A comparison of the load-settlement curves from experiment and from computation is shown in Figure 4c. The computed curve is slightly conservative but the method of analysis was successful in predicting the non-linearity that was observed in the experiment.

The concept of SSI has been discussed in the past by a number of engineers[4] and now is being extended to include all kinds of ground-supported structures under a variety of loadings at any time after construction. Thus, the scope of the technology of soil-structure interaction is very large. However, all engineers who design foundations are practicing soil-structure interaction to some degree because attention should always be given to the move-

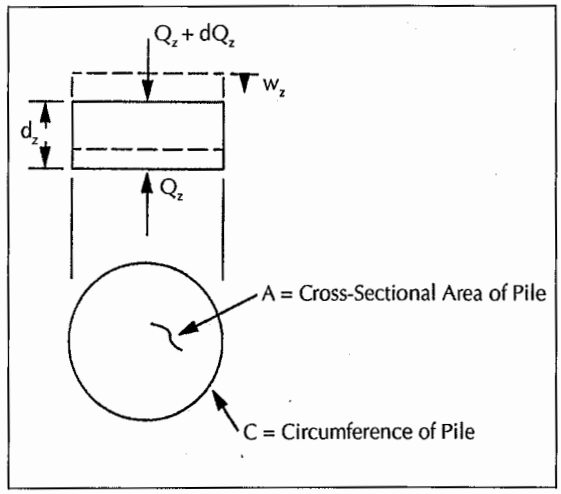


FIGURE 2. An element from an axially loaded pile.

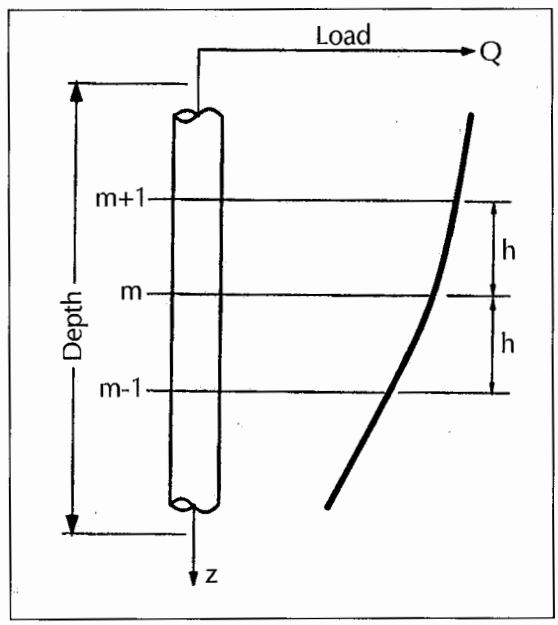


FIGURE 3. Subdivision of an axially loaded pile and a segment of a load-distrbution curve.

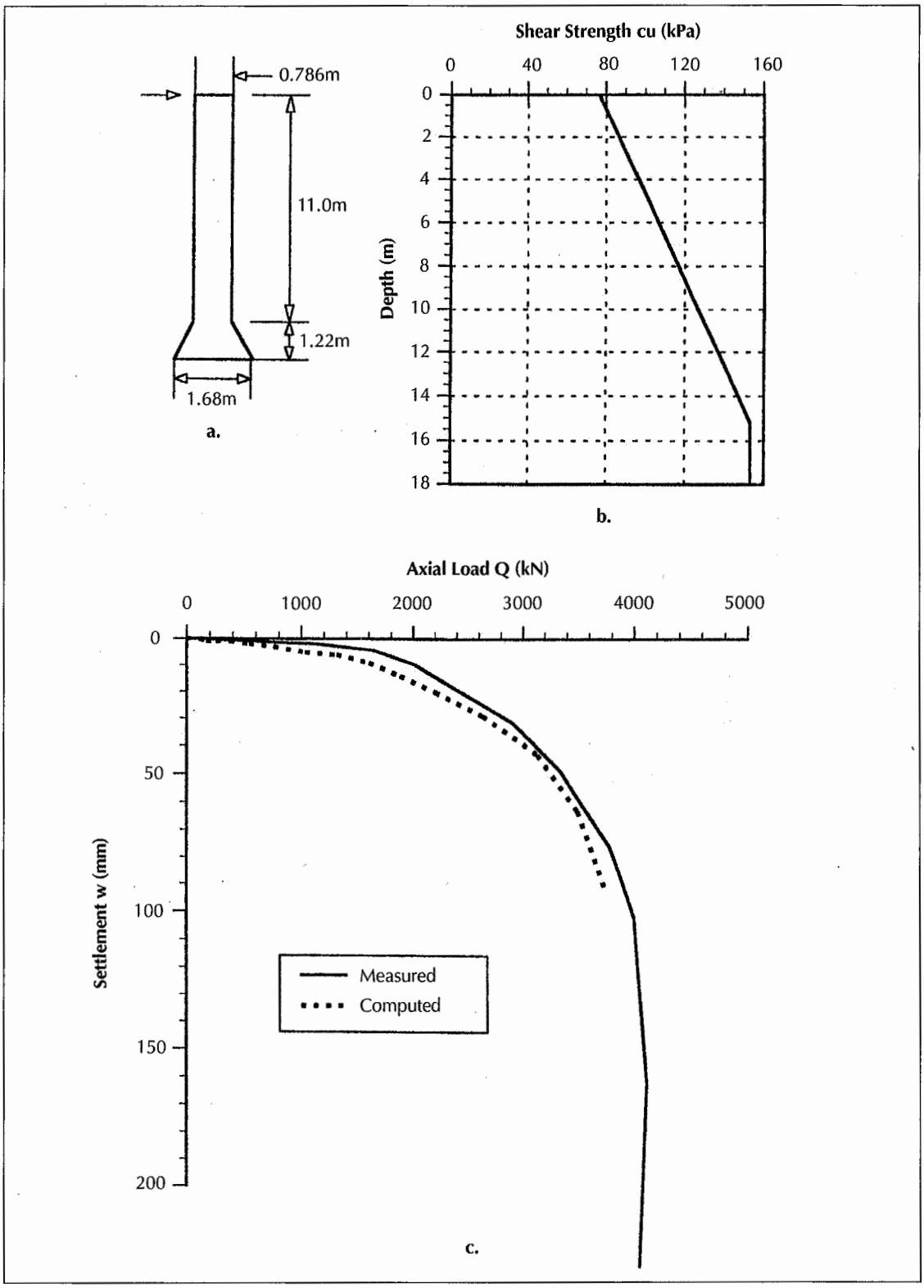


FIGURE 4. Case study of a bored pile under axial loads.

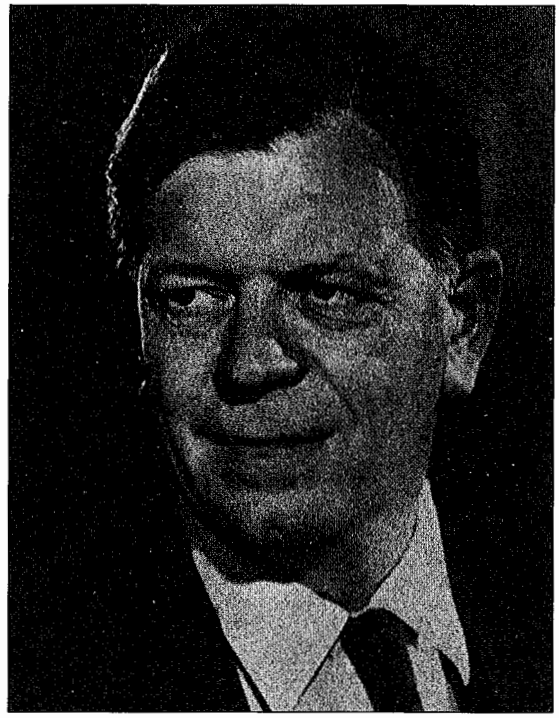
ment of a foundation due to the imposed loadings.

Building Blocks of Soil-Structure Interaction

The listing of a few names is risky because the names of many who have contributed to the technology will be omitted. However, there are a number of persons who cannot be left off any list.

A.W. Skempton. In 1951, a paper by the most significant contributor in the twentieth century to soil mechanics in Britain discussed the load-settlement behavior of a footing on clay.[5] The theory of elasticity provided the basis for Skempton's proposal, but his method was confirmed by comparing analytical results with experimental results. Early researchers in soil-structure interaction used Skempton's method for obtaining the *q-z* curves for piles and drilled shafts and for obtaining insight into the *p-y* curves for piles under lateral loading in clay.

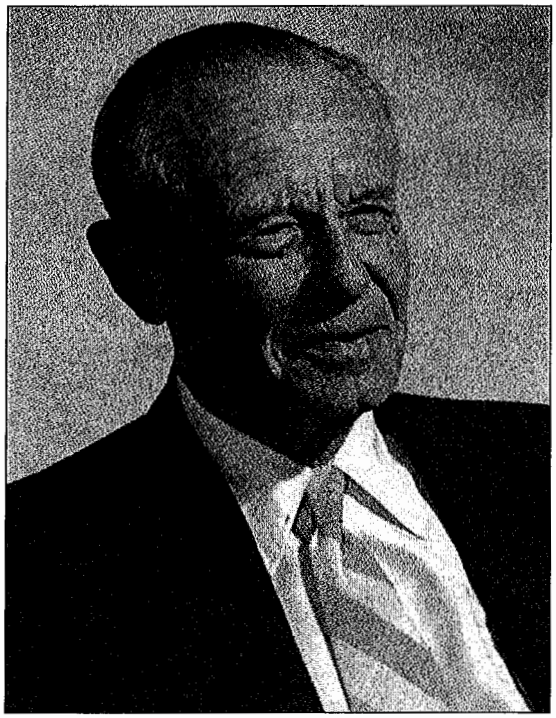
Karl Terzaghi. In 1955, a paper by the "father of soil mechanics" in *Géotechnique* discussed subgrade modulus, gave the differential equation for a pile under lateral loading and presented a discussion of the response of soil to pile deflection.[6] Unfortunately, no experimental evidence was presented to confirm his concepts. The late Professor Raymond Dawson of the University of Texas at Austin, met Terzaghi in 1936 at the First International Conference on Soil Mechanics and Foundation Engineering. Terzaghi visited Austin on many occasions and spoke at all but one of Dawson's Texas Conferences on Soil Mechanics and Foundation Engineering. Terzaghi lectured at the University of Texas during World War II and was in Austin to speak at the eighth Texas Conference in 1956. He visited the site at Lake Austin where Hudson Matlock was performing some of the early tests of piles under lateral loading and remarked that he was not particularly happy with the paper on subgrade modulus and had been pressured to write the paper by many engineers. However, Terzaghi's writing and his evident interest in the Matlock's research were valuable contributions to the idea of soil-structure interaction.



A.W. Skempton.

Arthur Casagrande. Much of soil-structure interaction depends on a detailed and intimate knowledge of the soils at the site. Perhaps no other contributor in the twentieth century did as much as Casagrande to further the knowledge of the behavior of soils under a variety of loadings. During his years of teaching at Harvard University he was responsible more than anyone else in imparting knowledge to a large number of students who then went on to make significant contributions of their own. His close association with the U.S. Army Corps of Engineers, particularly at the Waterways Experiment Station at Vicksburg, Mississippi, and with service on large numbers of consulting boards, led to significant contributions that continue to affect the knowledge and practice of soil mechanics.

John Schmertmann. Contributions of aspects of soil-structure interaction appear in many textbooks and technical articles. An excellent example of a contribution that continues to be useful and important was written by Schmertmann and his co-workers,[7,8] which allows the settlement of a footing to be computed resting on sand that had been tested by



Karl Terzaghi.

Arthur Casagrande.

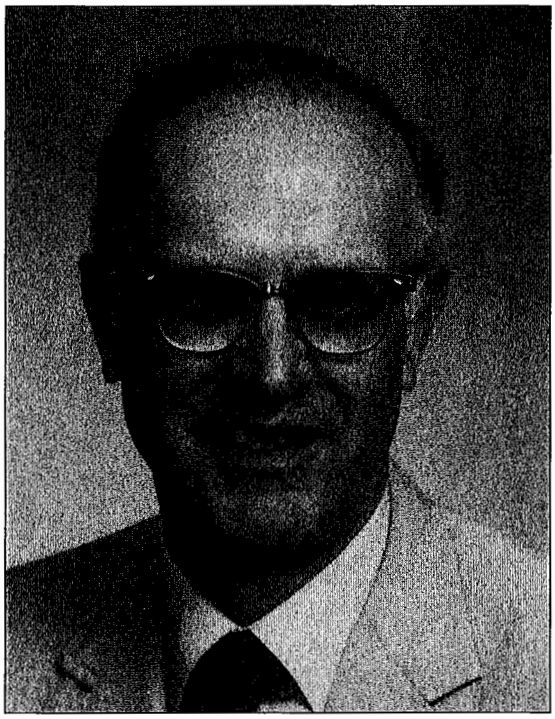
the Dutch cone. The device is being used extensively in parts of the United States and more so in Europe and provides an excellent pattern to the resistance against the penetration of a standard cone. Schmertmann based his method on the results of computations with the finite-element method and provided validation by obtaining excellent agreement between results from analysis and from experiment.

The Oil Industry With Offshore Structures. In the early 1950s, the oil industry ventured into deeper water in the Gulf of Mexico. Earlier platforms had been constructed in the gulf in shallow water and given the name Texas Towers. The so-called Texas Towers had been constructed off the east coast of the United States to support early-warning radar. Some information was available for lateral loading of a platform from wave forces during hurricanes, but information on piles under lateral loading was limited. One of the companies was installing a platform near the Mississippi River delta off the coast of Louisiana and asked two engineering companies to submit designs for piles under lateral loading. Widely divergent answers came in and a company meeting with the two designers revealed that basic information was lacking. Accordingly, a research program was initiated on single piles and pile groups under axial and lateral loading that lasted about two decades. The research continues today on multiple fronts by many investigators under a variety of sponsors. An elevation view of a sketch of the arrangement of piles in the field is shown in Figure 5 and a photograph of a test in progress is shown in Figure 6.

Remote-Reading Strain Gauges & High-Speed Computers. Oil industry engineers favored the testing of prototype piles in the field installed in uniform soil. Fortunately, for the research program on piles, remote-reading strain gauges were well established as research tools that could be used to provide data that defined the response in detail of piles under a variety of loadings. Also, the digital computer had become available. The first used at the University of Texas (an IBM-650) was a rotating-drum machine that used punched cards for input and output. Programming was accomplished by the Bell System and users would go to the third floor of a science build-

ing when time had been reserved for an hour's use of the computer, perhaps at 3 to 4 A.M., to attempt a solution of the non-linear differential equation by difference-equation techniques for the pile under lateral loading. The university rapidly added computing power in a special building but not without some resistance. When a famous mathematics professor was told in the faculty dining room about the new computer, he exploded: "What this campus needs is brains, not another typewriter!"

Skilled Programmers. Much of the progress in soil-structure interaction is due to the work of skilled programmers. Solutions of complex problems are done routinely with convenient input and output. The writer was sitting at a personal computer (PC) at home with input data for a pile under lateral loading. The non-linear differential equation was solved by iteration for multiple runs almost faster than the output could be called up. Page after page of output could have been printed showing deflection, bending moment, shear and soil reaction, and families of curves were available



John Schmertmann.

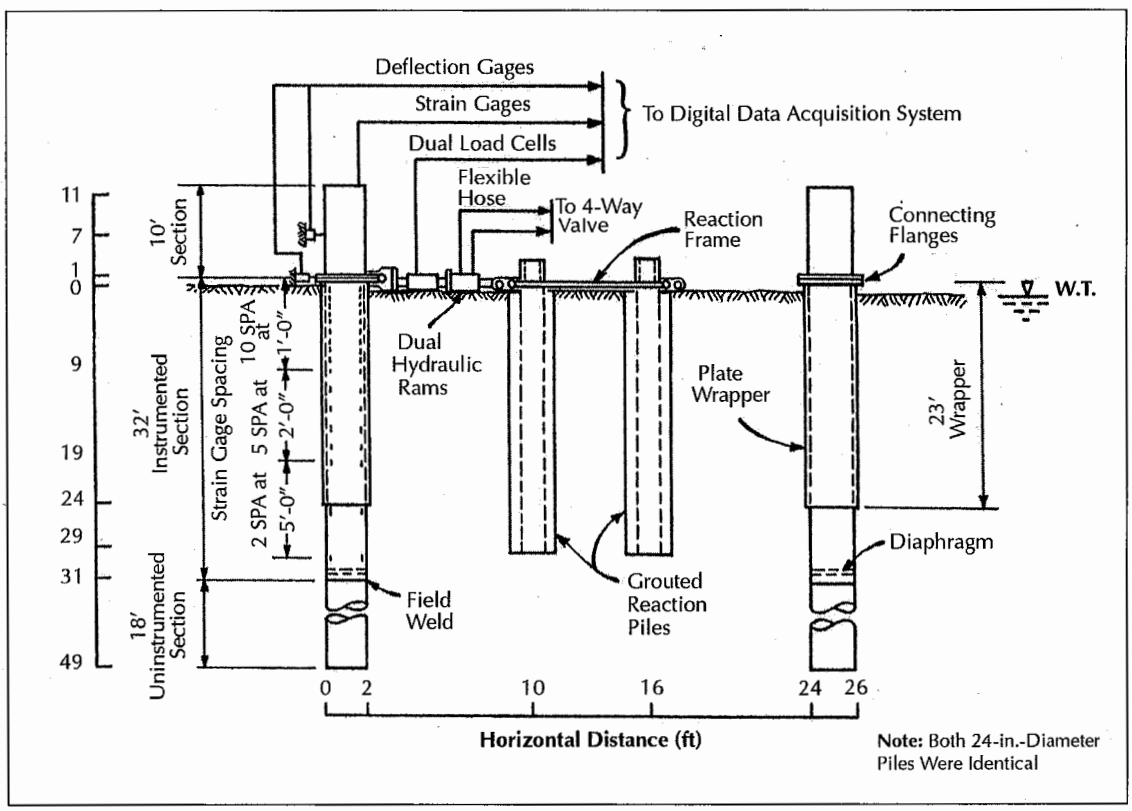


FIGURE 5. The setup for testing 24-inch-diameter pipe piles under lateral loading.

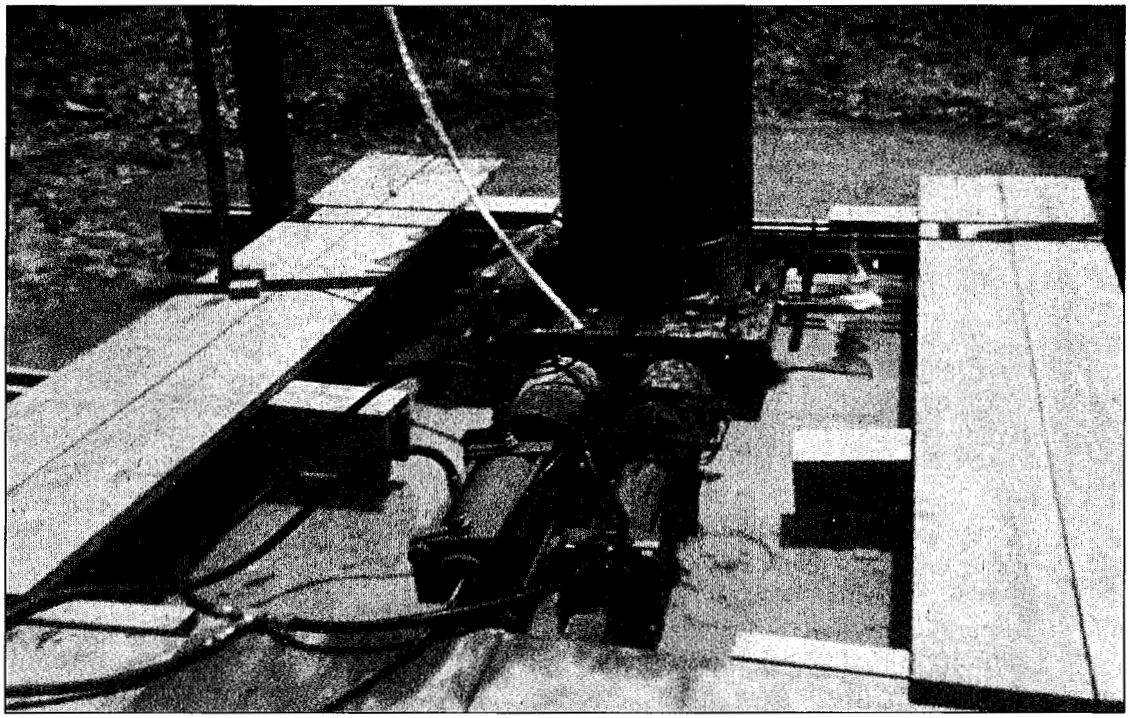


FIGURE 6. A view of hydraulic rams and load cells in a test of piles in overconsolidated clay.

showing the same information in graphical form. The exceptional speed of the solution, due to the coding and to the characteristics of the PC, enables the importance of the parameters in a solution to be evaluated with precision. An engineer at work with a PC is shown in Figure 7.

Significant Achievements

One significant achievement in SSI was determining the response of a pile to axial loading. The loadings may be static, repeated, sustained or dynamic. (Seismic loadings are in a special category and will not be addressed here, even though the concepts of SSI have been applied to designs of structures subjected to earthquakes.)

Other problems tackled include a pile subjected to lateral loading, a pile subjected to torque and the interaction of closely spaced piles in a group. These three topics are important separately but are components of the problem of the behavior of a group of piles subjected to three-dimensional loadings. The finite-element method is applied to the problem of a piled raft, and finally a presentation is made of the response of foundations subjected to dynamic loadings.

These achievements represent substantial progress in SSI but much remains to be done, not only relating to the noted topics but in extending SSI into additional topics.

Isolated Pile Under Lateral Loading. The analysis of the laterally loaded pile by the finite-difference method has been developed extensively by a number of authors since the early 1950s.[9–14] Their work proved the versatility and the theoretical applicability of the finite difference method in dealing with the highly non-linear soil-pile-soil interaction.

The *p-y* method is being used extensively in the United States and elsewhere. To illustrate its use, references are cited from Italy,[15] France,[16] Britain,[17] Australia,[18] and Norway.[19] The method is included in publications of the Federal Highway Administration[20] and the American Petroleum Institute.[21] The publications have guided the design of onshore and offshore pile foundations in the United States and elsewhere.

The laterally loaded pile is modeled as shown in Figure 8. The mechanisms are

shown to represent the soil and depict the soil as a non-linear material. The deformation of an elastic member under axial and lateral loading can be found by solving Equation 4, the standard beam-column equation:

$$\frac{d_2}{dx^2}\left(EI\frac{d^2y}{dx^2}\right)+Q\left(EI\frac{d^2y}{dx^2}\right)-p-W=0 \qquad (4)$$

where:

Q = axial load on the pile;
y = lateral deflection of the pile at point x along the length of the pile;
p = soil resistance per unit length;
EI = flexural stiffness; and
W = distributed load along the length of a portion of the pile.

A physical definition of the soil resistance p is given in Figure 9. Figure 9a shows a profile of a pile that has been installed by driving or by some other method, and shows a thin slice of soil at some depth x_i below the ground surface. The assumption is made that the pile has been installed without bending so that the initial soil stresses at the depth x_i are uniformly distributed, as shown in Figure 9b. If the pile is loaded laterally so that a pile deflection y_i occurs at the depth x_i, the soil stresses will become unbalanced as shown in Figure 9c. Integration of the soil stresses will yield the soil resistance p_i with units of F/L:

$$p_i = E_s y_i \qquad (5)$$

where:

E_s = a parameter with the units, relating pile deflection y and soil reaction p.

It is evident that the soil reaction p will reach a limiting value (and perhaps decrease) with increasing deflection. Furthermore, the soil strength in the general case will vary with depth. Therefore, only in rare cases will E_s, sometimes called the soil modulus, be constant with depth.

The bending stiffness EI of a metal pile will probably be constant for the range of loading of principal interest. However, the EI of a reinforced concrete pile will change with the bending moment. In many designs, it is desirable to reduce the bending stiffness by reducing the wall thickness of a steel-pipe pile or by reducing the number of bars in a reinforced-concrete pile. Thus, in the general case the bending stiffness will not be constant with x nor with y.

In view of the non-linearities of Equation 4, numerical methods must be utilized to obtain a solution. The difference-equation method can be employed with good results. Equation 6 is the differential equation in difference form, where the pile is subdivided as shown in Figure 10 (on page 16):

$$y_{m-2}R_{m-1}+y_{m-1}(-2R_{m-1}-2R_m+Qh^2)+y_m(R_{m-1}+4R_m+R_{m+1}-2Qh^2+k_mh^4)+y_{m+1}(-2R_m-2R_{m+1}+Qh^2)+y_{m+2}R_{m+1}-W_mh^4=0 \qquad (6)$$

where:

$R_m = E_mI_m$
$k_m = Esm$

The pile is subdivided into n increments and $n+1$ equations can be written of the form of Equation 6, yielding $n+5$ unknown deflec-

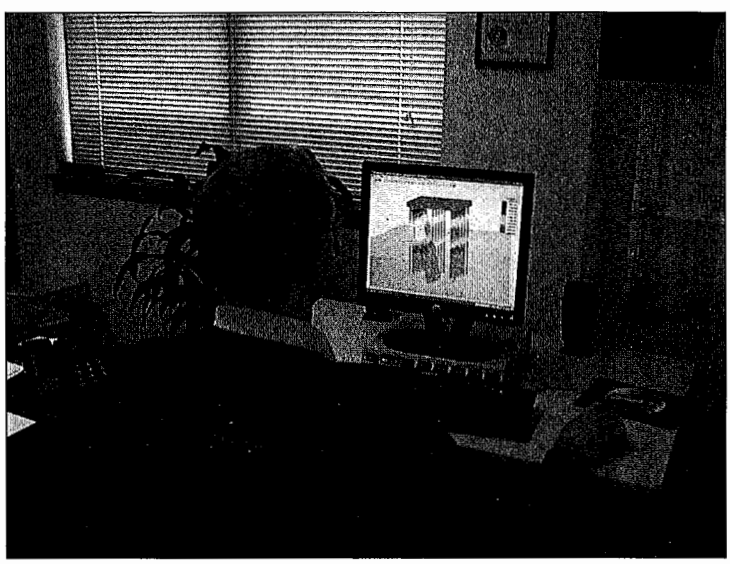


FIGURE 7. Typical activity of an engineer at a personal computer.

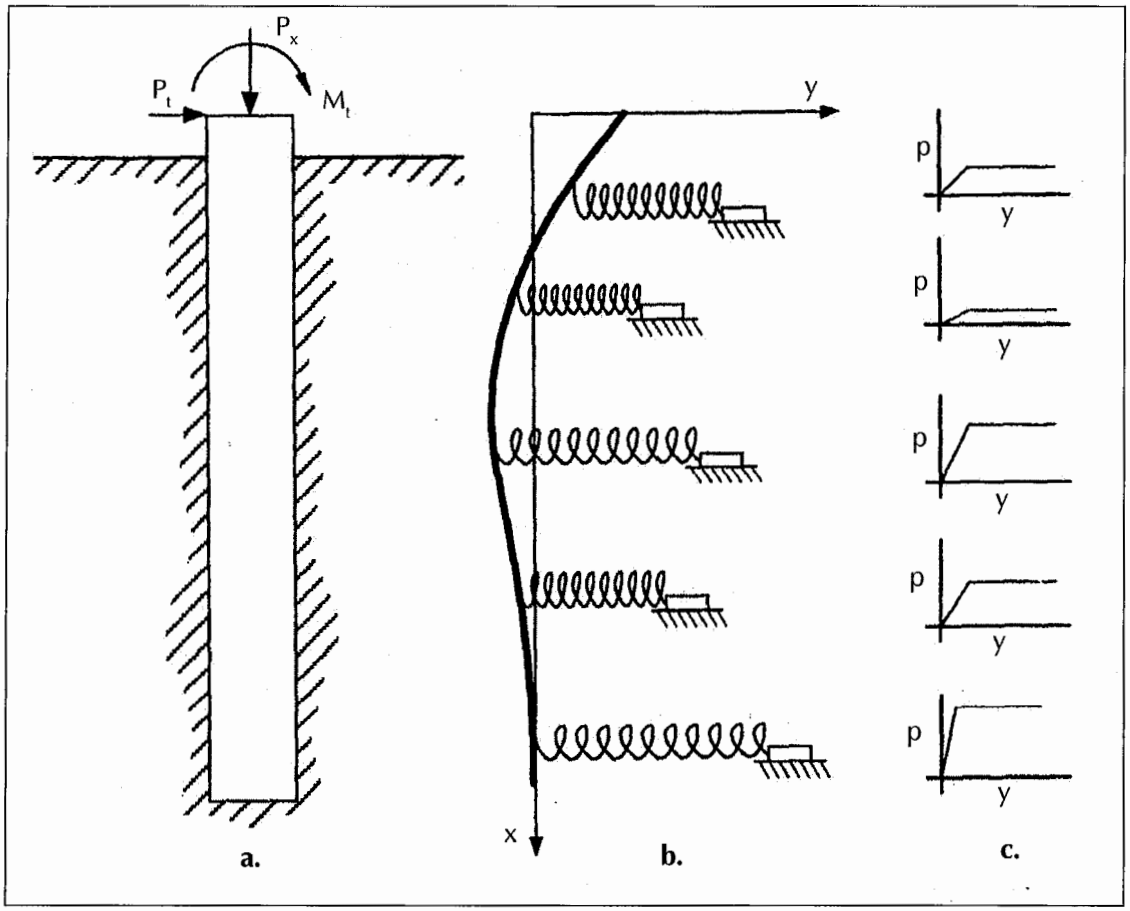


FIGURE 8. A model of a laterally loaded pile.

tions. Two boundary conditions at the bottom of the pile and two at the top of the pile allow for a solution of the $n+5$ equations with selected values of R and k. The value of n and the number of significant figures in y are selected to yield results with appropriate accuracy. The solution of the equations proceeds readily by Gaussian elimination. The value of n ranges from perhaps 50 to 200; on most computers double-precision arithmetic is necessary with about fifteen significant figures.

The solution proceeds as illustrated in Figure 11. Figure 11a shows a pile subjected to a lateral load. Figure 11b shows a family of p-y curves where the curves are in the second and fourth quadrants because soil resistance is opposite in direction to pile deflection. Also in Figure 11b is a dashed line showing the deflection of the pile, either assumed or computed on the basis of an estimated soil response. Figure 11c shows the upper p-y curve enlarged with the pile deflection at that depth represented by the vertical, dashed line. A line is drawn to the soil resistance p corresponding to the deflection y with the slope of the line indicated by the symbol E_s. Figure 11d shows the values of E_s plotted as a function of x. In performing a computation, the computer utilizes the computed values of E_s and iterates until the differences in the deflections for the last two computations are less than a specified tolerance. If desired, bending moment along the pile can be computed during iterations, using the appropriate difference equation, and the value of EI can be computed and varied along the pile with each iteration.

After deflections have been computed, difference equations can be employed to compute rotation, bending moment, shear and soil reaction as a function of x. The number of iter-

ations for a tolerance of 0.00025 millimeters is usually less than 20. A high-speed computer can converge to a solution in less than one second of central-processor time. Thus, if *p-y* curves are available, a solution to a given problem can be obtained with little difficulty.

Soil-response curves have been obtained from several full-scale experiments. The piles were instrumented for the measurement of bending moment as a function of depth. Loads were applied in increments and a bending-moment curve was obtained for each load. Two integrations of each curve yielded pile deflection and two differentiations yielded soil reaction.[22] The cross-plotting of deflection and soil resistance yielded experimental *p-y* curves.

Methods for predicting *p-y* curves have been worked out for soft clay,[12] for stiff clay below the water surface,[13] for stiff clay above the water table[23,24] and for rock.[25] Several authors have made use of reports in the technical literature on instrumented tests and on uninstrumented tests to make other recommendations.[26–30]

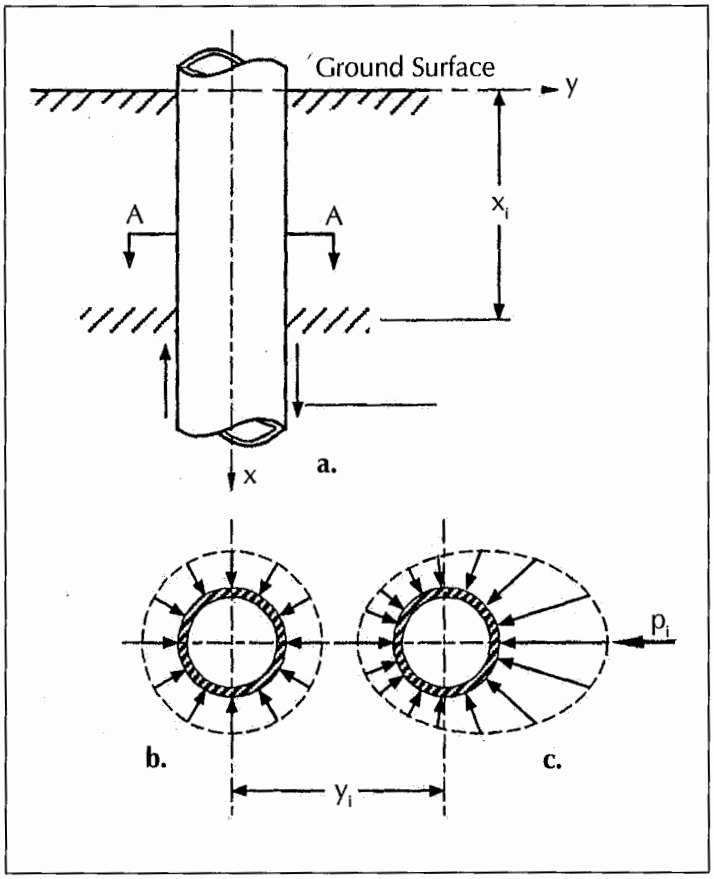


FIGURE 9. Definition of *p* and *y* as related to the response of a pile to lateral loading.

Price and Wardle reported the results of lateral-load tests of a bored pile (identified as TP12) with a length of 12.5 meters and a diameter of 1.5 meters.[31] The location of the tests was not given and is listed as the location of the British Building Research Establishment for convenience. The reinforcement consisted of 36 round bars, 50 millieters in diameter, on a 1.3-meter-diameter circle. The yield strength of the steel was 425 N/mm^2. The cube strength of the concrete was 49.75 N/mm^2. The bending moment at which a plastic hinge would occur was computed to be 15,900 kN-m at concrete strain of 0.003.

The authors installed highly precise instruments along the length of the pile. The readings allowed the determination of bending moment with considerable accuracy. The properties of soil reported by the authors, and the interpretations used for the following analyses, are shown in Table 1.

The lateral load was applied at 0.9 meters above the ground line. Each load was held until the rate of movement was less than 0.05 millimeters in 30 minutes. The load was reduced to zero in stages and held at zero for one hour. The computer program LPILE was used in computing the response of the pile with the conditions indicated.

The comparisons of pile-head deflection and maximum bending moment are shown in Figure 12 (on page 18). The curves for deflection show that the computation is about 20 percent unconservative for the larger loads and in good agreement for the smaller loads. The maximum bending moment from the experiment is about 12 percent higher than the

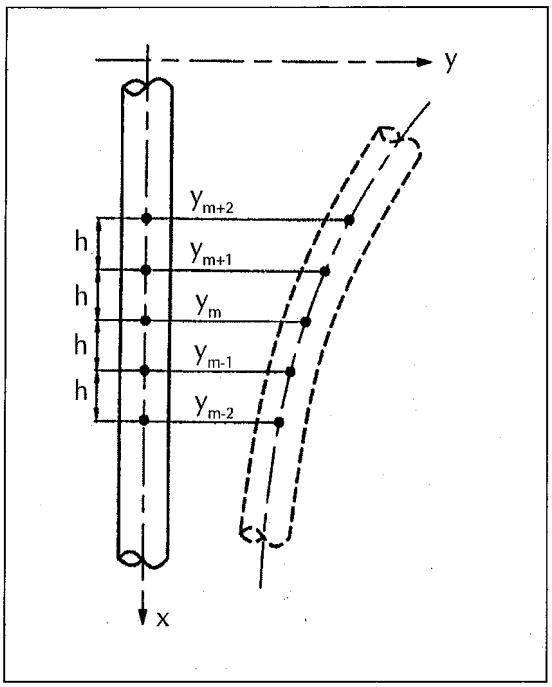


FIGURE 10. A representation of a deflected pile.

computed value at the same lateral load. The computer yielded a lateral load of 4,520 kN to cause a plastic hinge.

Isolated Pile Subjected to Torque. Most piles are installed in groups. The non-linear models for isolated piles under axial loading and lateral loading were presented earlier because they are employed in the analysis of groups. For a similar reason, the analysis of an isolated pile subjected to torque is presented here.[32] While the models for single piles under axial and lateral loading have numerous independent applications, the model for torque has little utility except in the analysis of a group that rotates about a vertical axis.

The increment in Figure 13 represents a free-body element taken from a cylindrical pile subjected to torque. The soil around the element is assumed to exhibit a resistance to rotation as a function of the angle of twist. The torque at the top of the increment is T and $T+dT$ at the bottom. The change in torque dT along the length Δx is due to transfer of torque to the soil, as shown in Equation 7:

$$dt = ((k_\theta)(\theta_x))((2\pi r)(\Delta x))(R) = (2\pi R^2)(k_\theta)(\Delta x) \quad (7)$$

where:

k_θ = stiffness of soil; and
θ = angle of rotation.

The angle of twist over the increment is computed from the standard equation from mechanics:

$$\theta_x = (T_x \Delta x)/GJ \quad (8)$$

where:

G = modulus of elasticity in shear; and
J = polar moment of inertia.

Equations 7 and 8 may be solved in a number of ways. An expeditious way is to select values for the angle of twist at the bottom of the pile. The two equations can then be solved for the corresponding torques. Even if the stiffness of the soil is non-linear with the angle of twist, iteration will not usually be necessary if the length of the increments is chosen to be relatively small.

Stoll pointed out that the torsion-load test on piles is simpler and more economical to perform than the axial load test.[33] The back-analysis of the torsion response of test piles yields values of shear modulus for the soil that are in good agreement with those deduced from t-z curves from axial load tests. Seed and Reese reached a similar conclusion in their research that used the torsion vane to derive the load-transfer curve for side resistance under axial loading.[34]

Interaction of Closely Spaced Piles in a Group. Stresses are transferred through the soil around a loaded pile but the stresses diminish rapidly with distance. If piles are close together, the transfer of stress from one pile to the next must be considered, and the *efficiency* of the piles in the group could be reduced. At the outset, mention of the cost of performing a field test of a pile group is appropriate, especially if the piles are instrumented. The cost is significantly reduced if tests are performed on small-sized piles in the laboratory. However, the inability to model the stress from the overburden seriously limits the value of tests of piles under axial loading; the limitation is less severe for tests of small-sized piles under lateral load but the limitation remains. The cen-

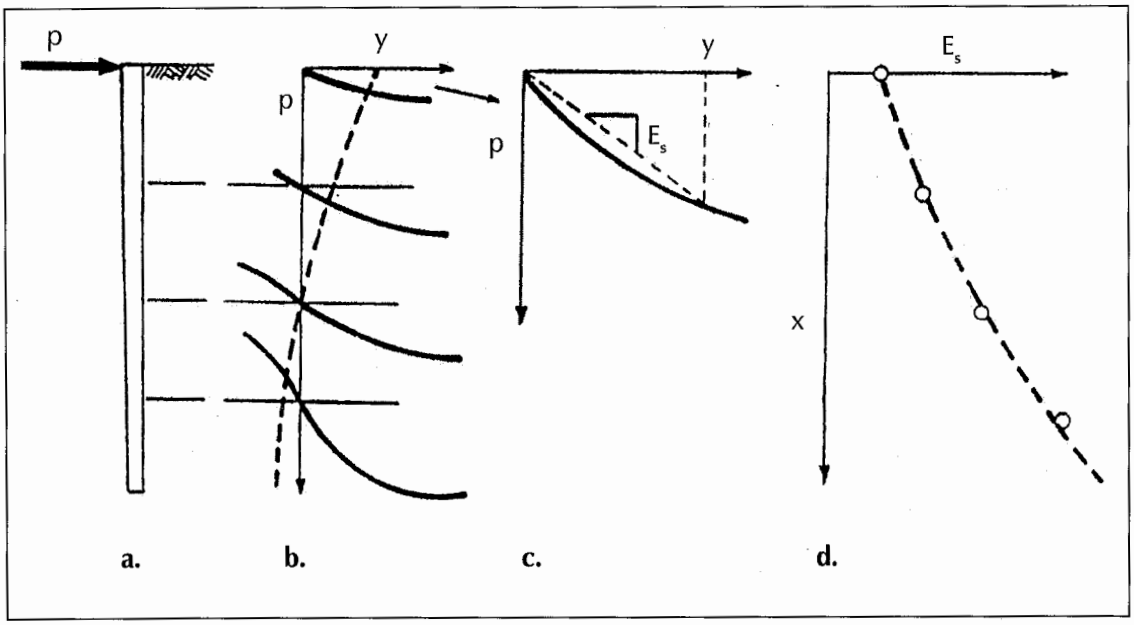


FIGURE 11. The procedure for solving the response of a laterally loaded pile.

trifuge may be used to test piles that are properly modeled but such tests are expensive because of the nature of the test and because of the large number of parameters that must be addressed. The result is a significant lack of data on the behavior of closely spaced piles.

O'Neill made a comprehensive study of data that were available on the interaction of closely spaced piles.[35] With regard to the behavior of piles under axial loading, he wrote the following:

> In view of the preceding observations concerning the effects of time, soil characteristics (consistency, initial density, stress history, permeability) on efficiency, along with the effects of cap contact, driving order, and pile-capacity variation, it is not reasonable to apply general efficiency formulas to design.[35]

With regard to piles under lateral loading, the failure to model the overburden stress is less of a problem because the pile obtains much of the resistance from near-surface soils. Further, the magnitude of the loading is much less than for axial loading and a number of pile groups have been tested in the field.

TABLE 1.
Reported Properties of Soil at Garston

Depth (m)	Description	N_{SPT}	Unit Weight (kN/m³)	Friction Angle (°)
0–0.36	Fill	18	—	—
0.36–3.5	Dense Sandy Gravel	~65	21.5	43
3.5–6.5	Coarse Sand & Gravel	30	9.7	37
6.5–9.5	Weakly Cemented Stone	~61	11.7	43
9.5–	Highly Weathered Sandstone	~140	—	—

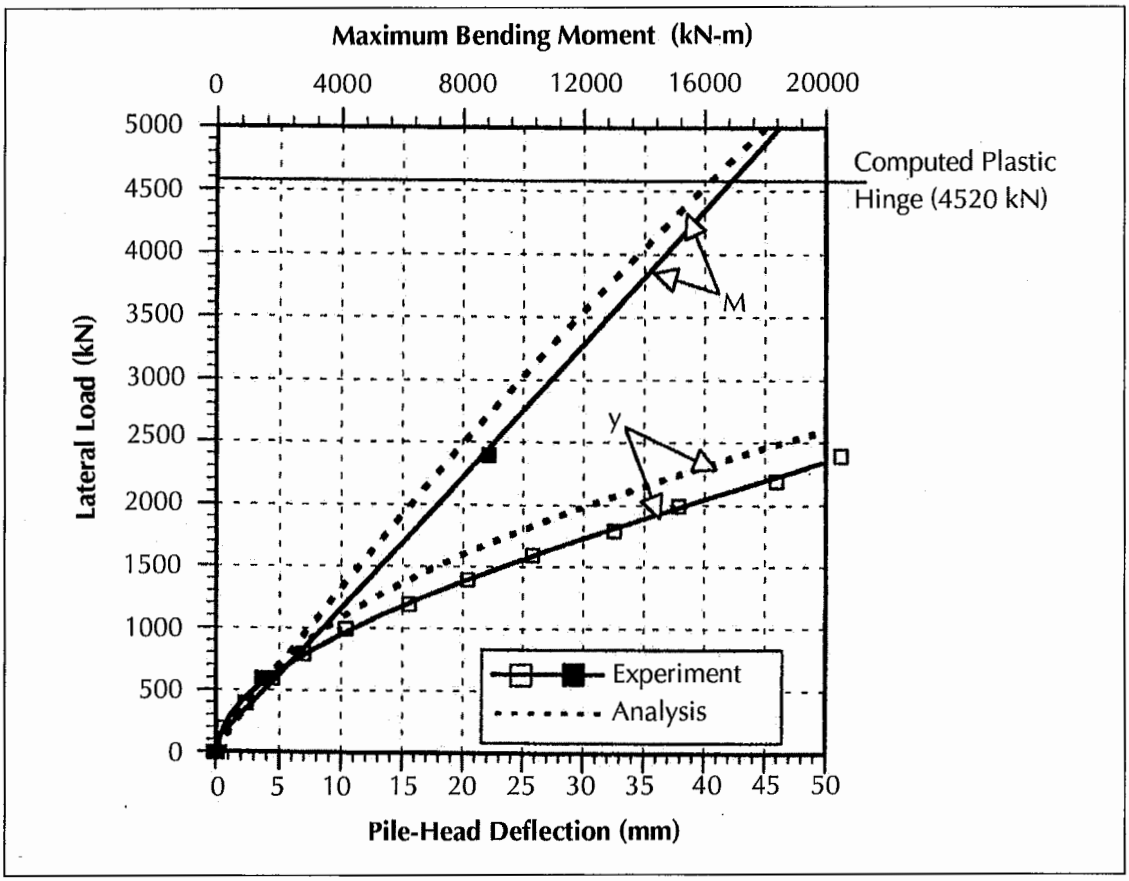


FIGURE 12. A comparison of the experimental and computed values of maximum bending moment and pile head deflection (static loading, Garston).

For axial load, with no contact between the cap and the soil, the theoretical capacity for a group can be computed by assuming that the soil inside the group moves downward with the piles. The failure of the block, including piles and enclosed soil, can be computed where the capacity in side resistance occurs along an area around the group and the capacity in end bearing occurs in the horizontal area at the base of the piles (including the areas of the points of the piles and the enclosed soil). O'Neill's work for piles under axial load showed that the *block model* failed to agree with some experiments; however, the model has the advantage of theoretical correctness.[35]

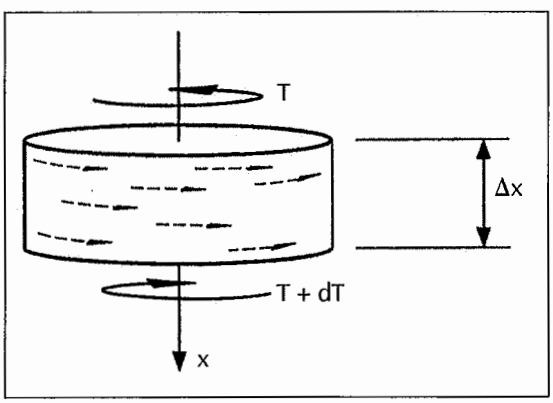


FIGURE 13. An element of a pile subjected to torque.

With regard to piles in a group under lateral load, the results of a number of experiments provide data that can be analyzed to lead to recommendations. Scott studied the results of experiments with closely spaced piles where an isolated pile was loaded in addition to the group.[36] Experiments in the field and the laboratory were included. The results of that

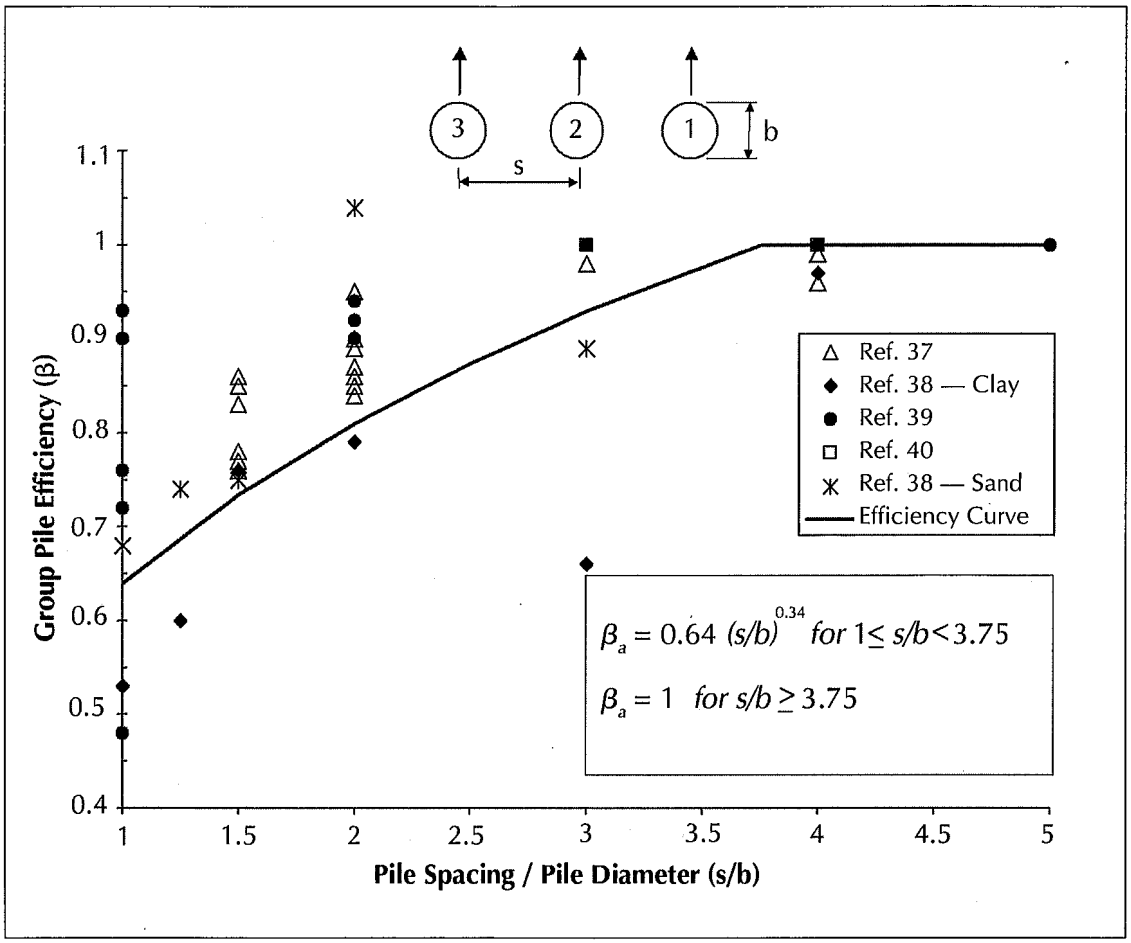


FIGURE 14. Curve giving reduction factor, β_a, for piles in a row.

study provided significant guidance in the recommendations that follow. The current recommendations are presented in Figures 14 through 16, where experimental data are plotted along with the equations for the following cases: piles in a row, leading piles in a line and trailing piles in a line — all as a function of the relative spacing *s/b*, where *s* is the center-to-center spacing and *b* is the diameter of the pile. (The data in the figures are derived from a number of references as shown.[37–42]) The aim of employing the recommendations of close spacing is to obtain a value *f* for each of the piles in a group that may be used to modify *p-y* curves to account for close spacing. The *p-y* curves for each of the piles in a group may be modified as shown in Equation 9:

$$p_{gi} = p_{spi} f_i \quad \textbf{(9)}$$

where:

p_{gi} = the modified values of p, point by point, for pile i;

p_{spi} = values of p for isolated pile for pile I, and

f_i = modification factor for pile i, in the group (value is less than 1).

The value of *f* for each pile in a group is obtained by multiplying the values of β as found in Figures 14 through 16. For a pile that is neither in line or side by side, an expression is derived for piles that are *skewed*, as shown in Figure 17 (on page 22) where the skew angle is given by the angle ϕ. The sketch and the equation in the figure show the procedure for accounting for the skew. The value of f_4 for Pile 4 in the group shown in Figure 18 (on page 22) is equal to

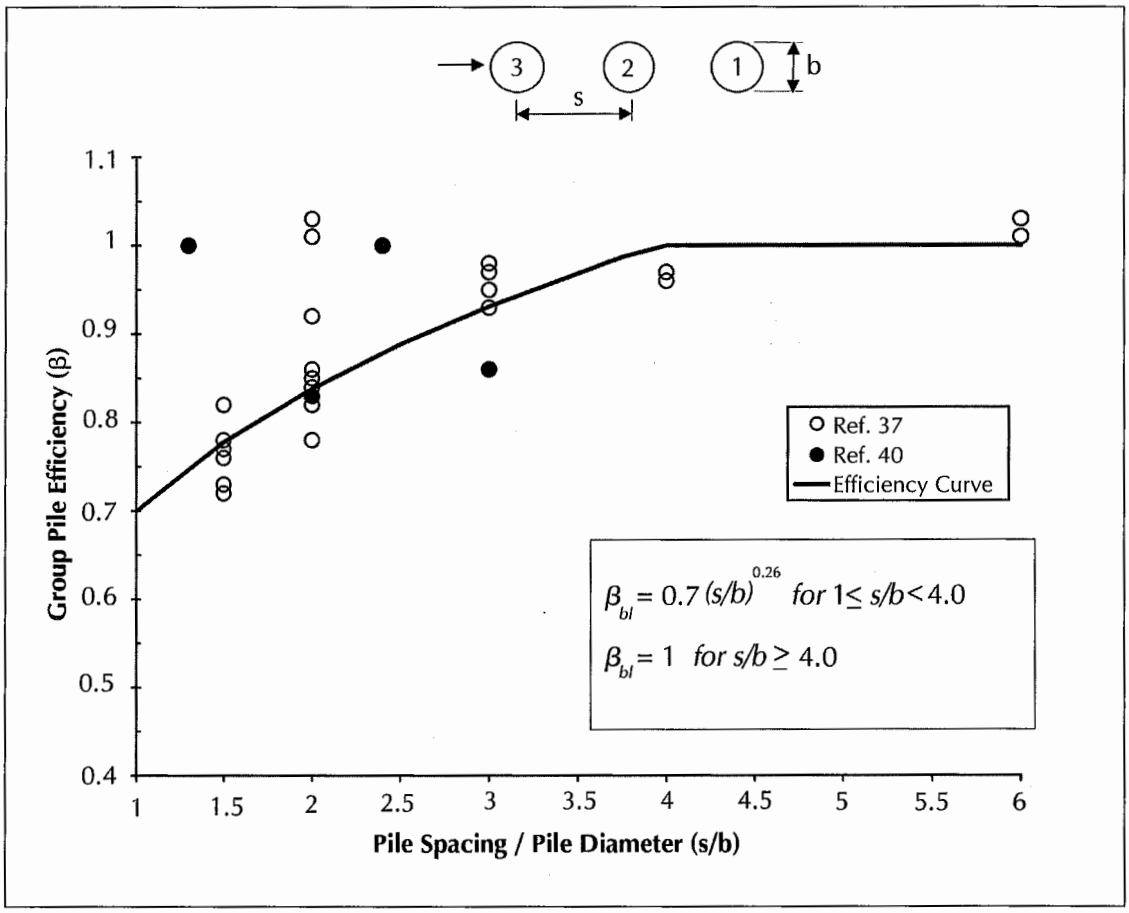


FIGURE 15. Curve giving reduction factor, β_{bl}, for leading piles in a line.

$\beta_{34}(sidebyside)\beta_{24}(trailing)\beta_{64}(leading)\beta_{14}(skew)\beta_{54}(skew)$.

Referring to Figures 14 through 16, the following values are computed: β_{34} is equal to (0.42)(2.0)0.34, or 0.8101; β_{24} is equal to (0.48)(3.0)0.38, or 0.7287; and, β_{64} is equal to (0.70)(3.0)0.26, or 0.9314. For the skewed piles, the distance r is equal to 3.606 b and the angle ϕ is equal to 33.69°. Substituting into the equation in Figure 17 β_{14} equal to 0.8509 and β_{54} equal to 0.9809, then f_4 is equal to (0.8101)(0.7287)(0.9314)(0.8509)(0.9809), or 0.4589

Each of the values of p for the single, or isolated, pile is multiplied by the factor 0.4589 to obtain values of p for a pile in a group. This example shows that computations by hand are tedious; therefore to speed calculation the computations are included in the code for the computer program GROUP. The engineer should note that the exercise shows that individual piles in a group will sustain much less load than isolated piles. While there is no doubt that piles that are closely spaced will behave less well than isolated piles due to the transfer of stresses through soil, the scatter in the data in Figures 14 through 16 is significant and several relevant parameters are omitted. Tests of a group of piles under lateral load will be very expensive but such tests could be considered for a large project.

Group of Piles Under Three-Dimensional Loading

The pile group is among the problems in soil-structure interaction for which a solution has been obtained. The coordinate systems, along with the loading, are shown in Figure 19.[43] The pile cap is assumed to be rigid and a set of equations can be written where the

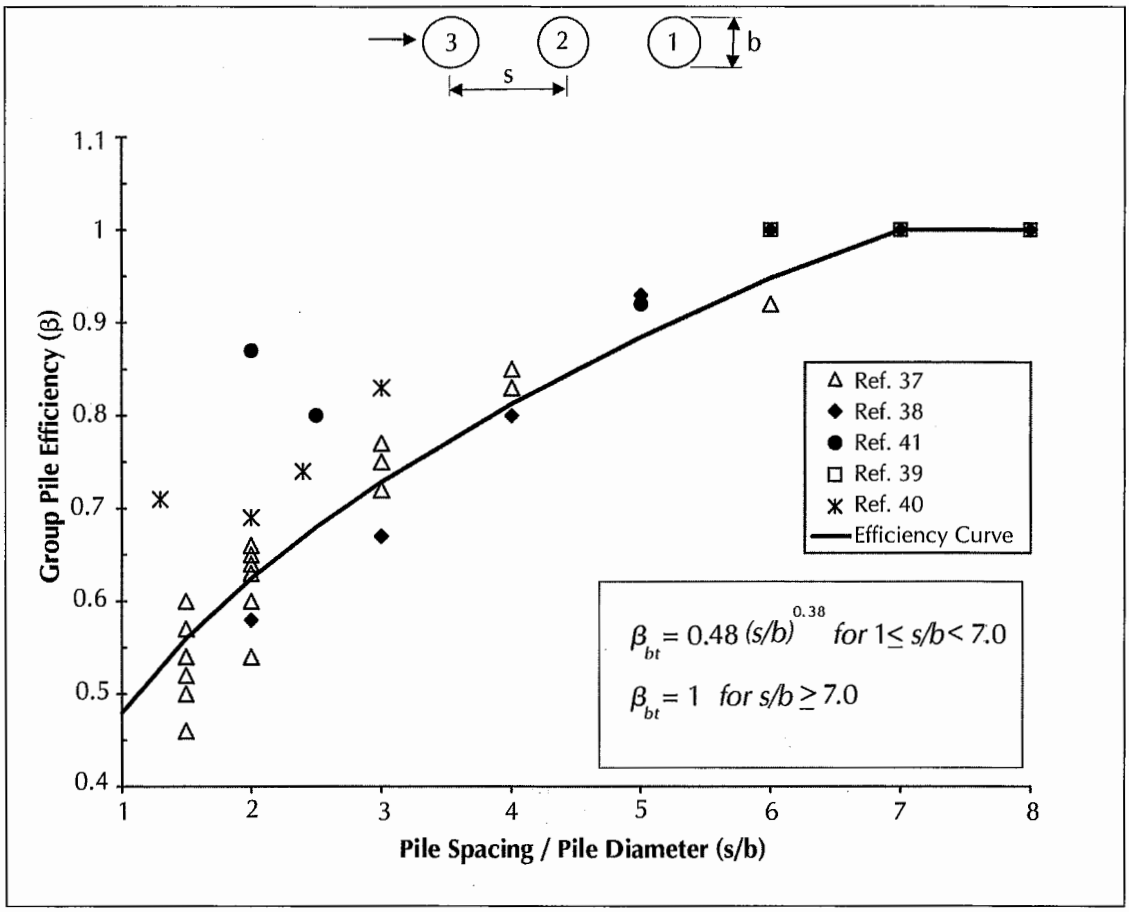


FIGURE 16. Curve giving reduction factor, β_{bt}, for trailing piles in a line.

applied loads are in equilibrium with the resistances at the pile heads. The pile-head resistances may be computed with procedures presented herein for axial load, lateral load and torque, taking close spacing into account. Because of the non-linearity of the soil, and perhaps of the pile material, iteration must be employed in achieving a solution.

An example of a pile-supported bridge bent is presented to illustrate the capability available to the engineer to investigate a number of parameters that may be addressed. The placement and orientation of the supporting pipe piles are shown in Figure 20 (on page 24). The figure shows the cap to be embedded in the soil but resistance against the cap is ignored. The problem becomes a piled raft if the reaction of the soil against the base of the cap is considered. The piles are steel pipe 30 inches in diameter with a wall thickness of 0.75 inches and a penetration of 94 feet. The strength of the steel was 36 ksi. Three layers of soil were assumed to exist at the site: 10 feet of soft to medium silty clay with an undrained shear strength of 900 psf at the top of the layer to 1,140 psf at the bottom; 35 feet of loose to compact sand with a friction angle of 33° at the top and 36° at the bottom of the layer; and a stratum of very stiff clay with a undrained shear strength of 4,000 psf to a depth of 167 feet. The water table was assumed to be at or near the ground surface and the submerged unit weight of each of the strata was assumed to be 60.5 pcf.

The curves showing the computation of the response of a single pile to axial load are shown in Figure 21 (on page 25). The failure load in compression is 1,430 kips and in tension is 1,260 kips. The pile-head movement to

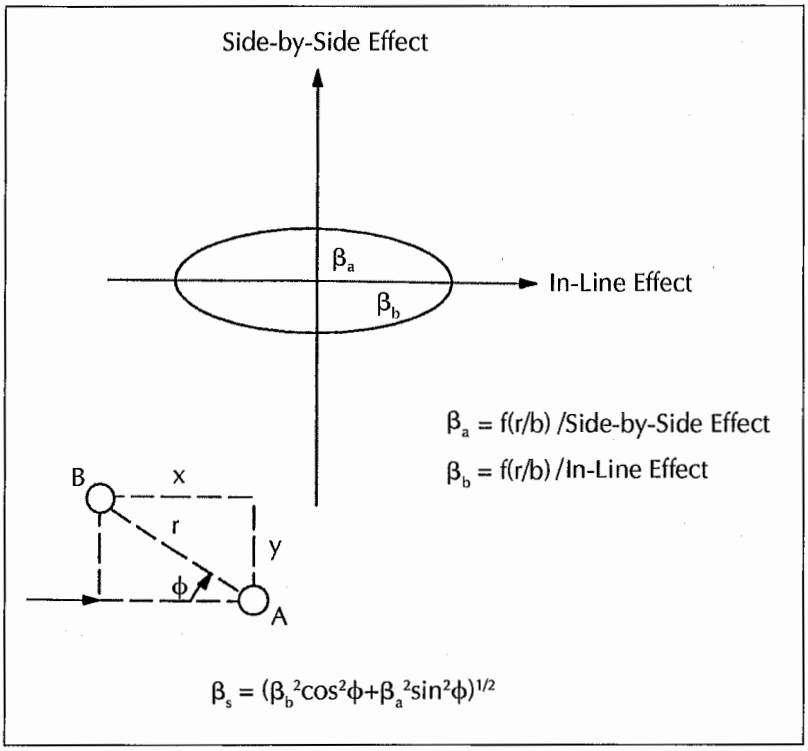


FIGURE 17. A system for computing the reduction factor for skewed piles.

achieve these loads is more than 2.5 inches. The curves are presented to allow the determination of failure of one of the piles in compression or tension and may be termed a geotechnical failure. Failure due to the development of a plastic hinge is assumed to occur when the combined stress becomes equal to 36 ksi and may be called a structural failure.

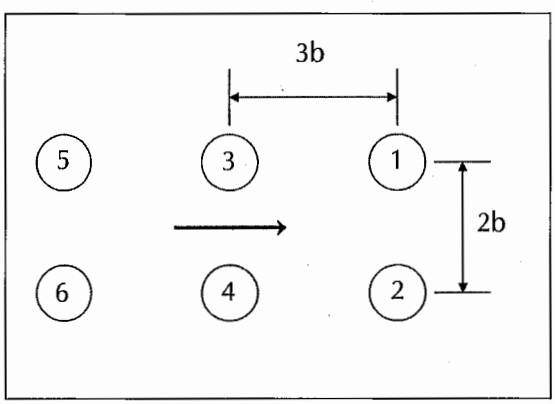


FIGURE 18. Pile group for example computations.

The loadings shown in Figure 20 are the so-called "service" loads or the sum of the dead and live loads during normal operating conditions. The matter of the loading of a structure needs careful attention and may require a considerable amount of time from an experienced engineer, especially if load- and resistance-factor design (LRFD) is being employed. For the present example, the service loads are factored upward to find the loading that causes failure, either by causing a pile to be overloaded or the presence of a plastic hinge. Some agencies define "failure" as the loading that produces a given deflection, say one-half inch. The writer prefers to define failure as noted and then to consider the movement of the piles and the structure under the service load.

Table 2 shows the axial load and maximum combined stress for each pile in the group for the service loads, along with the movement of the origin of the coordinate system. Table 3 (on page 26) shows similar information for the factored loading that caused failure. An examination of Tables 2 and 3 shows that none of the piles for the load factor of 2.0 reached the ultimate axial load and only one, Pile 8, reached the stress to cause a plastic hinge. If the loading is repetitive, the engineer may decide that the load factor of 2.0 constitutes the failure condition and then decide whether or not the design is adequate. If only a small number of the ultimate loads are expected during service, the engineer may decide to do a "push-over" analysis to find the load factor at which the foundation would collapse.

A further evaluation of the data in Tables 2 and 3 leads to some interesting observations.

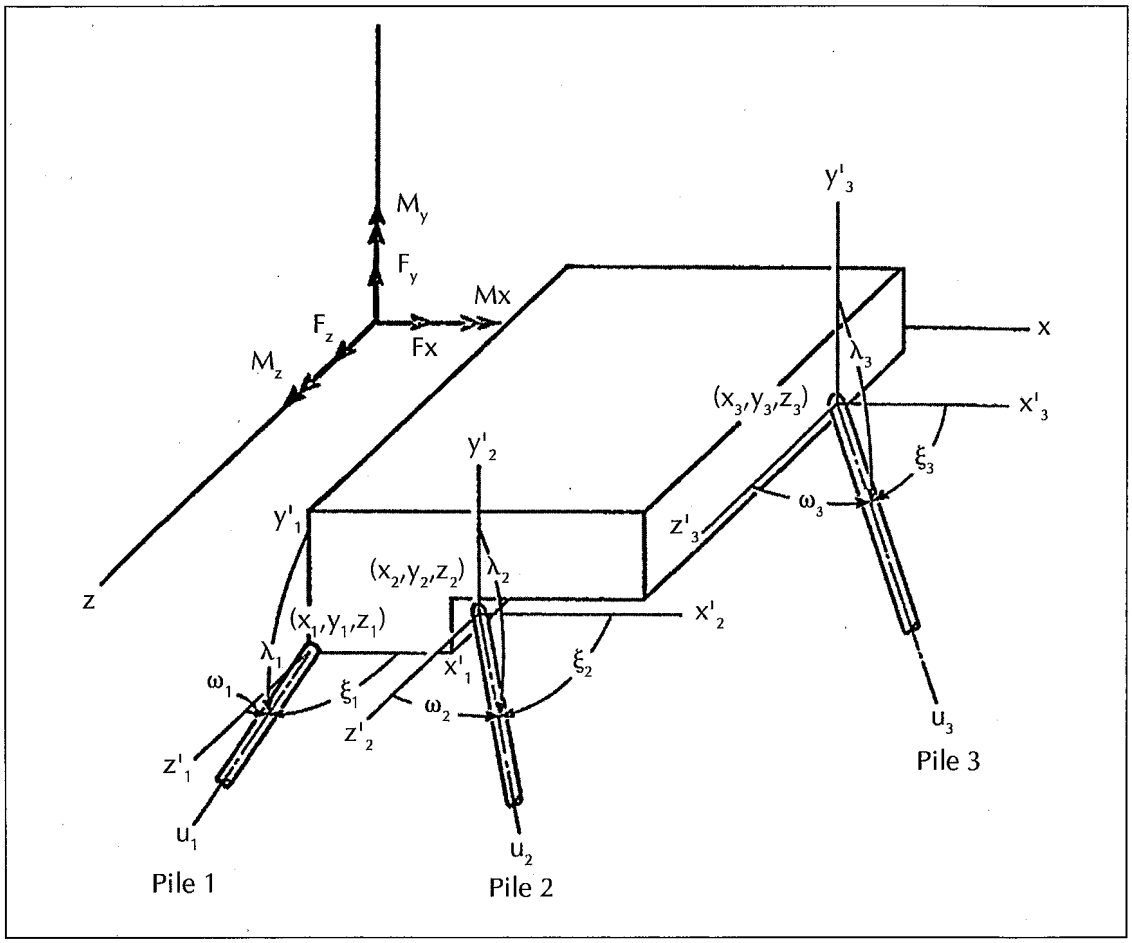


FIGURE 19. Coordinate system for a three-dimensional group of piles.

The engineer may investigate a number of parameters to seek a solution at minimum cost presuming that information on the cost of construction is available. The diameter, spacing and penetration of the piles may be modified. The use of piles of a different type may be investigated. Soil properties are seldom given in such specific terms as employed in the example. Analyses can be made using lower-bound values and upper-bound values to gain information on the desirability of a superior investigation of soil parameters.

A further point of interest involves the penetration of the pipe piles into the cap. For the present analyses, the pile heads were assumed to be free to rotate. The bending-moment curves for a free-head pile and a fixed-head pile are shown in Figure 22 (on page 26). The difference is dramatic, indicating the need to know the correct boundary condition. Conventional logic states that a small penetration into the pile head, just enough to resist the shear, is a free-head pile and a penetration of two or three diameters constituted a fixed-head pile. Actually, any pile-head penetration likely causes some rotational restraint and a fully fixed-head condition cannot be obtained. A model such as the one in Figure 23 (on page 27) is being used to work out the rotational restraint as a function of pile, the reinforced-concrete cap, amount of penetration and magnitude of loading. The computer code, ATENA, is being employed.[44]

Piled Raft

One of the most challenging problems in SSI is the piled raft. A structure (see Figure 24 on page 28), perhaps a high-rise building, is supported

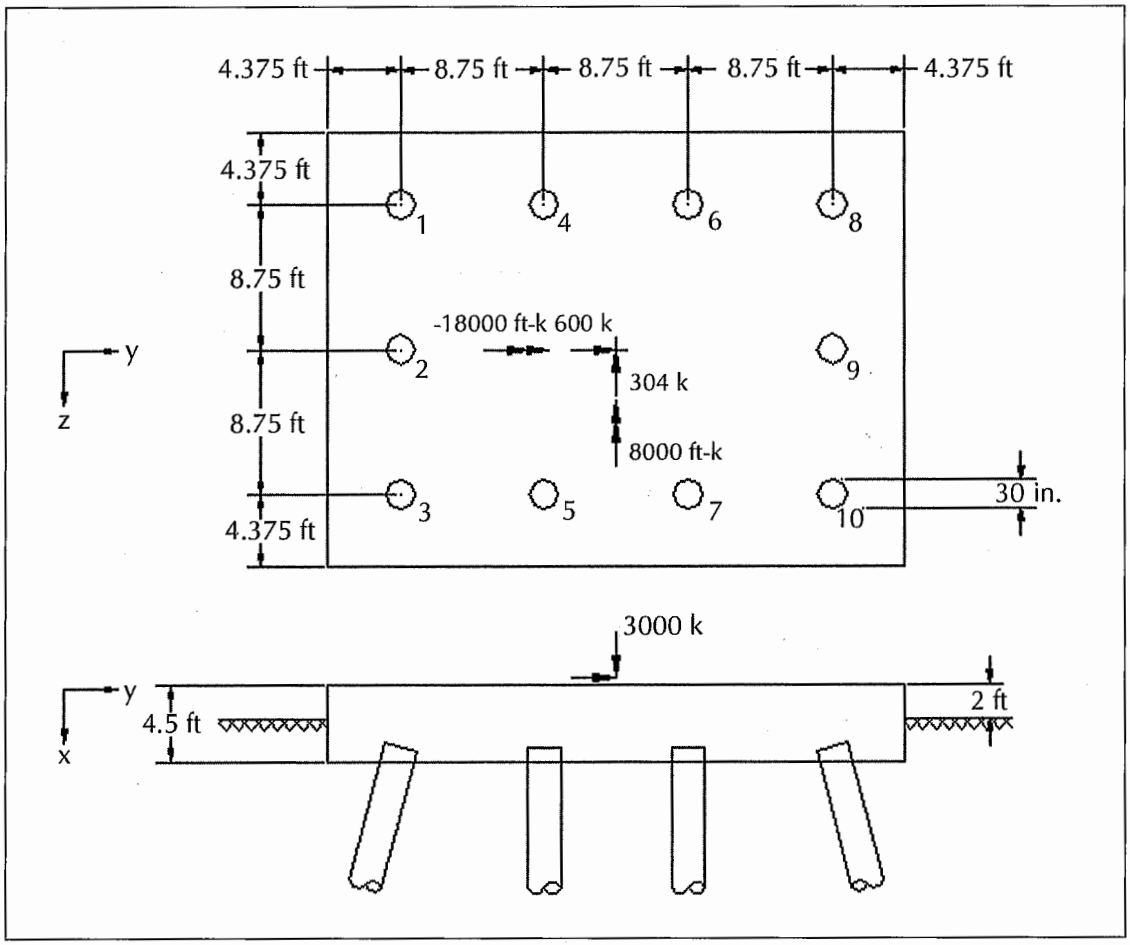


FIGURE 20. A pile-supported bridge bent.

both by piles and by a mat foundation. Considering each of these foundation elements separately leads to the conclusion that interaction is inevitable. The mat alone is certainly affected by the presence of the piles because the foundation is much stiffer than with the soil alone. The piles alone are affected by the earth pressure from the mat because the increased lateral stress on the piles affects the capacity in size resistance. The problem can be solved by using the finite-element method where appropriate elements are used to model the piles and the soil is modeled by softer elements.

A number of high-rise buildings in Frankfurt, Germany, were founded on piled rafts and extensive observations were made of the behavior of their foundations (see Figure 25 on page 29).[45,46] One of those buildings was Westendstrasse 1 and its foundation plan is shown in Figure 26 (on page 30). The raft had a thickness of 4.5 meters and supported a tower of approximately 208 meters in height. From the use of the instruments, the authors were able to determine the load shared by the raft and the piles. In terms of short-term settlement, the raft is far more flexible than the piles; therefore, the addition of piles to the foundation favorably affected the settlement of the structure.

The foundation was modeled with finite elements and Figure 27 (on page 30) shows the comparison of settlement from computation and from the field. The agreement is excellent. The computer analysis shows the deflection across the structure, as indicated in Figure 28 (on page 31). The movement of the raft was sufficient to develop the full resistance of the piles with a considerable portion of the load

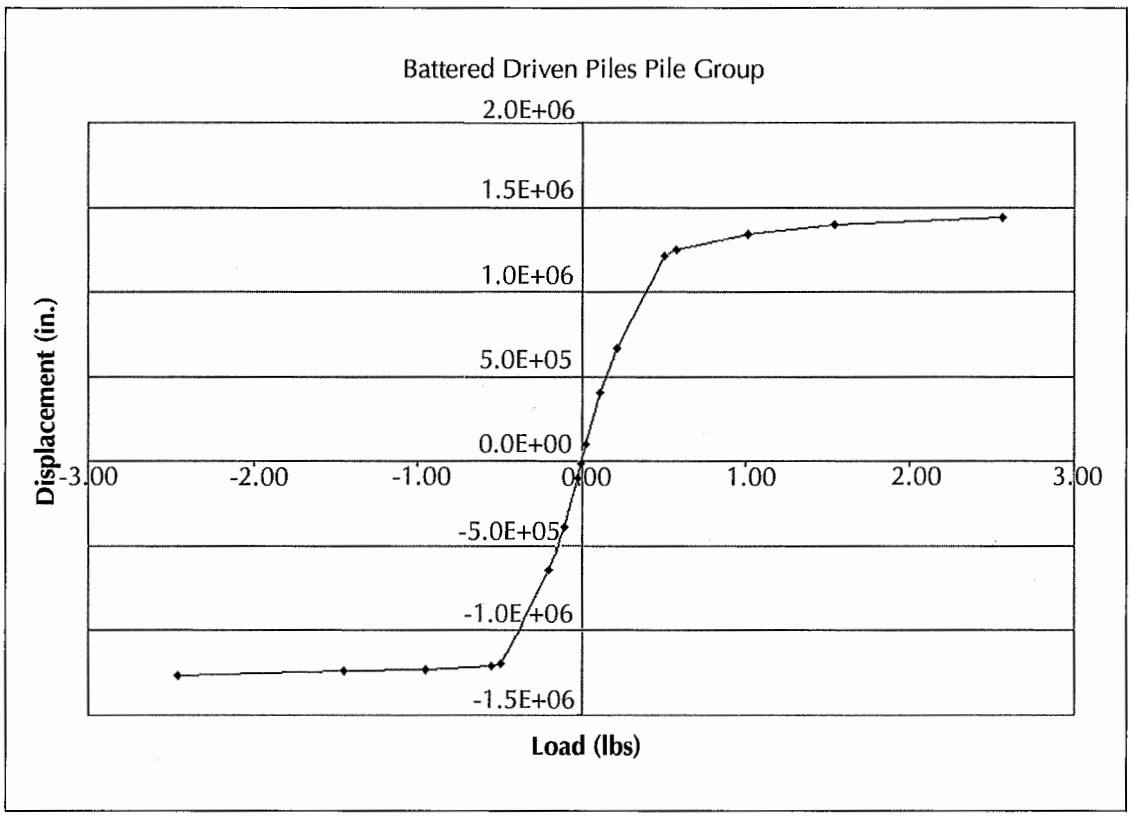


FIGURE 21. Axial load versus displacement for a single pile in a group.

TABLE 2.
Results From Service Loading

Pile No.	x_t (in.)	y_t (in.)	f_{max} (kips)	P_x (kips)
1	0.138	0.465	14.3	474
2	0.044	0.458	10.1	173
3	–0.054	0.443	10.0	–208
4	0.182	0.444	15.6	598
5	0.020	0.445	8.3	88
6	0.166	0.444	14.8	548
7	0.004	0.445	7.1	18
8	0.235	0.406	17.4	710
9	0.140	0.428	14.4	479
10	0.043	0.444	10.1	170

Note: Movement of coordinates: x = 0.093 in., y = 0.448 in.

TABLE 3.
Results From Load Factor = 2

Pile No.	x_t (in.)	y_t (in.)	f_{max} (kips)	P_x (kips)
1	0.164	1.477	27.6	541
2	0.082	1.487	25.2	303
3	–0.008	1.498	20.3	–37
4	0.215	1.460	29.3	674
5	0.266	1.460	30.7	767
6	0.155	1.459	26.6	517
7	0.205	1.460	28.5	648
8	0.417	1.402	36.0	1050
9	0.332	1.426	34.1	891
10	0.244	1.446	31.8	728

Note: Movement of coordinates: x = 0.210 in., y = 1.424 in.

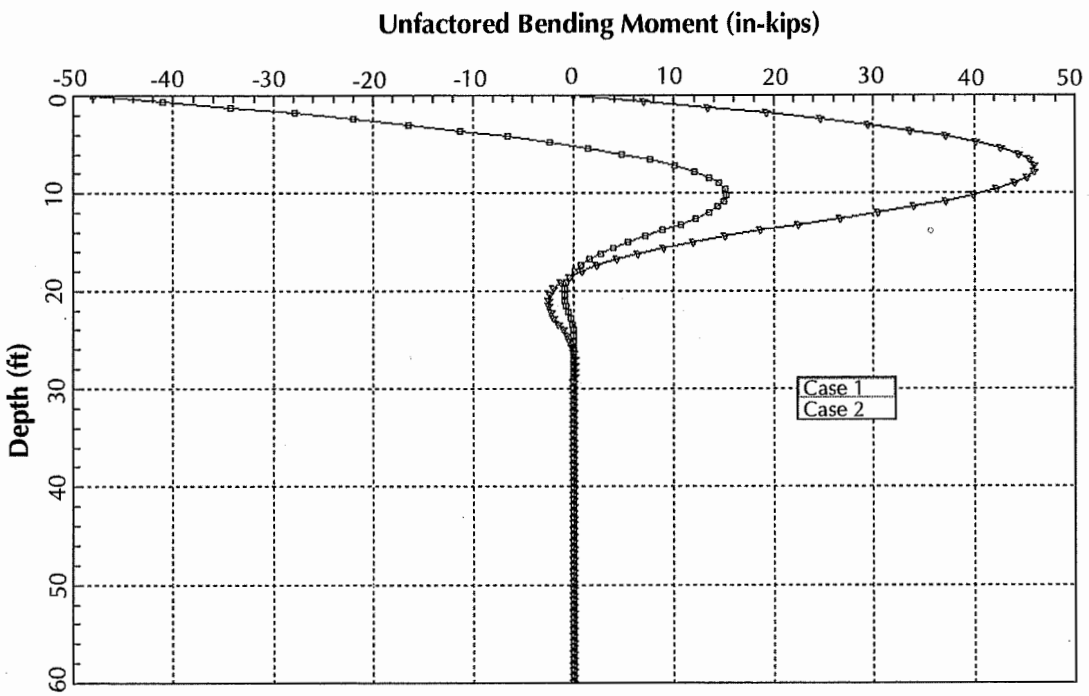


FIGURE 22. Typical bending moment curves for free-head and fixed-head piles.

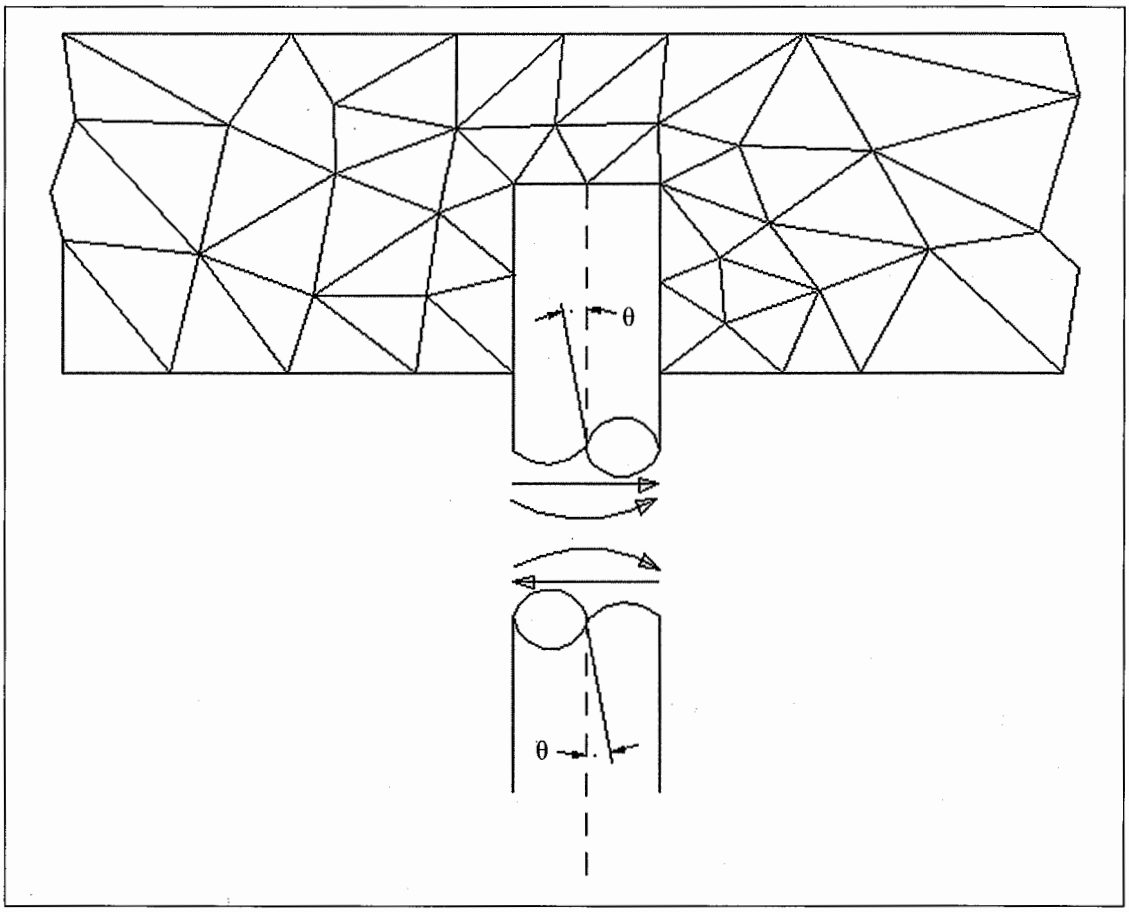


FIGURE 23. A finite-element model for solving for rotational restraint.

on the piles being transferred in end bearing, as shown by Figure 29 (on page 31), where the finite-element model reveals a stress bulb in the zone of the base of the piles.

For the analysis of Westendstrasse 1, linear finite-elements were employed because for much of the soil the stress-strain curve was judged to be in the linear range. Three-dimensional, non-linear finite elements can be run with a PC but runs for many problems will require several hours. The use of a PC for such problems will become more practical as the PC continues to gain in computational power.

Foundations Under Dynamic Loading

The important area of the response of foundations to dynamic loading has not been neglected by engineers in the development of SSI. A number of the standard finite-element codes can deal with inertia effects and are adaptable to the solution of dynamic problems. The computer code DynaMat can solve for the response of a mat foundation to vibrating machines considering the geometry of the foundation, the embedment depth, the material and radiation damping of the soils, the variation of soil properties with depth, and the interaction between the soil and the foundation. The three-dimensional, hybrid, finite-element method is used to compute the dynamic stiffness matrices for rigid foundations of circular or rectangular shapes. The analytical technique is based on the method proposed by Lin and Tassoulas[47] and results from DynaMat are in excellent agreement with those obtained by techniques proposed by Tassoulas and Kausel,[48] as well as by Chen.[49]

The computer program DynaN is based on the improved Novak Method, where a non-

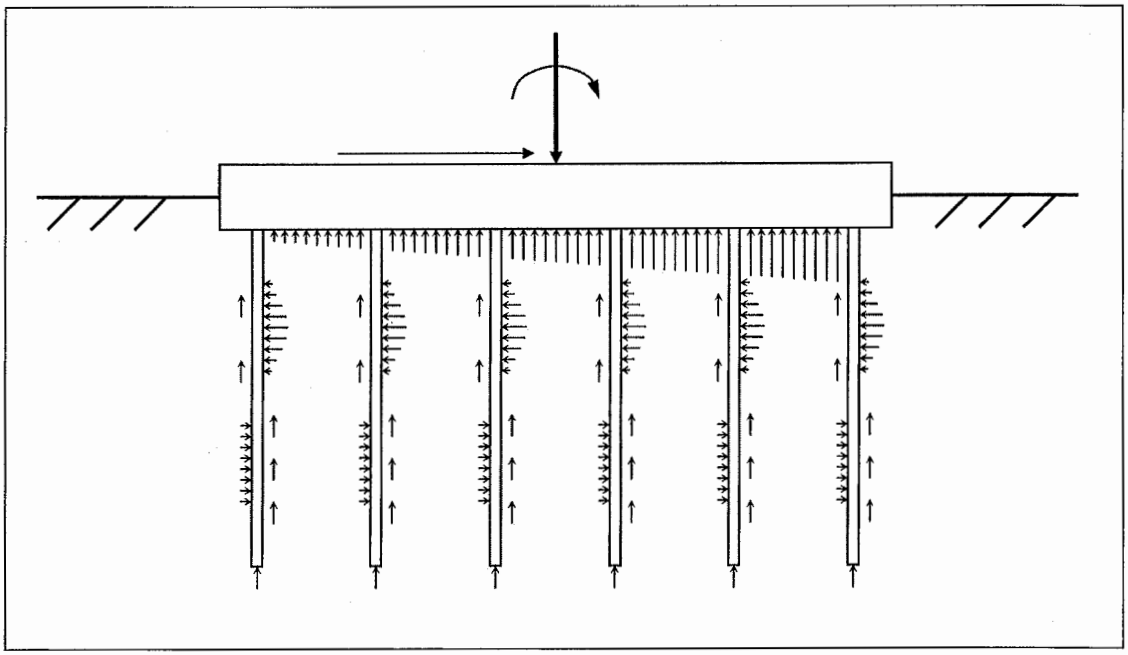


FIGURE 24. Typical design of a piled raft structure.

reflective boundary is formed between the near field and the far field to account for the mass of soil in the boundary. The program yields the dynamic response of both shallow and deep foundations under harmonic, transient and random loadings. Rotating or reciprocating machines, earthquake, wind, blast, sea waves and other sources can produce such loadings. The computational method has been applied to important problems in engineering practice.[50,51] In the development of the method, a series of dynamic experiments were performed on full-scale mat foundations[52] and on full-scale piles.[53,54]

Current Challenges for Civil Engineers

With a moderate expense, the response of many structures to gravitational, traffic and environmental loads can be observed. The making and reporting of such observations can be a valuable contribution to the profession and can further the development of knowledge of soil-structure interaction. Observations can be made by using traditional instrumentation but new instrumentation is being developed regularly. Jessica Binns writes in *Civil Engineering* that ground monitoring is possible using fiber optics sensors attached to a cable (as reported by Tarek Abdoun at Rensselaer Polytechnic Institute).[55] She also writes that Daniel Farhey, University of Dayton, has reported on the development of a wireless sensor system that may be used to investigate the response of a bridge to traffic and other loading.[56] John Dunnicliff has authored for a number of years the section in *Geotechnical News*, Geotechnical Engineering News, and has reported much useful information. Observations of completed structures can now be made with confidence by using many types of instrumentation available in the marketplace.

Too frequently soil investigations are given insufficient attention. Perhaps price competition has led to fewer borings and fewer in situ and laboratory tests. The result may not be a failure of the foundation but failure, nevertheless, if the foundation is much more expensive than necessary. Putting the lessons learned from Casagrande into use on evaluating soil properties allows for the application of the techniques of soil-structure interaction with obvious advantages.

Occasions may arise when engineers can give support to research to further knowledge

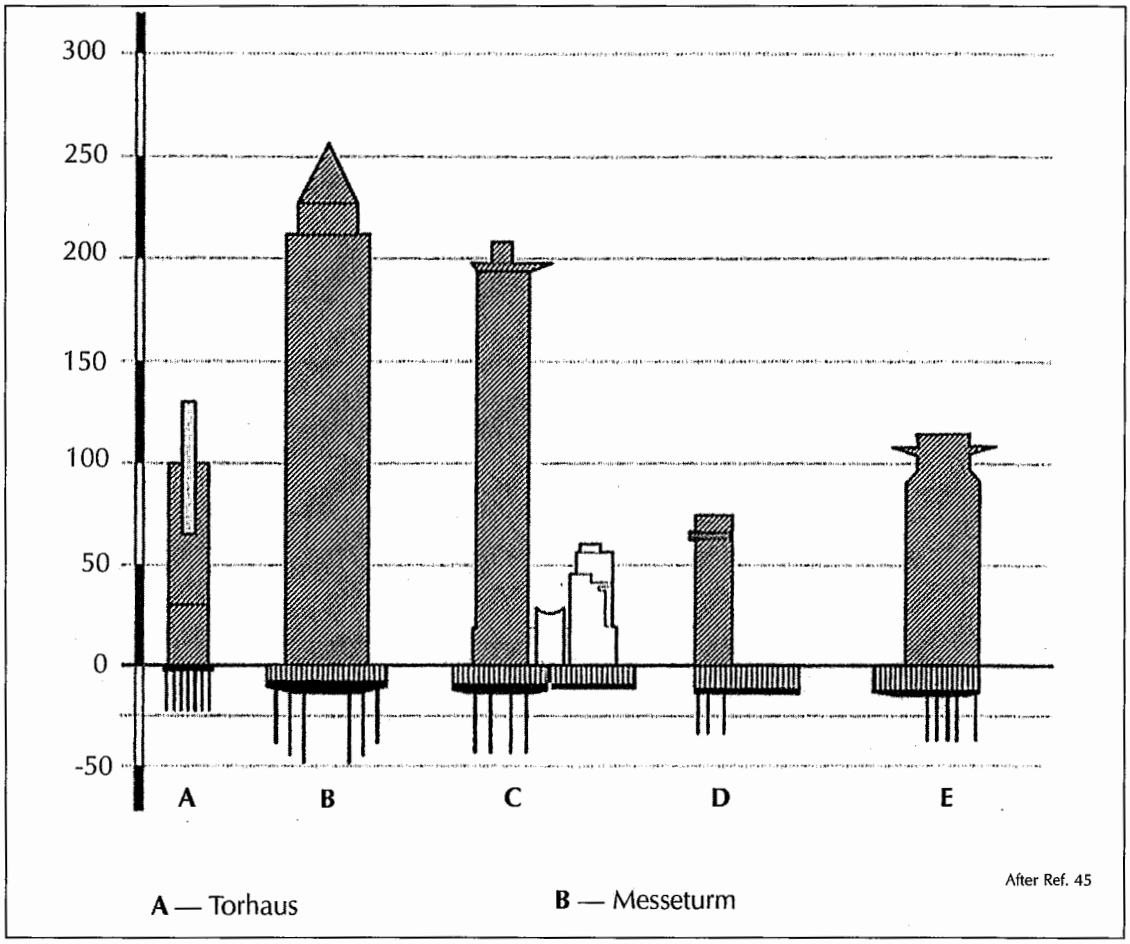


FIGURE 25. High-rise buildings in Frankfurt with piled raft foundations.

of SSI. Many useful areas exist. For example, most of the evidence regarding the influence of pile installation on soil properties is indirect. But methods and tools exist to obtain direct evidence of how driving a pile or installing a drilled shaft will modify the properties of the supporting soil. Presently, empirical parameters are used to modify the properties of soil to reflect pile installation. Later, when research allows for a solution of the problem, a geotechnical engineering student can be imagined asking, "Professor did you mean to say that during the last century *piles were designed based on the in situ properties of the soil?"*

As may be surmised, the technology of SSI is attracting research in many quarters[57,58] and remaining abreast of all such activity takes a sustained effort. Engineers in geotechnics and structures, and others, would be assisted immensely if the United States had a central agency to engage in research and to collect and collate information on such topics as soil-structure interaction, for example. The writer had an occasion to make an extended visit to the British Building Research Station and had useful visits to the Laboratoire Central des Ponts et Chaussées in France, geotechnical institutes in the Scandanavian countries, and similar organizations in India and Pakistan. The United States could benefit from the existence of such an organization.

Stepping Into the Future

Application of available tools to develop information allowing improved methods of analysis is presently possible. Many such tools are currently available, such as ATENA, but

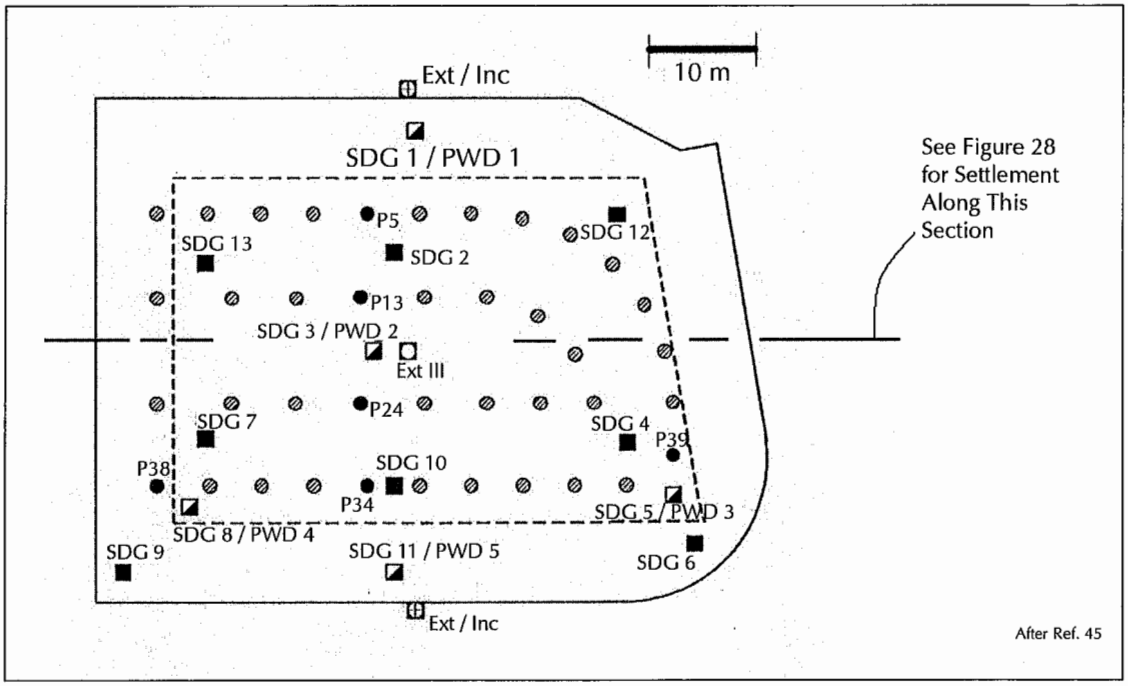


FIGURE 26. Pile locations at Westendstrasse 1.

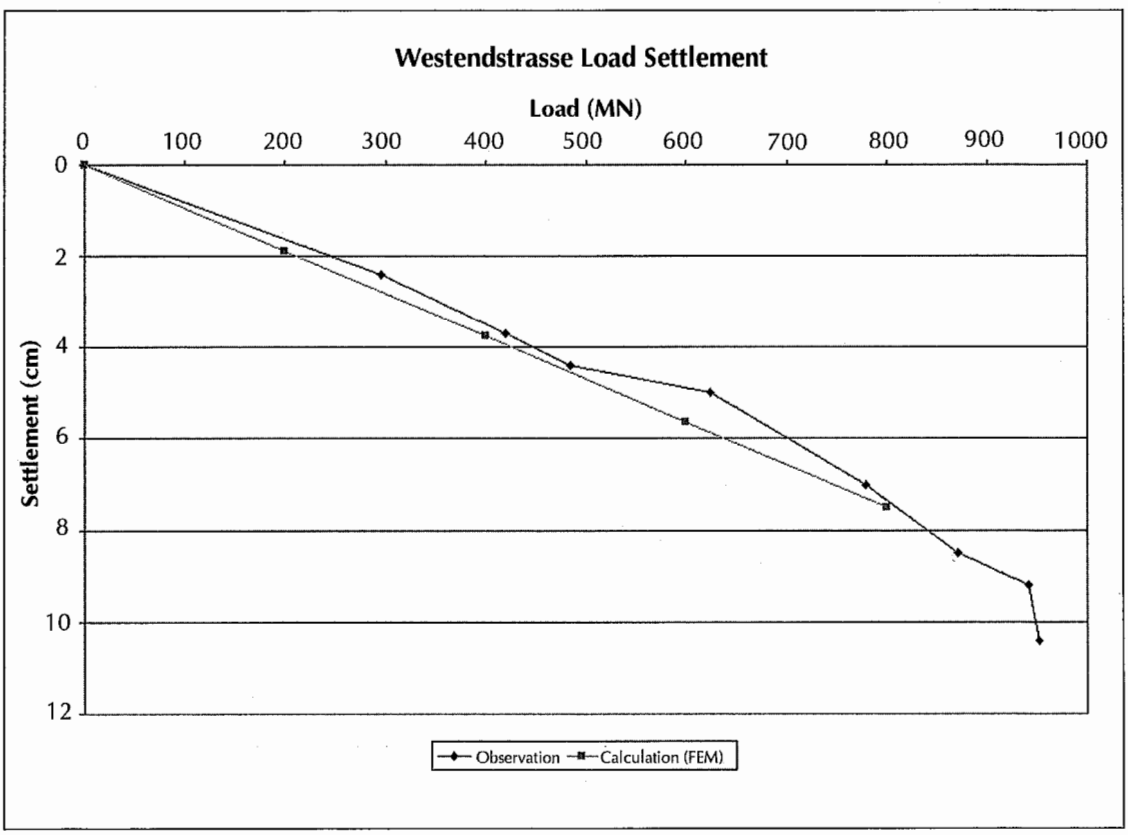


FIGURE 27. Load settlement at Westendstrasse 1.

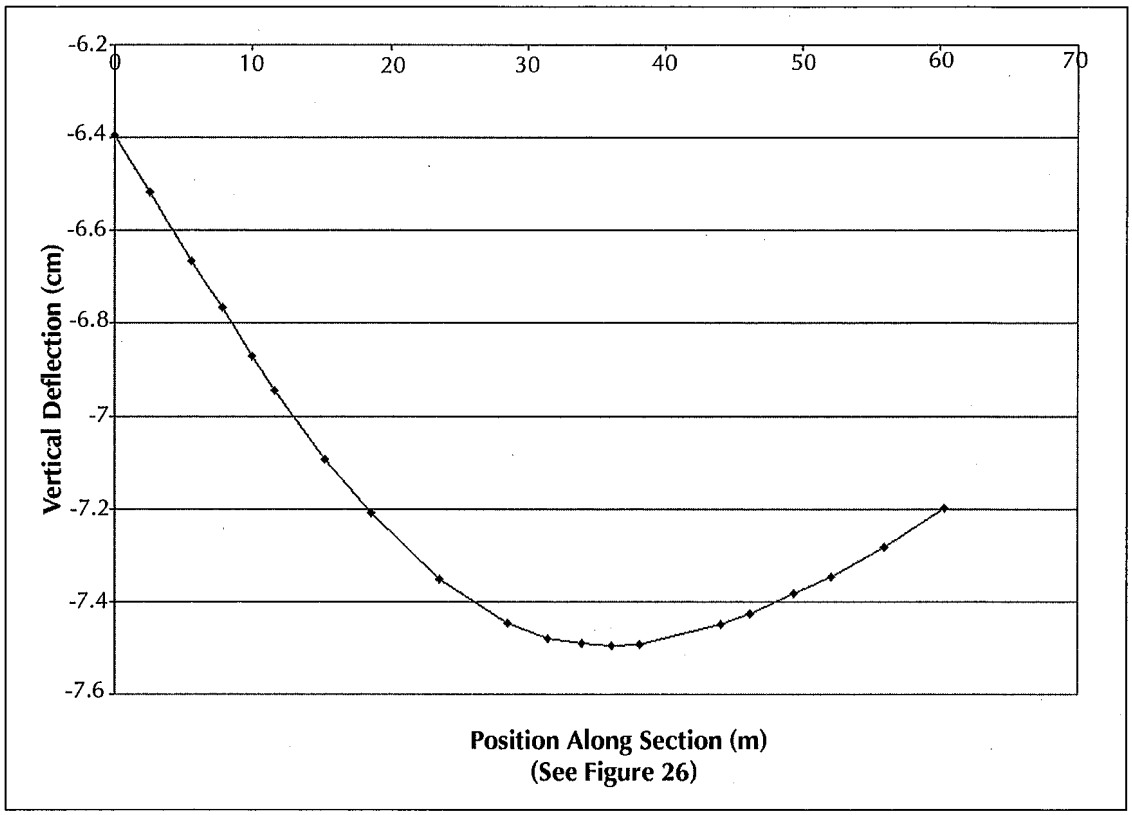


FIGURE 28. Settlement across a piled raft.

the effort in implementing such software should not be minimized. The theory should be understood and appropriate checks must be devised, preferably employing hand computations, to ensure correct results.

It is not too difficult to envision a computer program that will solve simultaneously the behavior of the foundation and the superstructure, accounting for a wide set of loadings, allowing the exploration of significant parameters, considering non-linearity as desirable and necessary, providing simplified input, and producing easily understood output in tabular and graphical forms. Such comprehensive software is well within the capabilities

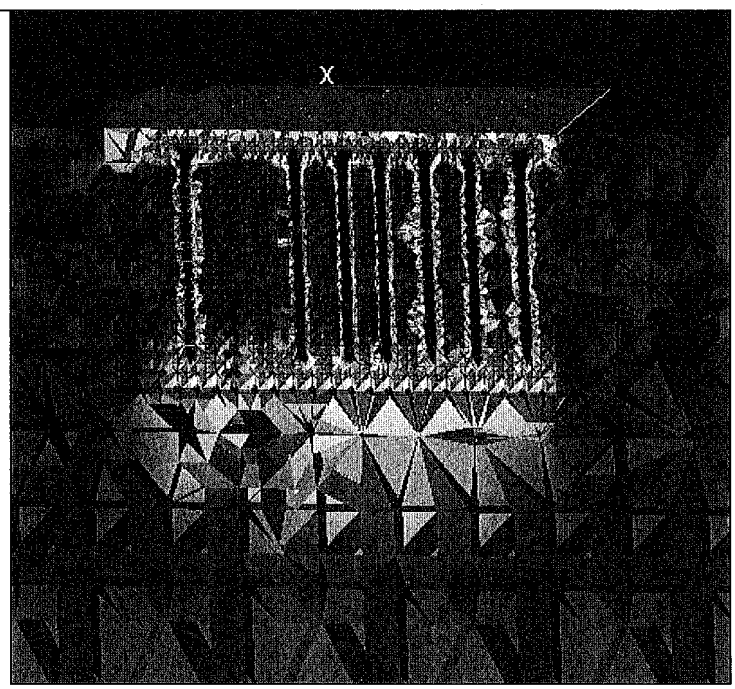


FIGURE 29. Stress bulb region produced by a piled raft system.

of many civil engineers to develop and will be seen in time. However, an enormous effort is necessary in performing the necessary verification, checking for adherence to building codes, providing example solutions with confirming experimental data, maintaining the code by responding to the inevitable "bugs," and writing and editing the supporting manuals, and preparing regular upgrades. Soil-structure interaction, more advanced than at present, would be a part of such a program. Such software will exist in the future and the future may already be here in some instances. However, the biggest problem may be finding users willing to pay the necessary fees that permit this software to be developed.

Current versions of the software based on the concept of soil-structure interaction are well supported by existing technology and even better versions of these computational methods are sure to appear in the future.

Conclusions

The contributions of a number of investigators to SSI have allowed the development of a number of useful analytical tools. Procedures based on the concept of SSI are in use widely and are important components in the solution of a number of important problems. SSI has attracted the interest of investigators in many countries and improvements are regularly being suggested. The future of SSI is bright and is expected to be more important in the future to geotechnical and structural engineers.

ACKNOWLEDGEMENTS — *The author thanks colleagues at Ensoft, Inc., for many contributions to the technology presented in this article. Thanks are also given to many investigators, some referenced here, who have contributed to the advancement of SSI. This article was originally presented to the Boston Society of Civil Engineers Section/ASCE as the Casagrande Lecture given on October 13, 2004.*

LYMON C. REESE *has done extensive research in the field of geotechnical engineering, principally concerning the behavior of deep foundations. He has conducted pioneering work in performing field studies with instrumented piles and drilled shafts, and has developed analytical methods now widely used in the design of deep foundations. He has authored over 400 technical papers and reports, and presented numerous invited lectures and talks around the world. Dr. Reese was elected to the National Academy of Engineering in 1974, and was elected an Honorary Member of ASCE in 1984. He spent three years on the faculty at Mississippi State University before joining the faculty of the University of Texas in 1955. He was Chairman of the Department of Civil Engineering from 1965 until 1972, and Associate Dean of the College of Engineering from 1972 to 1979. He continues at the University of Texas as the Nasser I. Al Rashid Chair Emeritus and Professor of Civil Engineering.*

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Replacement of the Cranston Viaducts Using Spliced Bulb-Tee Girder Technology

The application of bulb-tee girder technology provided economy and durability for this project, and its construction provided valuable experience.

FIROOZ PANAH & H. RAYMOND PALMER

The Rhode Island Department of Transportation (RIDOT) retained an engineering firm in 1995 to perform environmental studies, preliminary engineering and final design for its Freight Rail Improvement Project (FRIP). This project is an undertaking by RIDOT, in conjunction with the Rhode Island Economic Development Corporation, to enhance the infrastructure within the state. These enhancements will enable the development of the Quonset Point/Davisville Industrial Park as a major port container facility in the northeast United States. When completed, this project will provide a freight dedicated track (FDT) capable of transporting double-stacked containers and triple-stacked car carriers within the Northeast Corridor (NEC) right of way (ROW) and parallel to the existing Amtrak electrified, high-speed mainline. In addition to the construction of the new FDT, the project will require other new construction as well as modifications of many bridges, slopes and utilities within and adjacent to Amtrak's ROW. Although replacement of the Cranston Twin Viaducts — northbound (NB) and southbound (SB) — was not an objective of FRIP, the geometry (see Figure 1) and condition of the existing Cranston Viaducts led the designer to recommend to RIDOT that the viaducts be replaced.

The viaducts are located 0.62 miles south of the Rhode Island Route 10/U.S. Route 6 interchange on the Cranston/Providence city line. They carry Rhode Island Route 10 over two existing Amtrak rail lines, Cranston Street and Huntington Avenue.

Built in 1959, the old viaducts consisted of seven- and eight-span, simply supported,

FIGURE 1. The center span for the old viaducts.

composite steel stringers with a concrete deck. The piers generally consisted of multiple columns bearing on massive pier walls and spread footings. Pier walls were generally placed parallel to the railroad track and skewed to the centerline of the viaducts. Each viaduct carried two lanes of traffic over Cranston Street, the Amtrak NEC ROW and Huntington Avenue. The viaducts were separated by a 1.5-inch-wide longitudinal joint, and each were approximately 500 feet long and 50 feet wide. Spans varied in length with a maximum span of 90 feet. Each structure carried a 2-foot, 10-inch wide concrete safety walk with a bridge railing, a 3-foot-wide right shoulder, 12- and 14-foot-wide travel ways, and an 18-foot-wide paved, sand-filled median (see Figure 2). It was assumed that the sand-filled median was provided so that a third lane could be added to each viaduct in the future. This feature facilitated the construction staging during the replacement.

Feasibility & Type Study Phases

Replacement Decision. Placement of the FDT under the existing viaducts in a manner that minimized ROW requirements would have required the relocation of the massive bridge substructure elements, which proved to be impractical. Maintaining the viaducts would have required placing the FDT 50 feet away from the nearest existing track, significantly increasing the railroad corridor ROW. Implementation of this scheme would have required extensive retaining walls and property acquisition along the tracks. It would have resulted in two new piers and spans for the old viaducts, but would have left a large portion of the viaducts in generally poor to fair condition. Therefore, rehabilitation of the viaducts was necessary.

Full replacement of the aging viaducts in their entirety was another alternative. At the request of RIDOT, a feasibility study was

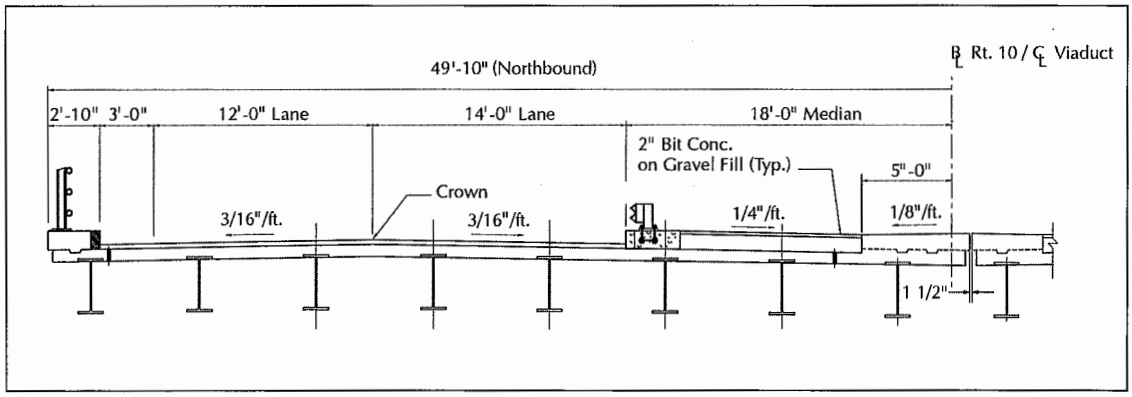

FIGURE 2. The old northbound viaduct.

performed to arrive at the best course of action by comparing the two alternatives. Although the initial construction cost for the full replacement alternative was more, it was found to be the most desirable. Implementation of this alternative would simplify the FDT construction, reduce property acquisition, eliminate vertical and horizontal track alignment deficiencies, andresult in two new viaducts upgraded to carry HS-25 loading. Moreover, when future maintenance costs were considered, the cost differential was even less. RIDOT concurred with the replacement recommendation, and directed the designer to further develop the replacement concept through a type study report.

Maintenance of Traffic. Before a replacement alternative could be considered, the designer developed solutions to ensure that both rail and vehicular traffic could be maintained during various stages of the construction. A key issue was the viability of placing four lanes of H20-44 loading on one viaduct during the construction. After examining various deck modification alternatives, the design team recommended that the NB viaduct (see Figure 2), which was slightly wider than the SB viaduct, be modified as shown in Figure 3 to temporarily carry four lanes of traffic.

In this scheme, low-density cellular concrete (LDCC) weighing 60 pcf was used to replace the existing sand-filled void in the median of the roadway. The superstructure and the substructure of the old NB viaduct were checked for the modified dead load and the additional two lanes of H20 loading. The analysis concluded that the superstructure was capable of carrying four lanes of traffic, and that only minimal shoring of the piers would be necessary while the new SB viaduct was under construction.

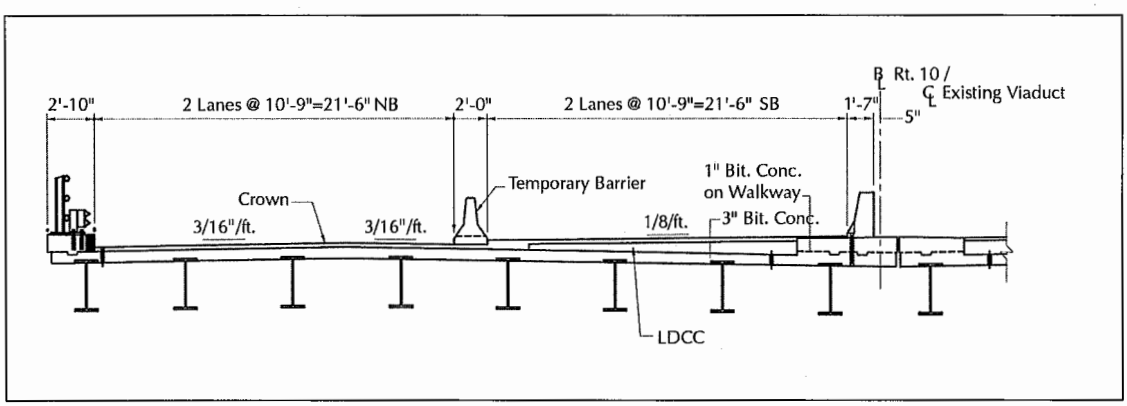

FIGURE 3. Modified northbound viaduct.

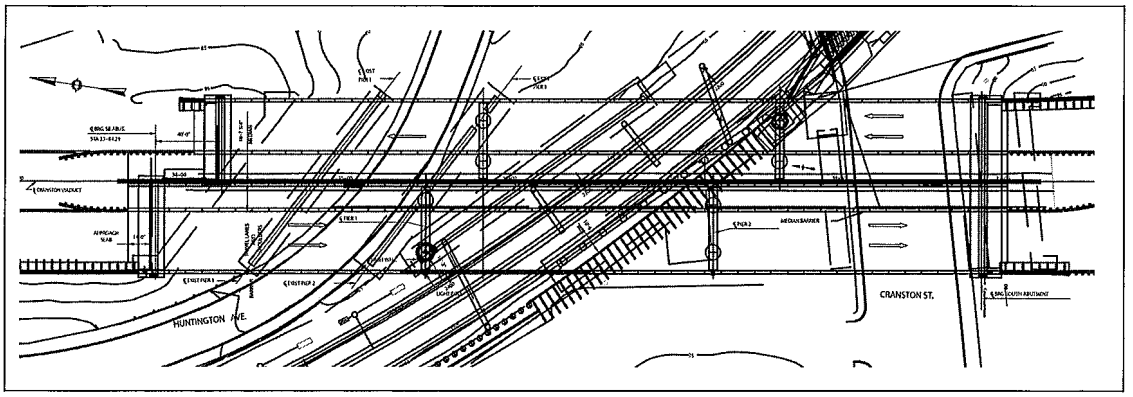

FIGURE 4. Plan for the new viaducts layout (old viaduct piers and abutments shown underneath.

Type Study Process. A type study was required to resolve several issues. Layout alternatives, superstructure alternatives and foundation alternatives were among the issues that were explored.

Layout Alternatives. For the replacement viaducts, two options were evaluated: a "skewed layout" and a "staggered square layout." In the skewed layout, the piers were placed parallel to the proposed railroad corridor and the abutments were also kept parallel to the piers; the spans of the two viaducts were therefore equal. Although this option offered some advantages, the staggered square layout was found to be simpler and less expensive overall. In this layout, the viaducts are each three-span continuous as follows:

- SB viaduct: 163-, 175- and 163-foot spans; and,
- NB viaduct: 158-, 180- and 123-foot spans.

Figure 4 shows this layout superimposed on top of old viaducts' piers and railroad tracks, and Figure 5 depicts the elevation view of the new SB viaduct. The out-to-out widths are 53.5 and 49.75 feet, respectively. The wider SB viaduct was chosen to temporarily carry four 11-foot-wide lanes (two lanes in each direction) during the construction.

Superstructure Alternatives. The following four alternatives were considered for the superstructure:

- Steel plate girders with cast-in-place (CIP) composite concrete deck;
- Precast concrete New England bulb-tee girders with CIP composite concrete deck;
- Steel trapezoidal box girders with CIP composite concrete deck; and,
- Precast segmental concrete trapezoidal box girders

These alternatives were evaluated for the fol-

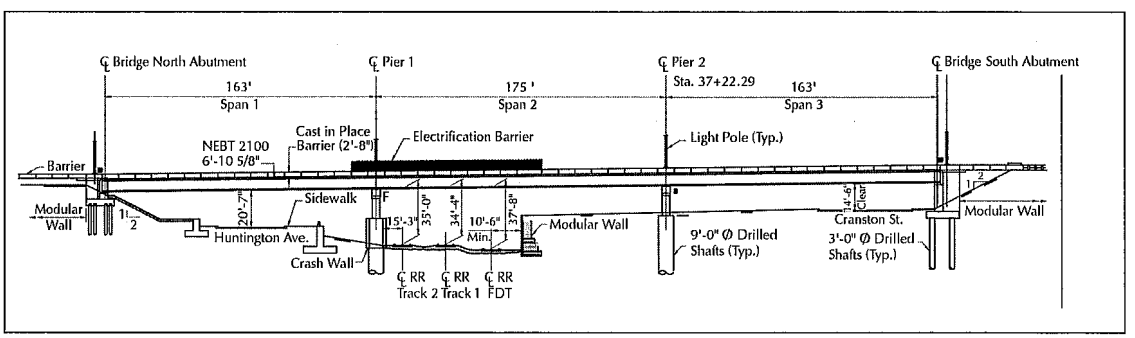

FIGURE 5. New southbound viaduct elevation.

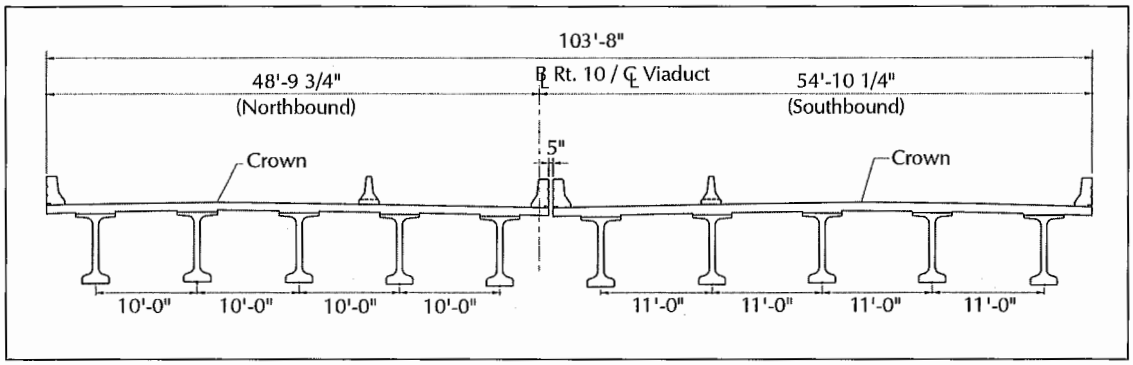

FIGURE 6. New southbound and northbound cross-sections.

lowing criteria: construction cost and schedule, constructability, aesthetics, functionality, maintenance and protection of traffic, and future maintenance cost. The second alternative was found to be the most advantageous, primarily for its lower initial and future maintenance costs (see Figure 6).

Foundation Alternatives. In addition to roadways and the NEC ROW, physical constraints (such as existing foundations) limited the placement of new piers and abutments for both viaducts (see Figure 4). In addition, due to the large axial and lateral loads of the new viaducts, it was clear that deep foundations were necessary in lieu of spread footings. Therefore, the foundation types were limited to either piles or drilled shafts. A pile system would have required large pile caps and excessive excavation supports, interfering with existing features beneath the viaducts. Therefore, because of their small footprint and their ability to penetrate obstructions, drilled shafts were found to be an economical foundation system.

Design & Construction of the New Viaducts

Drilled Shafts. During the final design, the shaft sizes and lengths were finalized as follows: for viaduct piers, 9-foot-diameter mono-shafts extending to a depth of 130 feet; for the abutments, a cluster of 3-foot-diameter shafts extending to a depth of 75 feet. Since the bedrock layer is very deep in this area, the shafts were not extended to the bedrock. The shafts were placed entirely in cohesionless soil and the loads were resisted both by side friction and end bearing. Prior to the construction of the production shafts, a 9-foot-diameter trial shaft was constructed and extensively tested using Osterberg load cells and multiple sets of strain gauges. (Further details regarding the design, construction and load testing of the Cranston Viaduct drilled shafts are provided in Reference 1.)

New England Bulb-Tee (NEBT) Girders. NEBT sections were developed in the late 1980s in response to a growing demand for more efficient precast pretensioned sections than the widely used AASHTO precast sections.[2] In order for NEBT sections to be competitive for longer spans, they had to be spliced and post-tensioned. The Cranston Viaducts presented the opportunity to demonstrate, for the first time, the potential of the NEBT sections for continuous girders in the New England region.

The span lengths are 163, 175 and 163 feet for the SB viaduct and 158, 180 and 123 feet for the NB viaduct, with girder spacing of 11 and 10 feet, respectively, as shown in Figure 6. The viaducts are designed for four lanes of HS-25 loading. During preliminary design, variable depth (haunched) girders were considered. However, precast producers generally prefer constant depth sections; therefore, a NEBT 2100-millimeter section made of 7000 psi concrete was selected for both viaducts.

The design incorporating NEBT sections was evaluated using a computer program specifically suited to analyze a post-tensioned concrete structure by accounting for the construction staging loads and by performing time-dependent analysis to estimate the creep

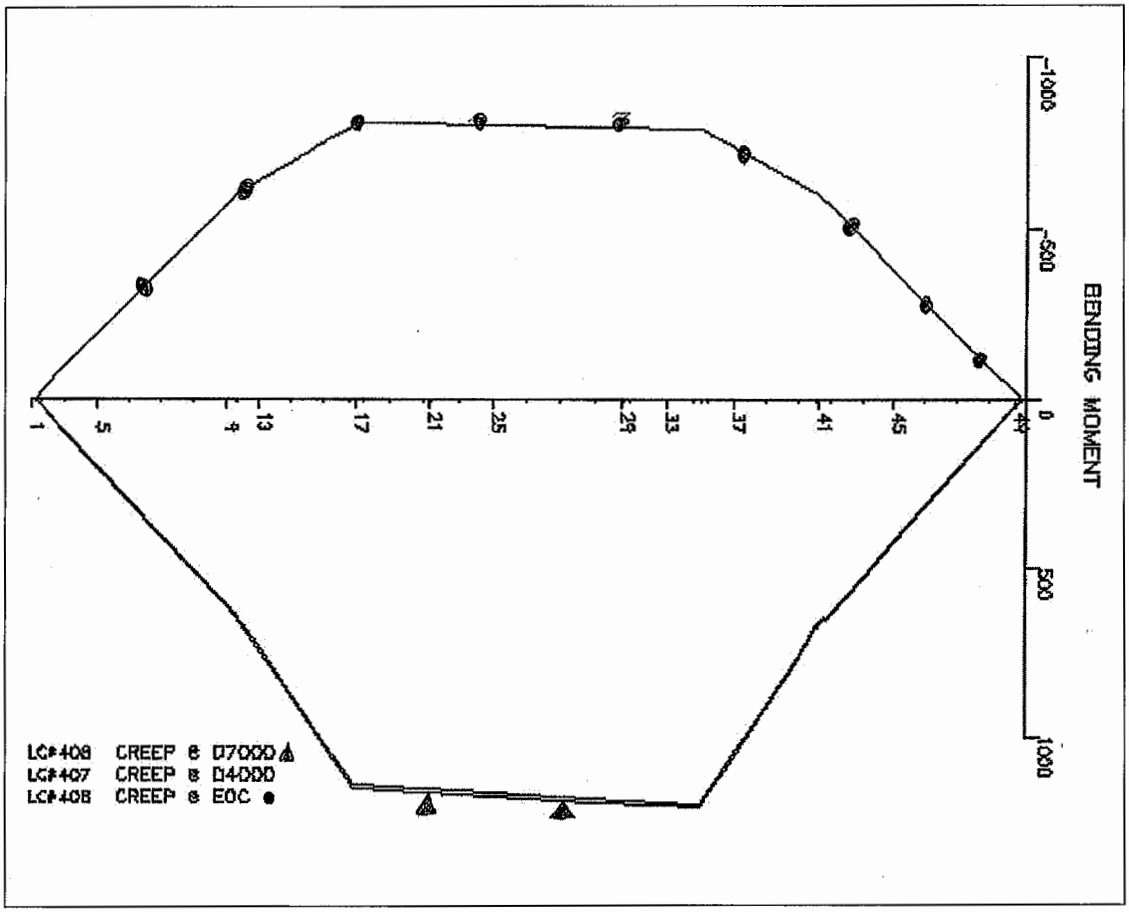

FIGURE 7. Creep-induced moments at various times.

and shrinkage of concrete. Such an analysis provides for the adjustment of prestressing forces over time. The forces in the precast girders were determined at each stage of construction and stresses were checked at all construction stages starting from the precast yard, on to each stage of erection, placement of the deck and ten years after the completion of construction. Stresses at the top and bottom of the girder and top of the deck were compared with the allowable stresses specified in AASHTO specifications for highway bridges and for segmental concrete bridges.[3,4]

The girders were designed based on the service loads and checked for the ultimate loads. Four 12.6-inch tendons were provided (two were stressed after the girders were spliced and two after placement of the CIP deck). For the service loads, the critical sections were the negative moment regions at interior supports. Forces induced by the creep of concrete and secondary post-tensioning moments resulted in positive moments at the negative moment regions and made the long spans viable (see Figures 7 and 8). The same sections were also found to be critical for the ultimate load check. In fact, in order to satisfy the ductility requirements, mild reinforcement had to be placed in the bottom flange (bulb) of the pier girders. This reinforcement consisted of twelve 24-foot-long No. 11 bars centered about the points of maximum moment locations. Apart from the critical bending sections described above, the shear stresses also approached allowable limits at the pier locations. In fact, subsequent load rating analysis performed by the engineering consultant for the viaducts indicated that load ratings were controlled by the shear stresses at these locations.

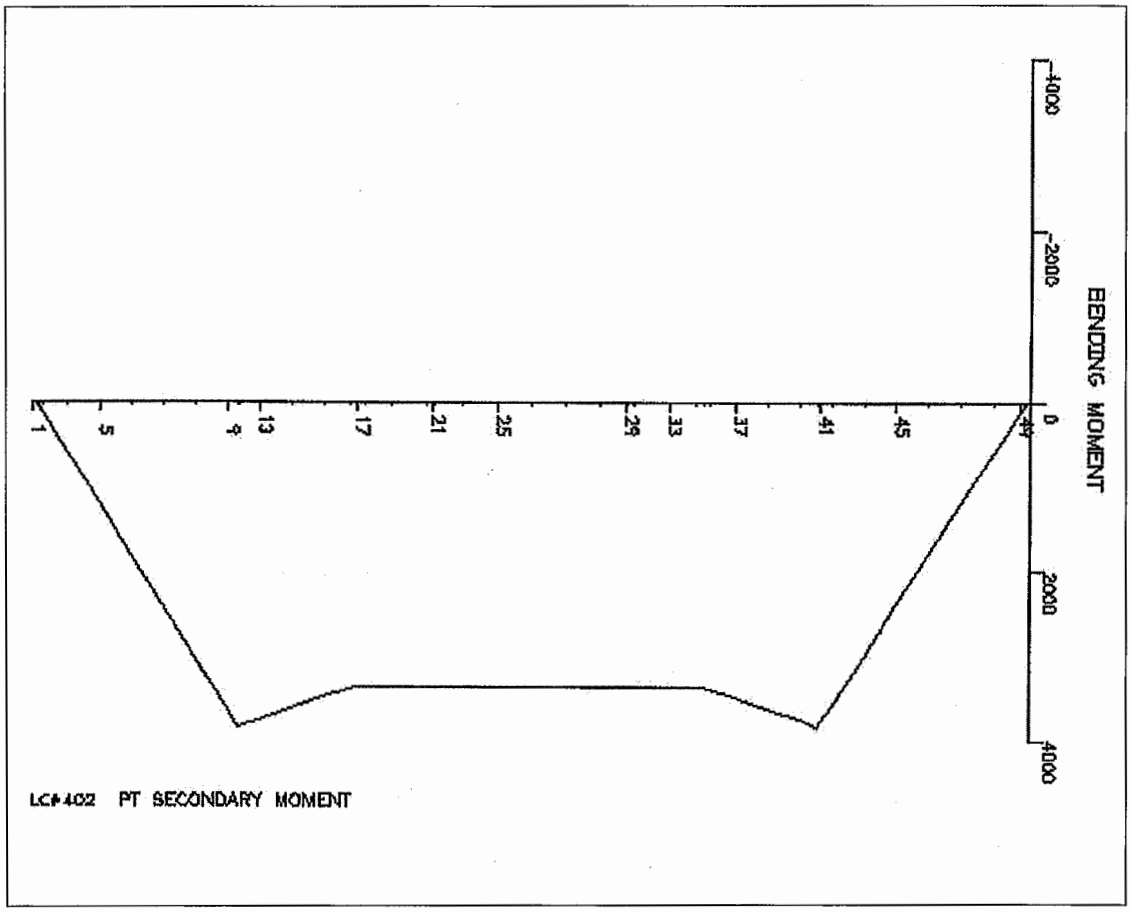

FIGURE 8. Post-tensioning secondary moment.

The splices were located near the points of counterflexure within each span and resulted in girders with reasonable lengths and weights for shipping and lifting (a maximum girder segment length of 104.5 feet and weight of 59 tons). Reinforced concrete diaphragms were provided at the pier and abutment locations within each span, and three galvanized steel intermediate diaphragms were provided in each span. The contractor elected to take advantage of these permanent steel diaphragms during the erection of the girders and used them to stabilize the girders, thus eliminating the need for temporary diaphragms. Steel diaphragms are generally easier and faster to erect than concrete diaphragms and are more advantageous where erection takes place above railroad tracks and active roadways. However, this advantage is somewhat offset by the superior durability of concrete diaphragms.

The splices were made of 1-foot-wide CIP concrete. Shear keys were provided at the ends of girders for splicing, while U- and hook-shaped reinforcing steel bars extended from the bottom bulb and top flange, respectively, into the splice zone. Shear stirrups were also provided in this zone. The post-tensioning ducts, which were each protruding 3 inches beyond the girder ends, were spliced in this zone (see Figure 9).

Erection Means, Methods & Sequences. The erection of the NEBT girders called for careful planning. The challenges included heavy girder weights (1.14 klf), adjacent active roadways and the electrified Amtrak NEC (which limited access to most areas). Several "pick points" were needed for each girder line that required the use of a very large cribbing area for the placement of multiple cribbing mats. Beam loads of 122,000 pounds with a lifting radius of 85 feet and an

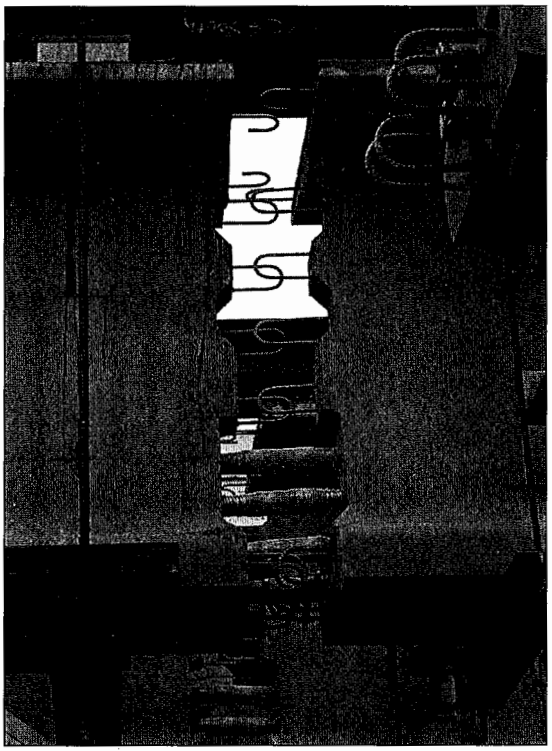

FIGURE 9. A splice detail.

Amtrak requirement that the crane be designed for 150 percent of the loads necessitated the use of a larger crane. For the picks with better access such as directly over Cranston Street or Pier 2, a smaller crane was used.

During the erection of abutment and pier girders, temporary erection towers (TET) were required at the back spans to support these girders. The TETs were centered along the splice locations supporting both the abutment and pier girders (see Figure 10). Erection brackets were required for the placement of the drop-in or center girders. These brackets were attached to the ends of these girders prior to lifting. Upon placement of the girders, the brackets were secured to the ends of the pier girders (see Figure 11). After completion of the SB viaduct, the TETs and the erection brackets were removed and reused for the erection of the NB viaduct.

The erection sequence is shown in Figure 12 and follows this order:

1. Once the abutments and piers were in place, and the TETs were placed, the abutment girders were erected, and permanent intermediate diaphragms were also installed.

2. Pier girders were then erected and the pier and abutment girders were connected at their ends using strong backs. Tie downs were provided to resist the uplift that would occur during the next step. Pier and intermediate diaphragms were also constructed.

3. Erection brackets were attached to the two ends of the drop-in girders and then the girders were hoisted in place and secured to the pier girders. After the post-tensioning ducts were spliced and rebar placed, concrete was placed at the splice locations. Once the concrete achieved a strength of 4,500 psi, two tendons in each girder were stressed and grouted.

4. The end diaphragms and the partial depth precast deck panels were constructed next. High-performance concrete (HPC) for the deck was placed after deck rebar was installed, and the two remaining tendons were jacked and grouted.

5. Finally, abutment backwalls, approach slabs and traffic barriers were constructed and the viaduct was opened to traffic.

Electrified Track & Railroad Coordination. A major concern during the design and construction was working in close proximity to the electrified Amtrak high-speed Acela service. This track alignment traversed the center of the project footprint at a significant skew (53 degrees from perpendicular to the baseline). The presence of the tracks affected many items, including the positioning and erection of the drilled shafts and piers, and the erection of the NEBT girders. The overhead catenary system providing the electricity to power the trains further complicated the erection procedure. For example, erection brackets, which were required to erect the center girders at the span splices, were directly over the Amtrak lines (see Figure 13) and at times as close as 6 inches to the catenary wires themselves.

Installation of any girders near the Amtrak lines had to be performed at night, when Amtrak allowed for two track outage and

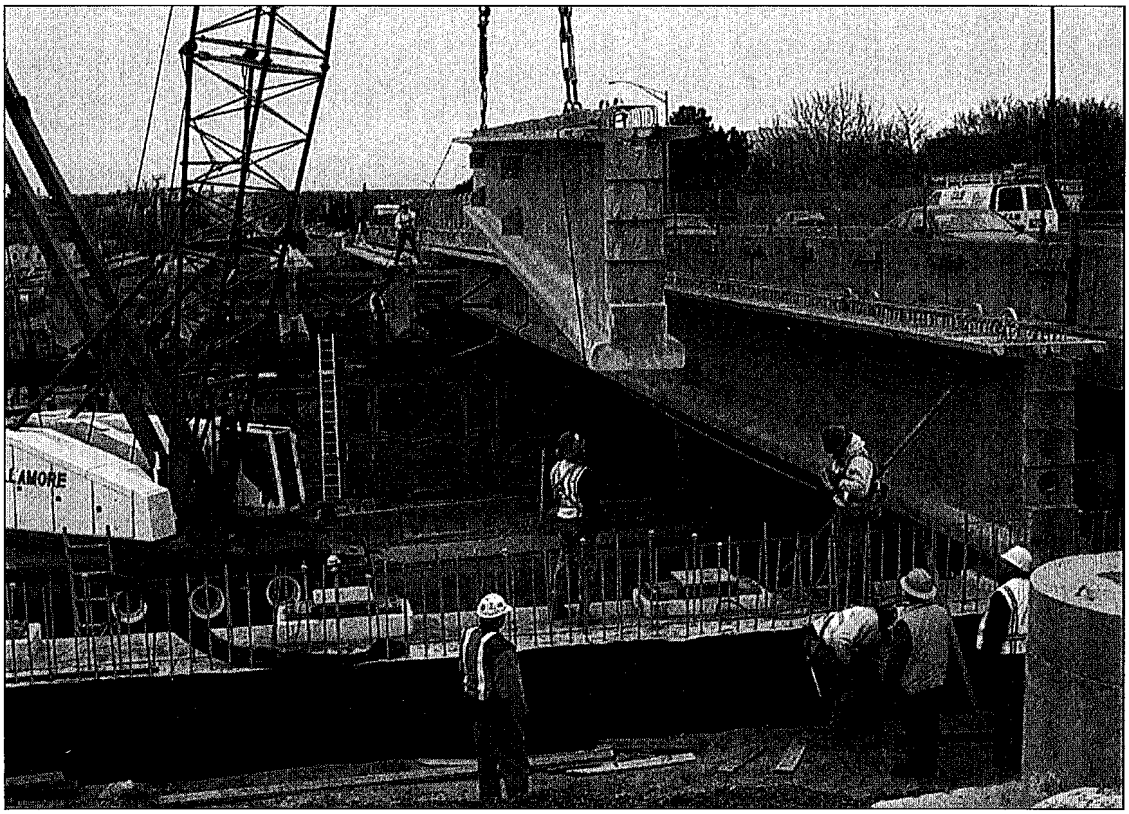

FIGURE 10. Placement of an abutment girder.

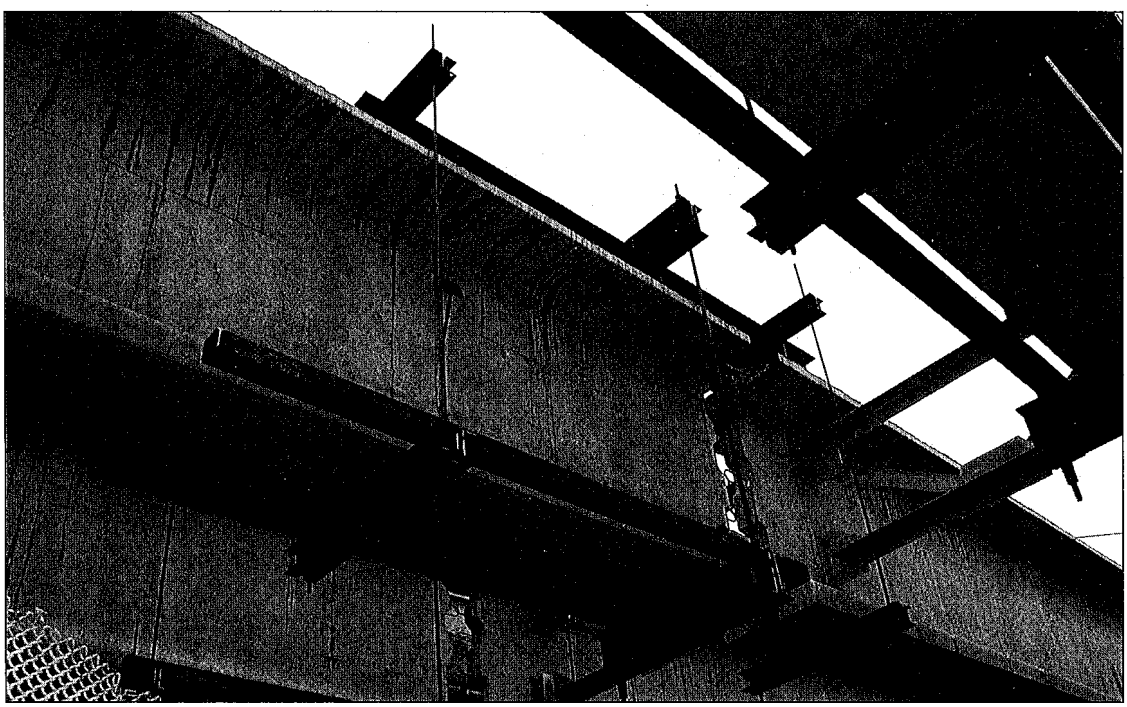

FIGURE 11. An erection bracket.

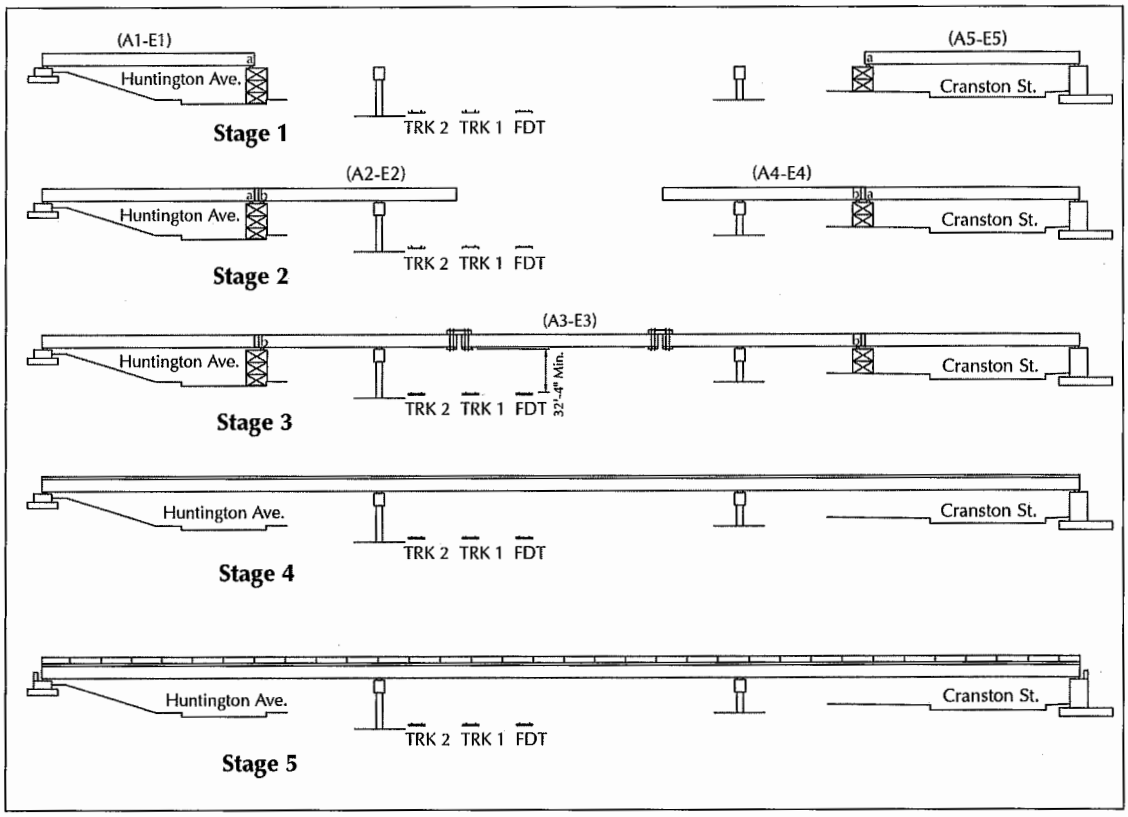

FIGURE 12. The erection sequence.

de-energizing of the lines below. This requirement limited the contractor's production rate. The original contract called for the contractor to have a window of track access between 11:30 P.M. and 4:00 A.M. seven days per week. Following the events of September 11, 2001, Amtrak added several runs between Boston and New York and was forced to perform track maintenance in the very early morning hours. The contractor was sometimes limited to a two-hour window in which to mobilize equipment near the tracks, perform the operation and demobilize away from the tracks.

The Deck System. The deck system consists of partial depth precast concrete stay-in-place (SIP) forms and HPC with a minimum concrete strength of 6,000 psi. The SIP forms were specified because the deck had to be constructed over railroad and vehicular traffic, and also to expedite the construction. During the type study phase, both galvanized steel forms and partial depth precast pretensioned panels were considered and the latter was selected for its superior durability and its compatibility with the NEBT girders. Since reflective cracks have at times been a problem associated with this system, particular attention was paid in developing details and specifications for this item. The designers carefully reviewed the available literature and studied RIDOT's and other state agencies' experience regarding precast SIP forms. A few of the specific measures included:

- Ensuring that the pretensioning strands were well developed beyond their support point.
- Ensuring that the panels were properly grouted at their support point to avoid any settlement after placing the HPC deck.
- Placement of a small amount of reinforcement (#4 bars at 1 foot) in the longitudinal direction directly above the panels.

The precaster of the NEBT girders was also the precaster for the 480 concrete deck panels

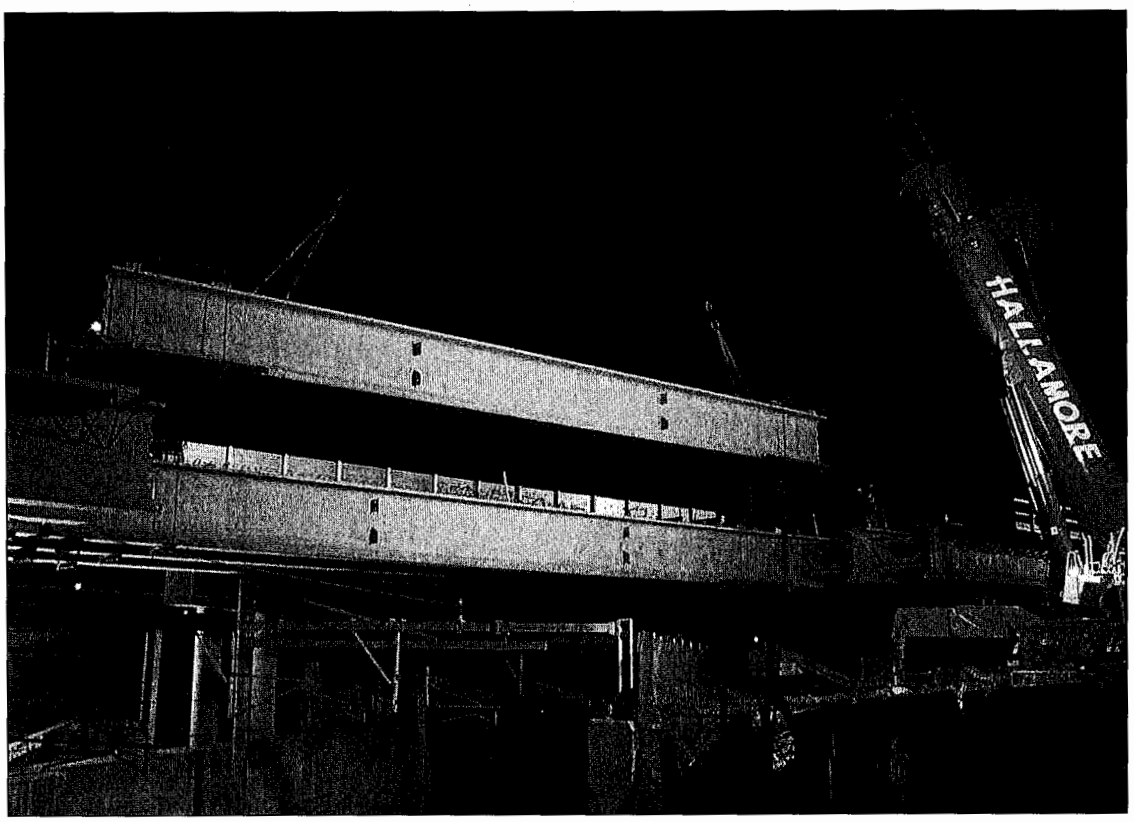

FIGURE 13. Installing girders over the electrified tracks at night.

to be used as SIP forms. These panels varied in plan from roughly 6 to 8 feet in length and from 7 feet (NB) or 8 feet (SB) in width; all were 3.5 inches thick. The panels had pretensioned single strands placed transverse to traffic and mild steel parallel to traffic.

The contractor elected to use plastic jackscrews as a temporary leveling device.[5] Once the panel was placed at the proper elevation, a foam backing previously installed between the panel and the edge of the girder acted as a form that allowed the placement of a grout bed (which would be the permanent load transfer mechanism from the deck to the girder). Once this grout bed achieved adequate strength, the plastic leveling screws were removed and the remaining void filled with grout. This system proved to be very efficient and permitted the contractor to easily adjust the panels' elevation and to place all the deck panels in less than a week per viaduct.

The HPC deck was also placed successfully. RIDOT engineers have substantial experience and expertise in the placing, testing and curing process for HPC and they have developed a thorough standard specification for doing so. To the satisfaction of all project participants, no visible shrinkage or reflective cracks have been observed on either viaduct.

Precasting of the NEBT Girders. The precaster produced 50 NEBT girders for this project (see Figure 14). The precaster was able to simultaneously cast three NEBT girders in the casting bed. On average, the production rate was one girder per day. To produce the modified NEBT girders specified for this project, the precaster simply added a flat spacer piece to the web section of the form to achieve the required added height. Also, the NEBT girders for this project required a 0.5-inch wider web to accommodate twelve-strand tendon ducts. The forms were controlled by hydraulic lines that moved them into place once the reinforcing cage was assembled. To accommodate the additional web/beam thickness, the forms were simply placed at the appropriate dis-

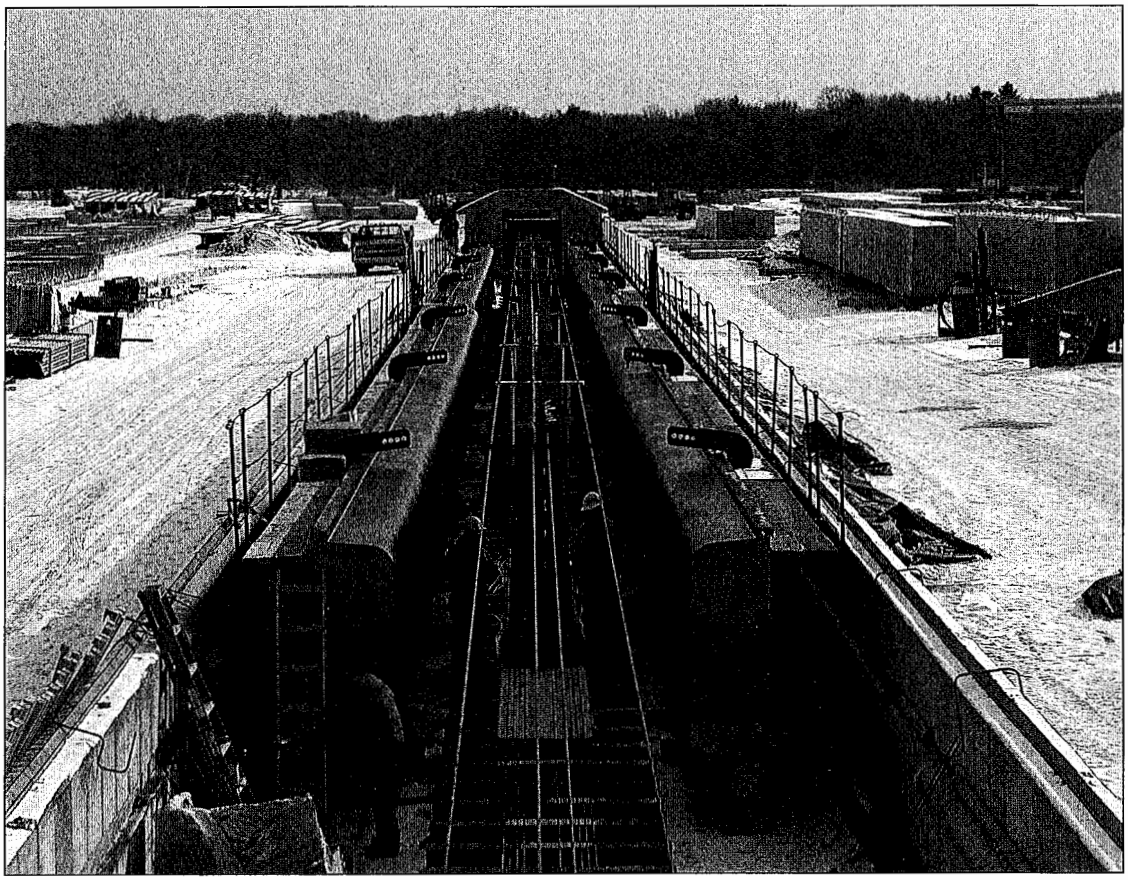

FIGURE 14. The casting bed.

tance apart. The top and bottom flanges were in essence maintained as the standard, though some templates were required at the bulkheads to properly align the pretensioning strands. No harped pretensioning was required for this bridge, but sheathed strands were used.

Post-Tensioning & Grouting Issues. As this project was going out to bid, the standards and specifications for post-tensioning grouts and grouting operations were undergoing major changes nationwide, primarily because of the problems brought to light in Florida in a few post-tensioned structures previously built. These problems were attributed primarily to grout application, such as the bleeding of grout water due to poor mixing controls (which resulted in the corrosion of the strands). Other problems that prompted closer scrutiny of grouting field practices included leakage through end anchorage protection details and an overall deficiency in the implementation and inspection of grouting procedures. According to the National Bridge Institute, the performance of prestressed concrete bridges is significantly better than bridges constructed with other materials. The revised specifications addressing these issues ensure that a higher standard is maintained.[6]

With this background, the designer recommended that the project's specifications be updated to reflect the latest improvements. Therefore, the project's specifications were revised to include:

- prepackaged grout mix for consistent blending of grout and admixtures;
- the inclusion of thixotropic grout properties for even distribution within the duct;
- the addition of downstream grout vents at high points to allow air to escape and

greatly reduce the occurrence of air pockets;
- injection of grout from grout tubes located at low points; and,
- the use of high-speed mixers and specific pumping pressures.

These items, combined with some simple on-site testing procedures, greatly increased the consistency and quality of the mix and application into the ducts. In addition, the post-tensioning subcontractor was required to have experienced technicians who possessed certifications from the American Segmental Bridge Institute.

A Problem Encountered During Grouting & Post-Tensioning

Despite the specification improvements, this installation was not without its unanticipated challenges. A serious problem was encountered during the grouting and post-tensioning operation for the two lower SB tendons (post-tensioning Phase 1) in the exterior girder. This problem happened when the contractor placed the strands and left them in the duct awaiting stressing. The contractor stressed the lower tendon and began to grout it. As grouting progressed, a leak was discovered at several of the grout tubes at the same location. The leaks were discovered to have been caused by a drilled hole through all the four tendon ducts, in effect connecting them. The problem was not identified until the grouting of the first duct took place. Grout infiltrated the second duct and locked the tendon awaiting stressing into a relaxed state. It also placed grout in the third duct, thereby reducing the diameter of the duct and eliminating the required area for installation. The contractor was unable to flush the ducts and the grout hardened in place. This incident highlights the importance of following all procedures, such as performing a pressure check of the ducts prior to grouting and the benefit of having an unlimited supply of water to flush out unwanted grout.

Several forensic processes took place to identify the extent of the problem. The contractor hired another engineering firm to determine how much of each duct was occupied. Both destructive (exploratory openings) and non-destructive (penetrating radar) tests were completed. Only an approximate limit of sound grout and concrete could be determined. Endoscopic photography images were presented to document these findings.

To remove the migrated grout in the third duct, the contractor proposed to cut an access hole into the duct and hydro-demolish the grout (see Figure 15). RIDOT agreed to view a demonstration of this procedure and determine whether the process was appropriate for use on the actual beam. A simulated beam section was created and a grout layer placed and successfully removed with the high-pressure water jets. The appropriate pressure threshold was established through this test to avoid damaging the post-tensioning ducts.

The second tendon was stressed despite having an unknown length of strand grouted in a relaxed state. Careful measurement of the expected and actual strand elongation was recorded. This information, coupled with the information provided by the engineering firm that performed the exploratory program, was useful in determining how much of the strand had been engaged. This length was determined to be 10 to 15 feet within the end span. Vacuum grouting of the un-grouted regions addressed the issue of corrosion protection; however, the loss of post-tensioning force in the unstressed length still remained. The contractor proposed to resolve this problem by external post-tensioning along the affected girder line. RIDOT and the designer allowed the contractor to do so only if the external post-tensioning were placed in between the girders in order to avoid negative visual effect (see Figure 16).

The external post-tensioning design was performed by the contractor's engineer and submitted for approval. Concrete anchorage blocks were constructed at the pier and abutment diaphragms with steel struts to prevent torsion of the eccentric anchorage. An x-ray analysis provided the exact location of post-tensioning, pretensioning and mild steel in the girder and was used to create a bolt template for the strut anchor plates. A single end jacking arrangement was installed at the abutment. Epoxy coated strands, placed within

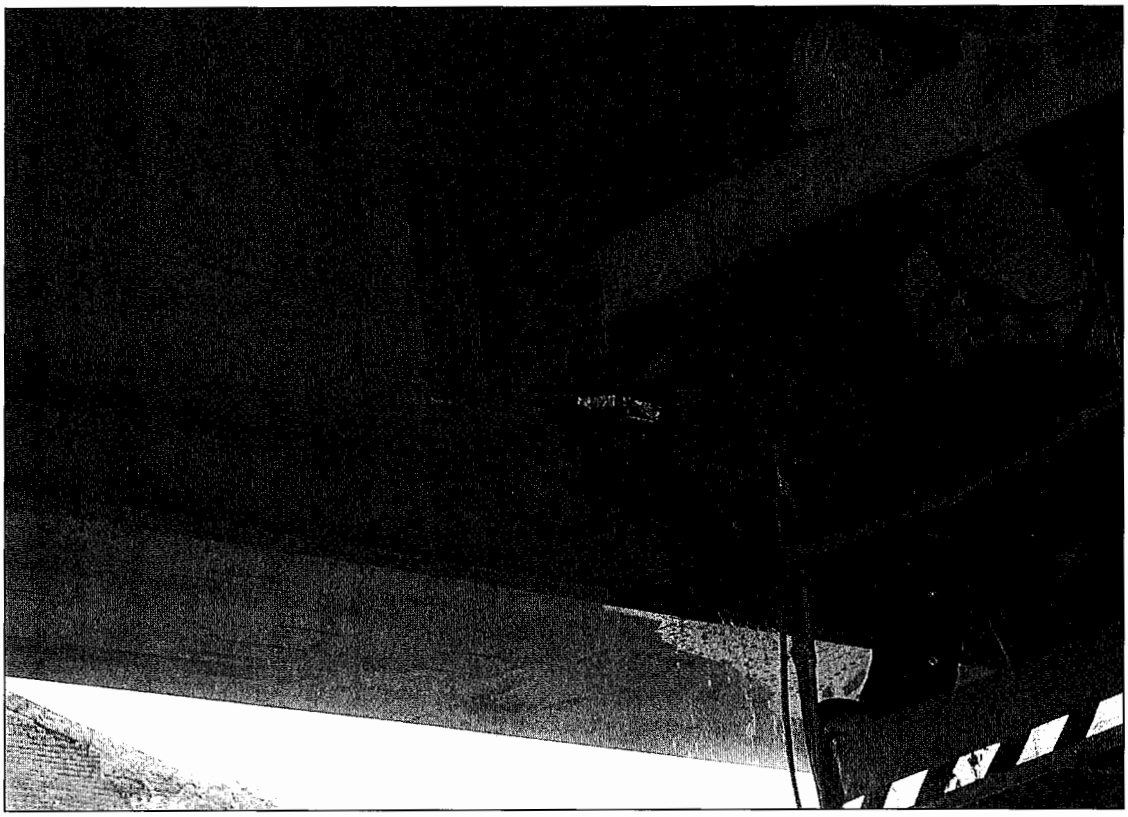

FIGURE 15. Removal of duct grout using hydro-demolition.

high-density polyethylene duct, were then stressed and grouted.

Lessons Learned

The problem encountered above highlights the importance of a well coordinated and executed grouting and post-tensioning operation. For these types of bridges, it is imperative to make sure that the subcontractor performing the post-tensioning and grouting operation is experienced; therefore, inclusion of prequalification requirements in the bid documents is essential. In this project, the NB viaduct post-tensioning and grouting operations proceeded more smoothly than for the SB viaduct because a different crew with greater experience oversaw the NB viaduct operation. The experience gained in the SB viaduct allowed the contractor to build the NB viaduct in nearly half the time that the SB viaduct was constructed.

During the course of constructing the SB viaduct, both the contractor and the designer became aware of several items that could be improved during the construction of the NB viaduct. Some of the improvements were:

- The precaster initiated a decision to pressure test the post-tensioning ducts prior to shipping the NEBT girders. This test was in addition to the pressure test required prior to grout injection.
- Tendon placement hangers were incorporated at the pier locations of the beams. The purpose of these hangers (which were made of #4 bars) was to allow the stressing of a post-tensioning duct while simultaneously protecting an adjacent ungrouted duct from the risk of being crushed. The hangers were arranged similar to a ladder and the ducts were placed at the required elevation. This procedure allowed the contractor to stress the second duct's tendons prior to grouting the first duct (as opposed to the procedure used for the SB viaduct).
- The concrete mix design for the splices

FIGURE 16. External post-tensioning for Girder A.

was revised to include 0.375-inch maximum aggregate size in lieu of the 0.75-inch size stipulated in the contract. This change ensured better concrete consolidation in the CIP splice locations.

The lessons learned through the construction of the SB viaduct proved very beneficial to the project schedule. The NEBT girders of the NB viaduct were erected in a significantly more efficient manner than for the SB viaduct. In the end, the cooperative effort of all project team members yielded a quality product for the owner.

Timeline for the Cranston Viaducts

The feasibility and type studies for the project were completed in November 1998 and the final design in November 2000. The contractor received notice to proceed in spring 2001. The SB and NB viaducts were completed and opened to traffic on December 2003 and 2004, respectively. The project was substantially completed in December 2005.

Project Accomplishments

The completion of this project represents a number of achievements:

- First application of splicing NEBT girders for a continuous bridge in New England.
- Provided the required horizontal and vertical clearances for the third track.
- Replaced aging viaducts that required constant maintenance with one that requires minimal maintenance.
- Improved load carrying capacity from H20 to HS-25 loading.
- Accommodates one additional lane of traffic to each viaduct in the future.
- Use of HPC for the deck and HSC for the NEBT girders.
- Replaced fifteen spans with six spans.
- Replaced 63 columns and six pier walls with eight columns.

- Replaced fifteen joints with four joints.
- Replaced sixteen lines of girders with ten.
- Provided major aesthetic improvements.

ACKNOWLEDGEMENTS — *The authors would like to thank Federal Highway Administration and RIDOT staff, particularly Kazem Farhoumand, David Fish and Georgette Chahine of RIDOT for their receptiveness of the post-tensioned spliced NEBT girder concept, and for their continued support and guidance through all phases of the project. The following firms and agencies participated in this project: Rhode Island Department of Transportation, owner; DMJM Harris, designer; Aetna Bridge Co., contractor; Northeast Concrete Co., precaster; Janssen and Spaans Engineering Inc., contractor's engineer; Case Foundation Company, Inc., drilled shaft contractor; Project Technologies Group Inc., owner's scheduling consultant; and, Simpson, Gumpertz and Heger performed the duct exploratory program. TANGO software was used to analyze the post-tensioned concrete structure. A DEMAG AC-500 crane was used to lift some of the girders; otherwise, a Link-Belt HC-268 crane was used. Photos are courtesy of Project Technologies Group, Inc.*

NOTE — *This article was first published in the proceedings of the International Bridge Conference in July 2005. It has been reproduced here with modifications.*

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Modeling the Effects of Soil-Structure Interaction on a Tall Building Bearing on a Mat Foundation

Examining soil-structure interaction can result in more efficient structural designs.

LEWIS EDGERS, MASOUD SANAYEI & JOSEPH L. ALONGE

The interaction of a building, its foundation and the underlying soils may have important effects on the behavior of each of these components as well as on the overall system behavior. For example, the relative stiffness of a building structure, its mat foundation and the soils that support the foundation will influence the stresses and displacements of both the structure and soil. Soil-structure interaction (SSI) effects are sometimes neglected by the use of a structural model supported on a fixed base. Other simple models assume an ideally flexible or infinitely rigid foundation on an elastic subsoil.

An investigation of the effects of SSI on the stresses and displacements in the structure and the soil of a model fifty-story steel frame structure with a concrete mat foundation bearing on a deformable soil was undertaken as a means of best understanding how to perform and apply SSI analyses. The study included investigating the effects of the stiffnesses of the building, its mat foundation and an elastic subsoil on the stresses, internal forces and displacements of the building, foundation and subsoils. The soil-structure model also considered the effects of foundation embedment.

Development of Computer Models

The study of SSI was conducted with a building frame model developed within the context of research on the effects of increased wind

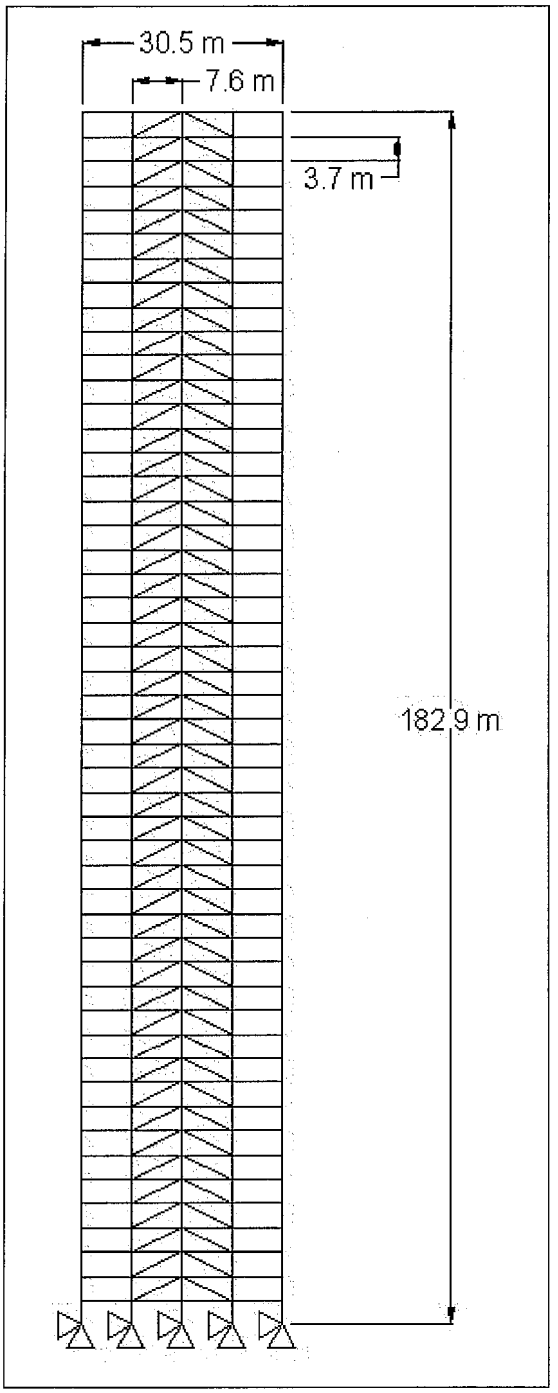


FIGURE 1. ANSYS model of the fifty-story building pin-supported on a rigid base.

loads due to global climate change on tall buildings in Boston.[1] The building model is representative of many taller buildings found in large cities.

For the initial analyses, an ANSYS model of the building frame was pin-connected to a rigid base (see Figure 1). The two-dimensional building model — 182.9 meters (600 feet) high by 30.5 meters (100 feet) wide by 0.3 meters (1 foot) — consists of 255 nodes and 550 elements. The framing system is based on a steel member with core bracing similar to that of a Pratt brace. The floor height is 3.66 meters (12 feet) and is typical of many tall buildings.[2] It uses the Beam3 as frame elements and Link1 as truss elements. The framing material is assumed to be steel and the flooring system is assumed to be metal decking with a concrete slab 10.2 centimeters (4 inches) thick. The wind loads were applied as point loads along the left side of the structure, and the vertical dead and live loads were applied together as uniform loads to the horizontal members. Gravity loads actuate the self-weight of the structure. The connection between the building and mat foundation was modeled by a pin connection at the base of the columns. Figure 2 shows a close-up of the building frame, lateral load carrying system and the pin connections to a rigid support. The structure is fixed at the base in order to examine its response (since the structure is ordinarily analyzed by ignoring the SSI).

Figure 3 shows the finite-element model of the building frame supported by a mat foundation bearing on a stiff linear elastic soil. This model was developed in order to simulate realistic foundation design conditions. The soil mass was represented by a mesh 213 meters (700 feet) wide by 152 meters (500 feet) deep, consisting of forty-two quadrilateral plane strain elements. A set of elements and nodes are added to the top of the soil mass to represent the concrete mat. The coincident nodes at the soil-concrete interface were merged so that there is full continuity with no slippage at this interface along the base of the foundation.

A two-dimensional, rather than three-dimensional, structural model was developed in order to increase computational efficiency. In order to represent a three-dimensional structure in two dimensions, the element stiffness and loadings were distributed over a unit two-dimensional slice of frame bents spaced

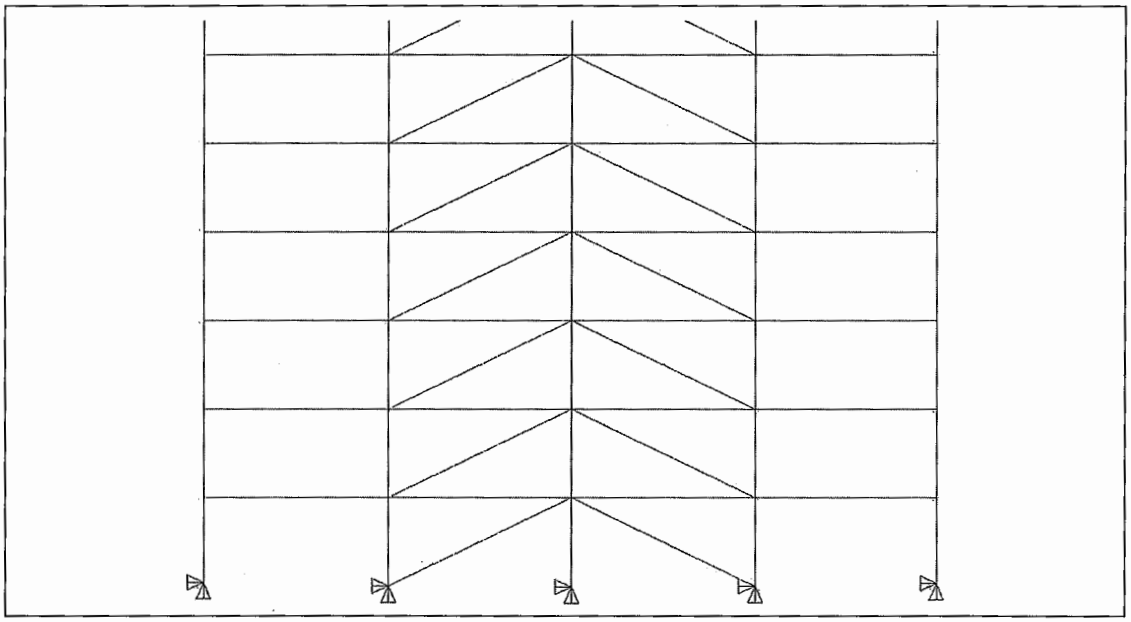


FIGURE 2. The lower floors of a fifty-story building pin-supported on a rigid base.

at 7.6 meters (25 feet). The bending rigidity, *EI*, and the axial rigidity, *EA*, of the building's beams and columns, located at the 7.6-meter (25-foot) bay spacing, were scaled by reducing the common elastic modulus parameter, *E*, by one-twenty-fifth. Thus, it was unnecessary to adjust the moment of inertia, *I*, or cross-sectional area, *A*, of each member. The material density for the discrete members was scaled by one-twenty-fifth so that full self-weight was evenly applied over each bay. No scaling was applied to the properties of the concrete mat foundation and the subsoil because they were continuous. Table 1 summarizes the material properties used for these analyses. A two-dimensional model would not be applicable for situations without symmetry in the third dimension. For these situations, full three-dimensional analyses can be performed with finite-element packages (such as ANSYS and PLAXIS 3D Foundation).

The dead, live and wind loads used in this analysis are based on the Massachusetts State Building Code.[3] Seismic loads are not considered. Dead load includes the weight of the framing, flooring, ceiling and mechanical, and partition load (assumed to be office space). The live load was assumed to be an average of the typical loads found in offices, office lobbies and corridors. The wind loads are based on Zone 3 for the Boston metropolitan area, and exposure B for towns and cities. All applied dead and live loads were transformed to represent the forces acting on a unit slice of the building model. To do so requires a one-twenty-fifth scaling of the weight of all steel elements spaced at 7.6 meters (25 feet) but no adjustment of distributed loads. The wind loads were concentrated at each of the building's floor levels. Tables 2 and 3 summarize the loads applied to the unit slice model.

Figure 4 shows the details of the finite-element model, modified to describe an embedded foundation. The computer model of the excavated areas includes both soil elements and elements representing foundation walls, mat foundation and two stories of an underground garage structure. The linear elastic model requires full continuity between the soil and the concrete walls and mat. The embedded model also includes two levels of columns and concrete beams to represent the underground part of the structure. The entire mesh includes 7,083 nodes and 6,948 elements. For the purpose of these linear-elastic analyses, excavation support issues have been neglected.

The ANSYS analyses for staged excavation and construction were performed as follows:

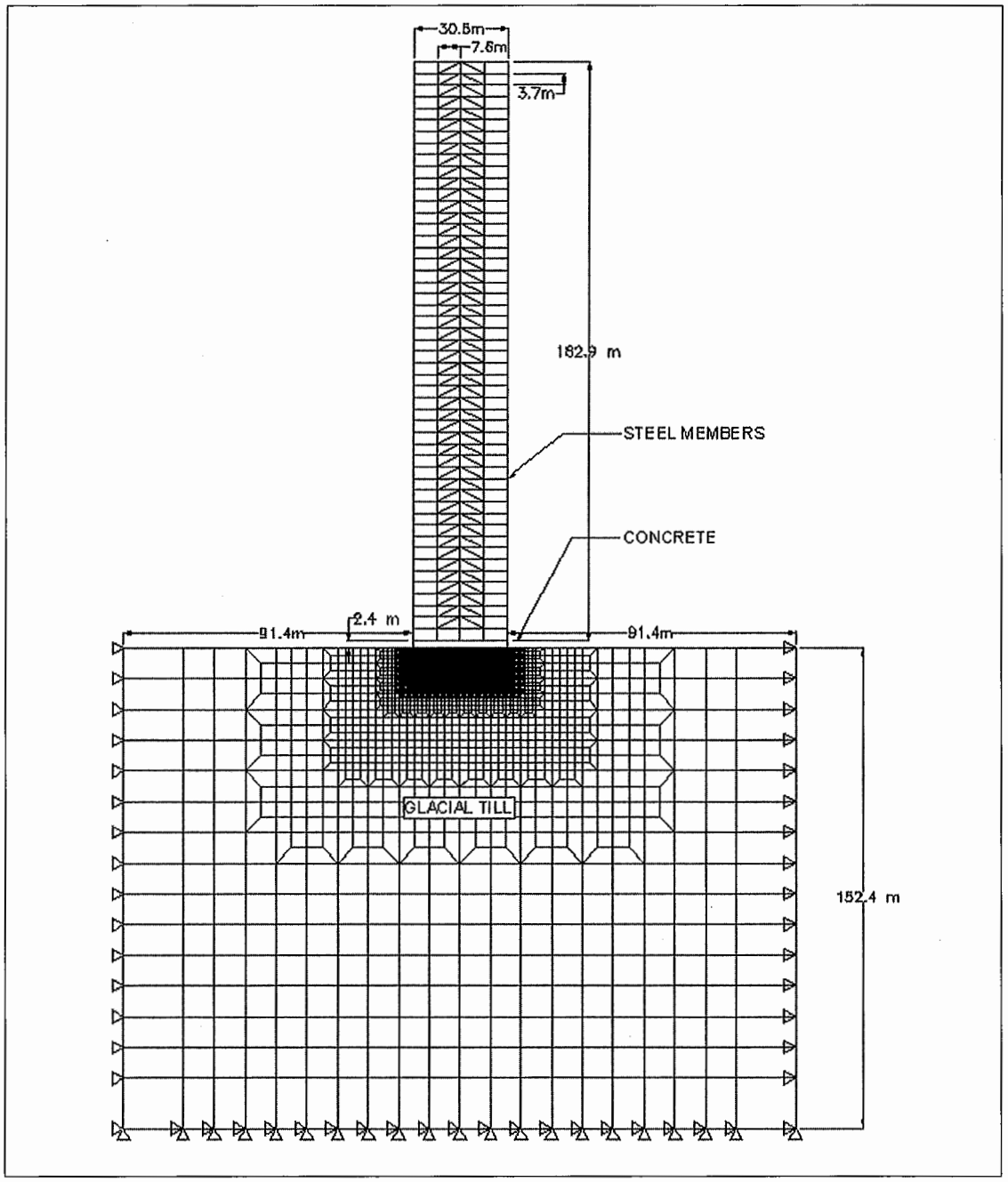


FIGURE 3. An ANSYS model of a fifty-story building supported on a mat foundation and soil.

Initial Step. Generate initial stresses in the soil (mat and building elements deactivated [see Figure 3]).

Step 1. Change material properties for the right foundation wall (soil to concrete [see Figure 4]).

Step 2. Change material properties for the left foundation wall (soil to concrete [see Figure 4]).

Step 3. Deactivate elements representing the excavated soil from between the walls (see Figure 4).

TABLE 1.
Material Properties

Material	Property	Full Value	Scaled by 1/25
Steel	γ	77 kN/m^3 (490 pcf)	3.1 kN/m^3 (19.6 pcf)
	ν	0.30	—
	E	199,946,880 kPa (4,176,000 ksf)	7,997,875 kPa (167,040 ksf)
Soil	γ	20.4 kN/m^3 (130 pcf)	—
	ν	0.25	—
	E	957,600 kPa (10,000 ksf)	—
	K_o	0.40	—
Concrete Foundation	γ	23.6 kN/m^3 (150 pcf)	—
	ν	0.15	—
	E	27,770,400 kPa (580,000 ksf)	—

Step 4. Activate mat elements and change material to concrete (see Figure 4).

Step 5. Activate the fifty-story frame (see Figure 3); apply dead and live loads (see Table 2).

Step 6. Apply wind (see Table 3) loads to the fifty-story frame.

(The sign conventions for the results presented here are that downward displacements [settlements] and compressive stresses are negative.)

Linear Elastic Analyses

Verification Analyses. An initial series of ANSYS runs were performed using the soil mesh of Figure 3, without the building frame, in order to verify the operation of the program, including mesh generation and the calculation of initial geostatic stresses.[4] Loading conditions included uniform one-dimensional vertical loading across the entire top boundary as well as uniform vertical loading applied to

TABLE 2.
Summary of Dead & Live Loads (N/.3m) Applied to the Unit Slice Model

Dead Loads, N/.3m (plf)		Live Loads, N/.3m (plf)	
Floor	220 (50)	Offices	222 (50)
Ceiling/Mechanical	44.5 (10)	Lobbies	444 (100)
Partition	89 (20)	Corridors	356 (80)
Steel	71 (16)	Average	342.5 (77)
Total Applied	427 (96)	Total Applied	342.5 (77)
Total Dead & Live Load Applied = 427 + 342.5 = 770 N/.3m (173 plf)			

TABLE 3.
Summary of Wind Loads Applied to the Unit Slice Model

Floor	Load, N (lb)
1-8	1121 × 8 (252 × 8)
9-12	1388 × 4 (312 × 4)
13-16	1601 × 4 (360 × 4)
17-20	1815 × 4 (408 × 4)
21-25	1975 × 5 (444 × 5)
26-33	2189 × 8 (492 × 8)
34-41	2455 × 8 (552 × 8)
42-49	2722 × 8 (612 × 8)
50 (Roof Receives Half)	1361 × 8 (306 × 8)
Total	98,350 (22,110)

the 30.5-meter (100-foot) central foundation mat area. The computed stresses for these simple loadings agreed with the results given by classical elastic theory.[5] The magnitudes of the ANSYS computed settlements were checked using simple elastic theory, including the effects of a rigid layer at finite depth[6] and foundation embedment.[7] Note that ANSYS does not automatically correct for the settlements due to the soil's self weight; therefore, these settlements must be subtracted from the total settlements caused by any additional loads. The ANSYS computations were also checked by comparison with the stresses and displacements computed by PLAXIS, assuming linear elastic soil behavior. Alonge describes these verification analyses in detail.[4]

Parallel ANSYS and PLAXIS analyses were performed for the building-mat-soil model in connection with parametric studies for SSI

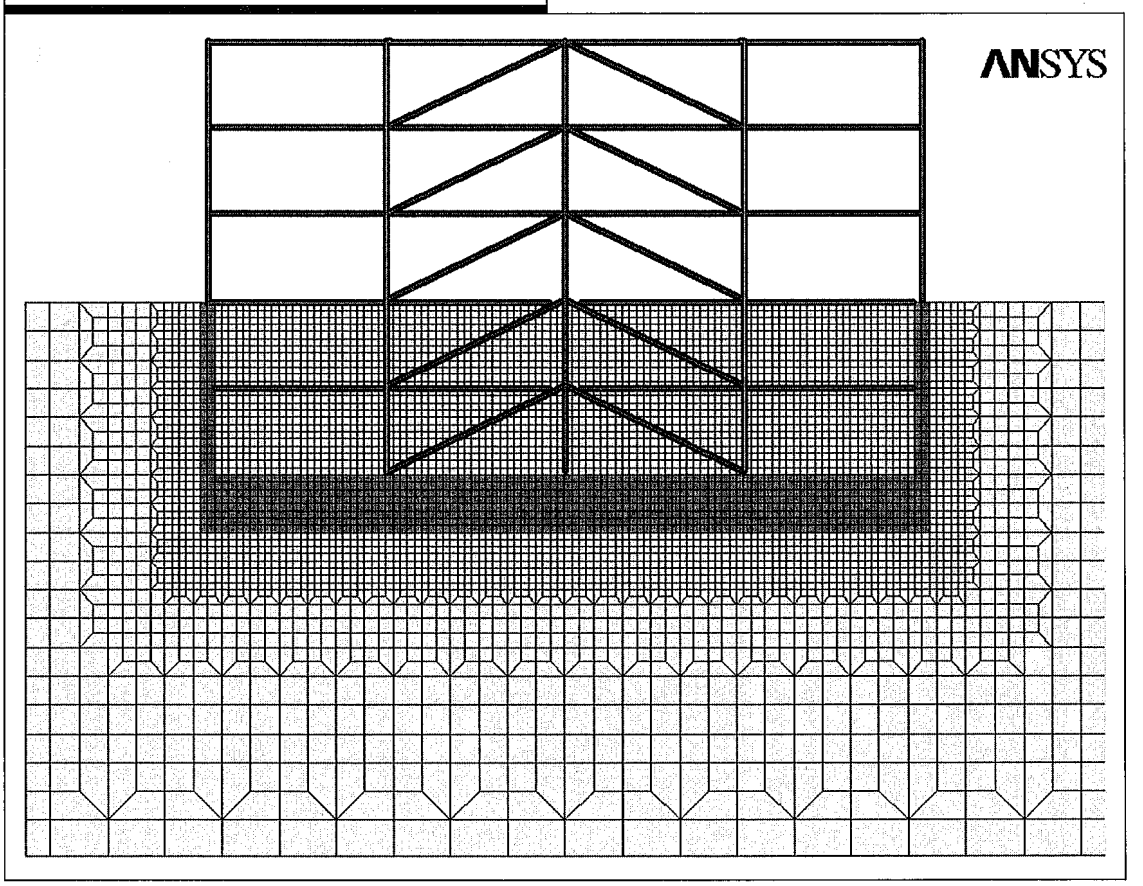


FIGURE 4. Embedded building substructure and foundation.

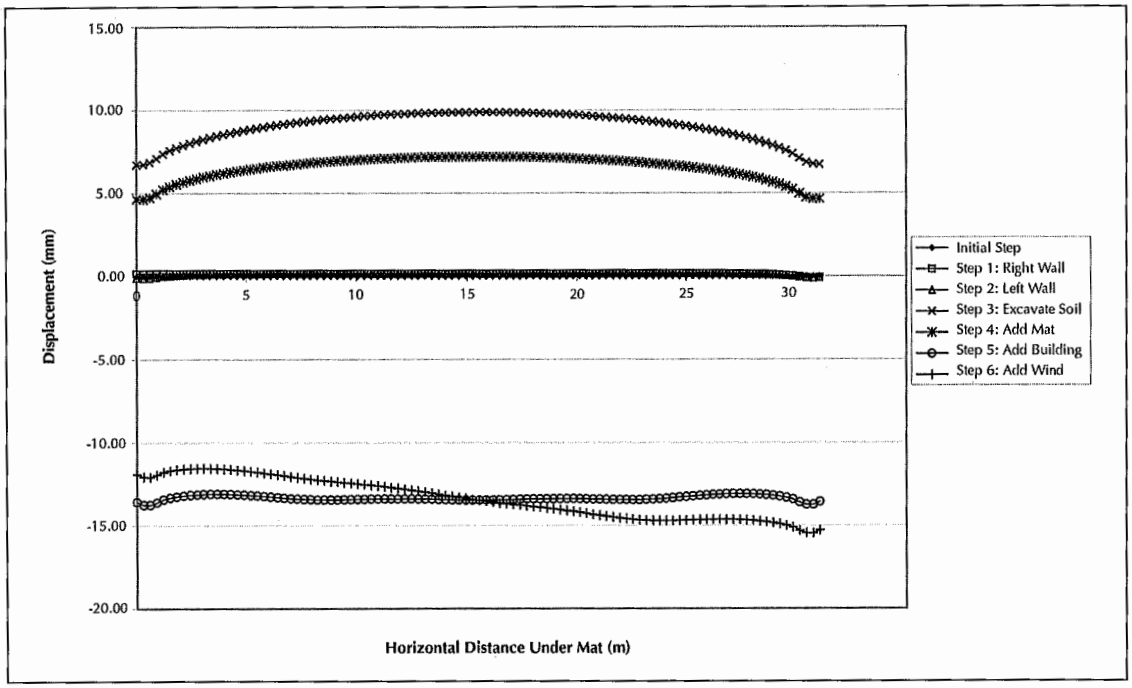


FIGURE 5. Stages of vertical mat foundation displacements.

(described later). Because PLAXIS cannot model the structure in detail, the column loads were first computed by ANSYS using the building-foundation-soil model. These column loads were then applied as an input to PLAXIS with a linear elastic soil model. The programs computed virtually identical stresses and displacements in the soil and concrete mat, which verified that the building reactions at the mat from the ANSYS model could be used in the PLAXIS model to find the stresses and deformations. This application is permitted due to the fact that the column reactions from ANSYS capture the load and stress redistributions in the combined building-mat-soil model.

Vertical Displacements (Settlements). Figure 5 shows the settlements computed by ANSYS for each step of the embedded model analysis, which had been adjusted for the settlements due to the initial self-weight of the soil. The first and second steps show little movement as the foundation walls are put into place. Step 3 shows a positive upward displacement due to the large mass of soil excavated from between the foundation walls. Step 4 shows some downward displacement due to the addition of the concrete mat. The net displacement is still up at this point because the concrete weighs less than the soil removed. Step 5 shows a net downward displacement because the weight of the building and loads are heavier than the soil. Step 6 shows the skew due to the addition of the wind loads. Note, however, that the details of excavation performance and support have been neglected for these elastic analyses.

Figure 6 shows the effects of foundation embedment on the final computed settlements. The surface model shows the same pattern of foundation settlements but with magnitudes that are almost twice as large as the settlements of the embedded foundation. The embedded foundation settles less because of the net load reduction due to the weight of excavated soil, the smaller thickness of soil beneath the foundation and the beneficial effects of confinement (e.g., see Reference 7). Note that the building frame model of Figure 1 experiences no settlements since it is pinned to a rigid base.

Stresses in the Foundation Mat. The thickness of the concrete mat, usually determined by punching shear around columns,[8] may affect the contact stresses, foundation settlements,

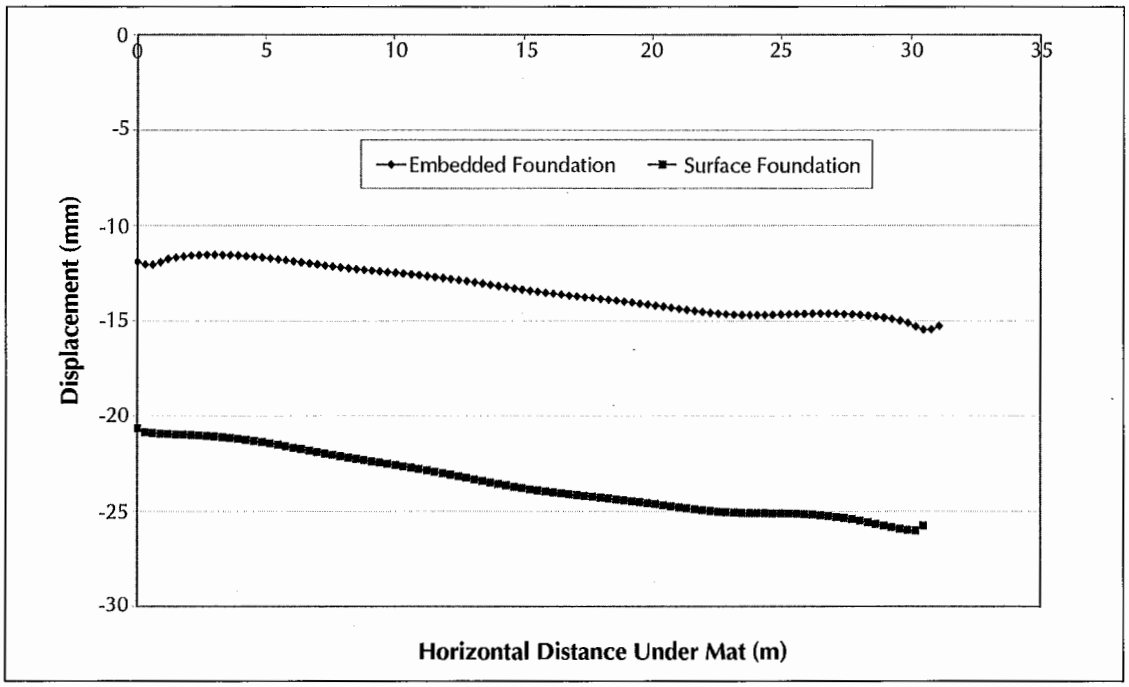


FIGURE 6. The effects of foundation embedment on computed settlements.

building deflections, and mat and building bending moments. The baseline computations were performed for a 2.44-meter (8-foot) thick concrete mat. ANSYS computations also performed for a 1.83-meter (6-foot) thick mat show similar general variations of vertical and horizontal contact stresses and settlements of the embedded foundation for the two mat thicknesses. However, the 2.44-meter (8-foot) thick mat produces slightly more uniform vertical contact stresses, as illustrated by Figure 7. The computed settlements and lateral deflection of the building centerline are almost identical for the two mat thicknesses.[4] Note also that contact horizontal shear stresses were greatly reduced for the embedded foundation because of load transfer into the building's foundation walls bearing on the foundation mat below grade. Alonge presents and discusses all of these results in detail.[4]

The bending stresses in the concrete mat are important because they are a measure of the mat bending moments. Figures 8 and 9 show the bending stresses at the nodes 0.3 meters (1 foot) below the mat top surface (see Figure 8) and the nodes 0.3 meters (1 foot) above the mat bottom surface (see Figure 9). The left-to-right wind loading causes an asymmetrical horizontal stress distribution. Building columns are located at 0, 7.7, 15.2, 22.9 and 30.5 meters (0, 25, 50, 75, and 100 feet). Significant stress concentrations were computed at column locations, compressive near the top of the mat (see Figure 8) and tensile near the bottom of the mat (see Figure 9). Between columns, the 1.83-meter (6-foot) thick mat shows smaller compressive stresses (see Figure 8) and larger tensile stresses (see Figure 9) than the 2.44-meter (8-foot) thick mat. This condition would require the use of more reinforcing bars in the 1.83-meter (6-foot) reinforced concrete mat.

Bending Moments in the Building Frame. Figure 10 shows the lower floors of the fifty-story frame for which the bending moments were computed by ANSYS for both the 1.83-meter (6-foot) and 2.44-meter (8-foot) thick mats (see Figure 11). Figure 11 shows the bending moments in the fifty-story frame due to its self-weight, dead, live load and wind loads. The bending moments are non-symmetrical due to wind loading. In general, bending moments in the beams and columns are not very different for either mat, indicating that both provide suf-

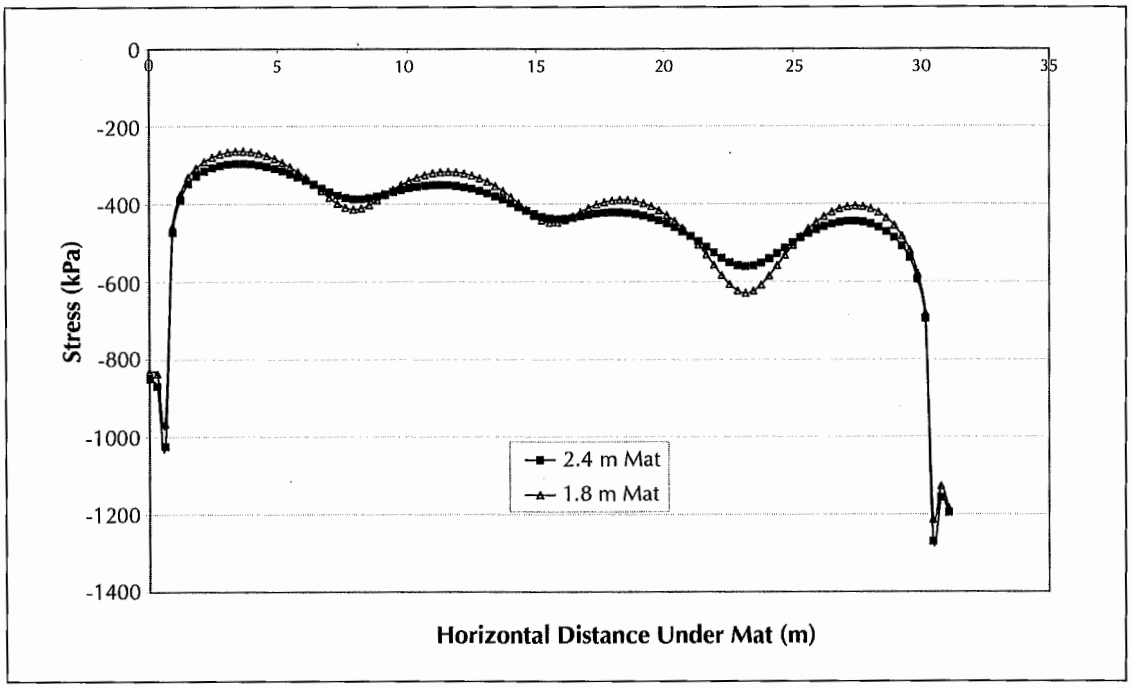


FIGURE 7. The effect of mat thickness on vertical contact stresses.

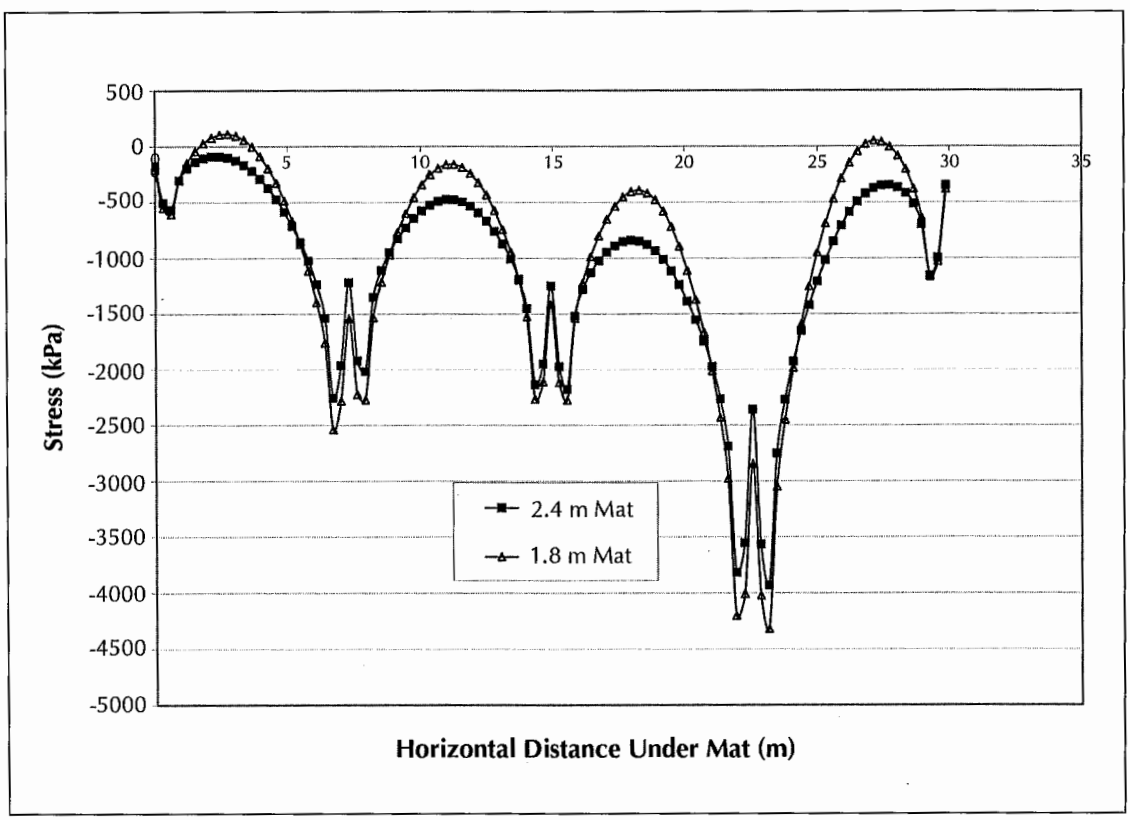


FIGURE 8. Bending stresses 0.3 meters (1 foot) below the top of the mat foundation.

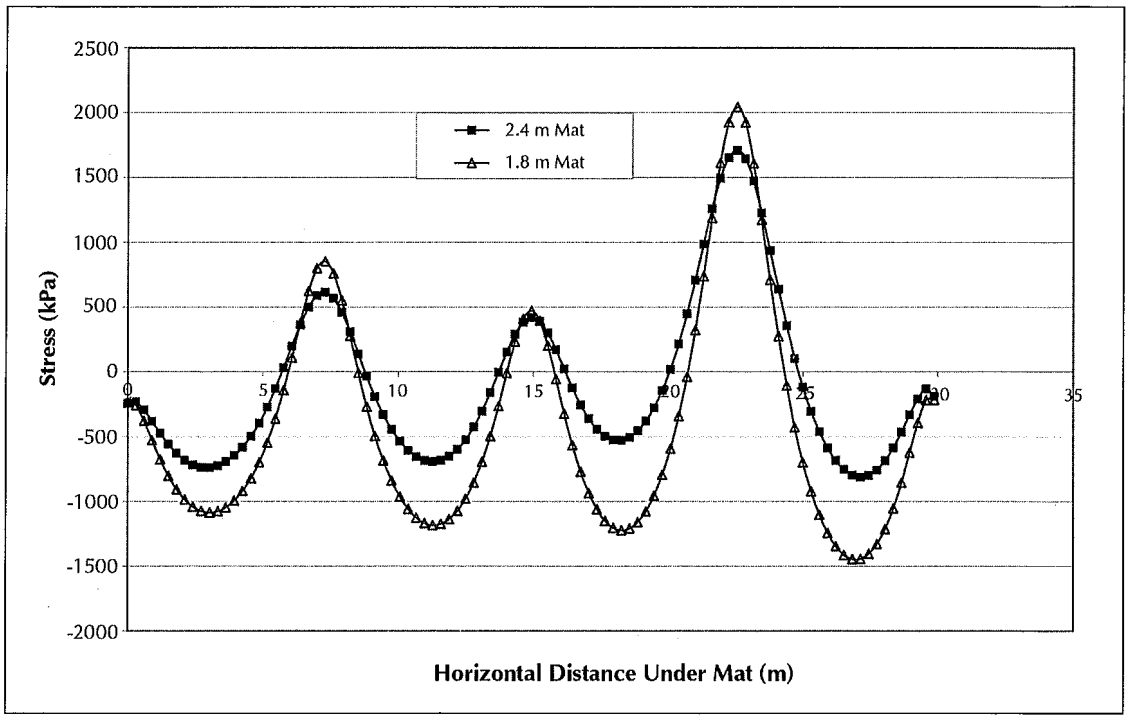
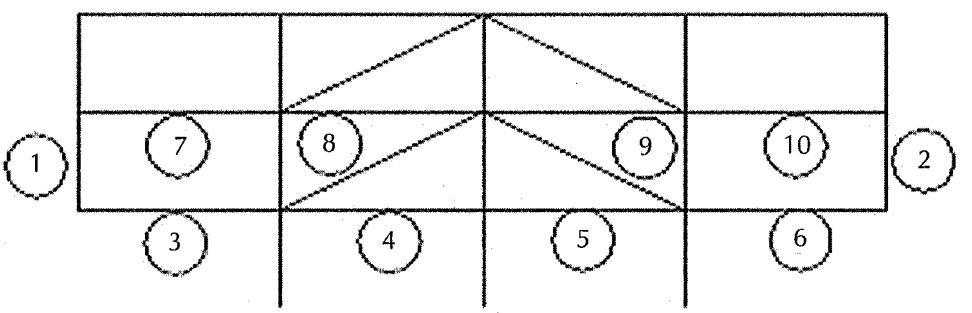


FIGURE 9. Bending stresses 0.3 meters (1 foot) above the bottom of the mat foundation.



1. Exterior Column 1, Floor 1
2. Exterior Column 2, Floor 1
3. Floor 1, Bay 1
4. Floor 1, Bay 2
5. Floor 1, Bay 3
6. Floor 1, Bay 4
7. Floor 2, Bay 1
8. Floor 2, Bay 2
9. Floor 2, Bay 3
10. Floor 2, Bay 4

FIGURE 10. The location of structural members for bending moment comparison.

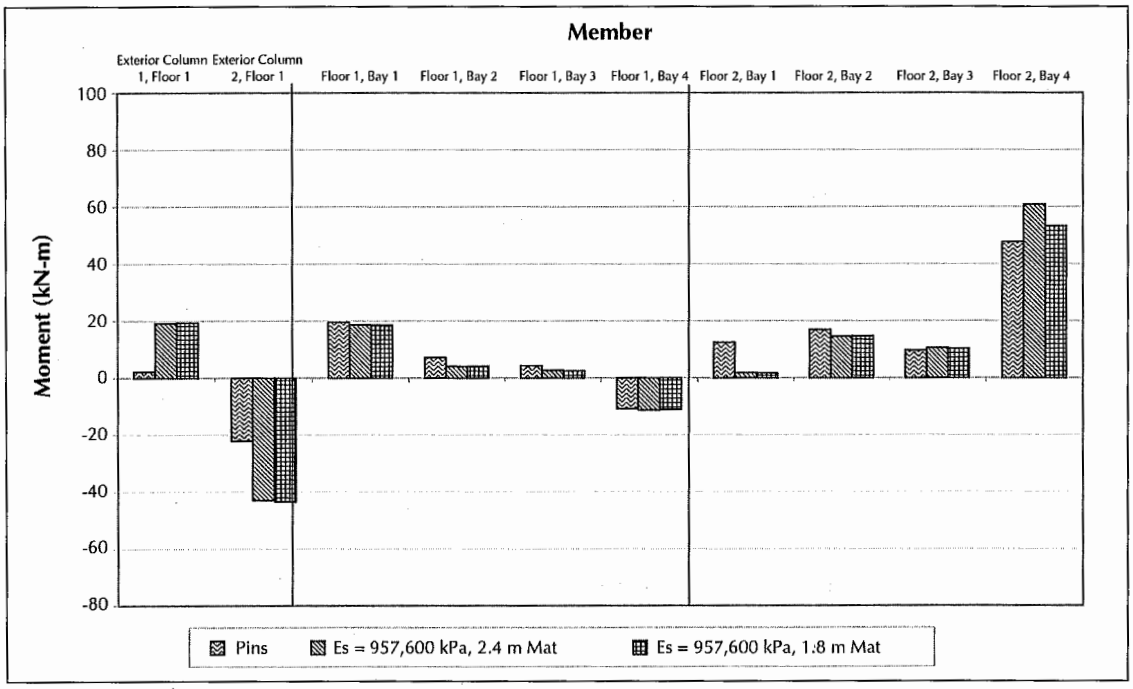


FIGURE 11. The effect of mat thickness on structural element bending moments.

ficient rigidity to uniformly and effectively support the structure. These two mat thicknesses satisfy the American Concrete Institute (ACI) punching shear requirements.

Figure 11 also shows the results of an analysis in which the structure was pinned to a rigid foundation, with no deformable soil. These results show variation in the bending moments at the leeward exterior column by a factor of 2. These results suggest the importance of modeling the foundation and subsoils with the structure in order to estimate SSI effects and the bending moments in the exterior columns and beams. However, note that the variation should be considered within the context of the overall column design using the interaction of axial loads with bending moments. Also, there are often other redundancies and load redistribution mechanisms in structural systems of tall buildings that facilitate the transfer of additional loads induced by mat relative deformations. Perhaps more significantly, the model did not directly account for construction staging and the application of loads with time. Effects predicted by the SSI analysis would likely be applied gradually as the structure was sequentially built. Therefore, the structure would have the ability to shift and transfer forces and moments more gradually. Even with these mitigating factors, however, the SSI model can lead to significant changes in structural member forces and moments.

Effects of Soil Modulus. The baseline studies have been performed with an effective stress Young's soil modulus, *E*, of 957,600 kPa (20,000 ksf) and a 2.33-meter (8-foot) thick mat. In order to investigate the effects of variations of the modulus typical of stiff soils likely to be used for the foundation support of tall buildings, the soil modulus was reduced to 239,400 kPa (5,000 ksf) and 478,800 kPa (10,000 ksf). Figure 12 shows that, with a reduction in soil modulus, the vertical stress distribution immediately below the foundation mat (0.3 meters [1 foot]) becomes more uniform, with less pronounced stress concentrations beneath the columns. Overall, the variations in soil vertical stress were insignificant. The horizontal stresses and shear contact stresses were similarly affected.[4]

The vertical displacements increase proportionally as the soil modulus is decreased (see Figure 13). Figure 14 shows the increase in

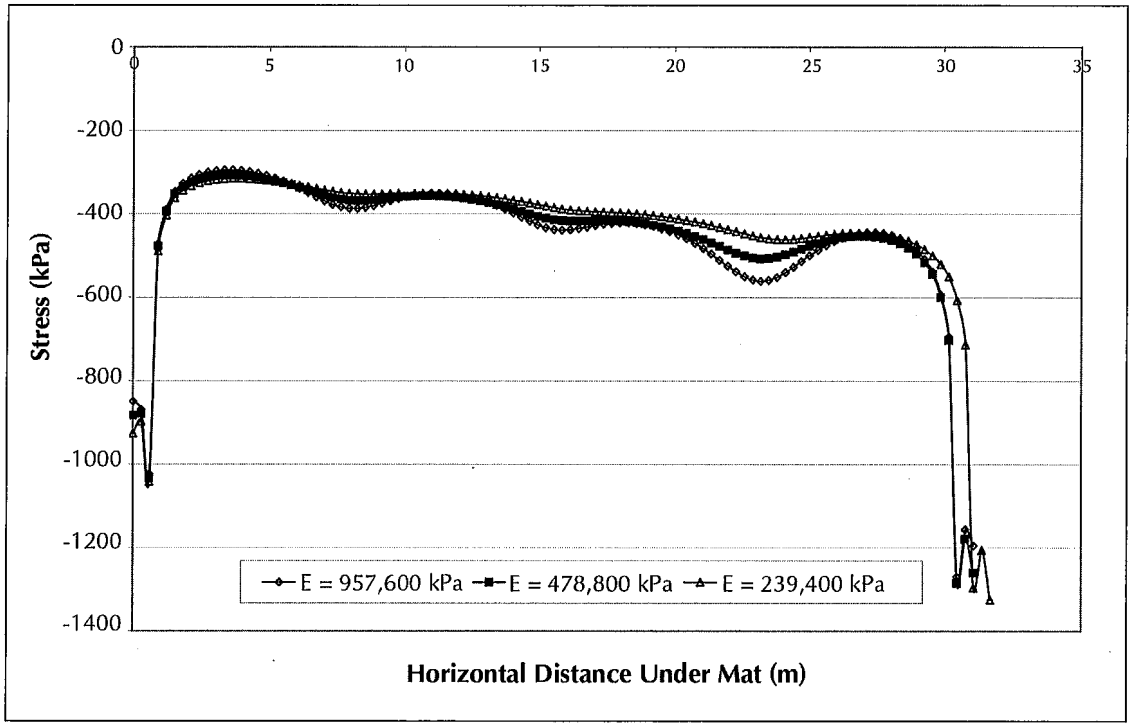


FIGURE 12. The effect of soil modulus on mat vertical stresses.

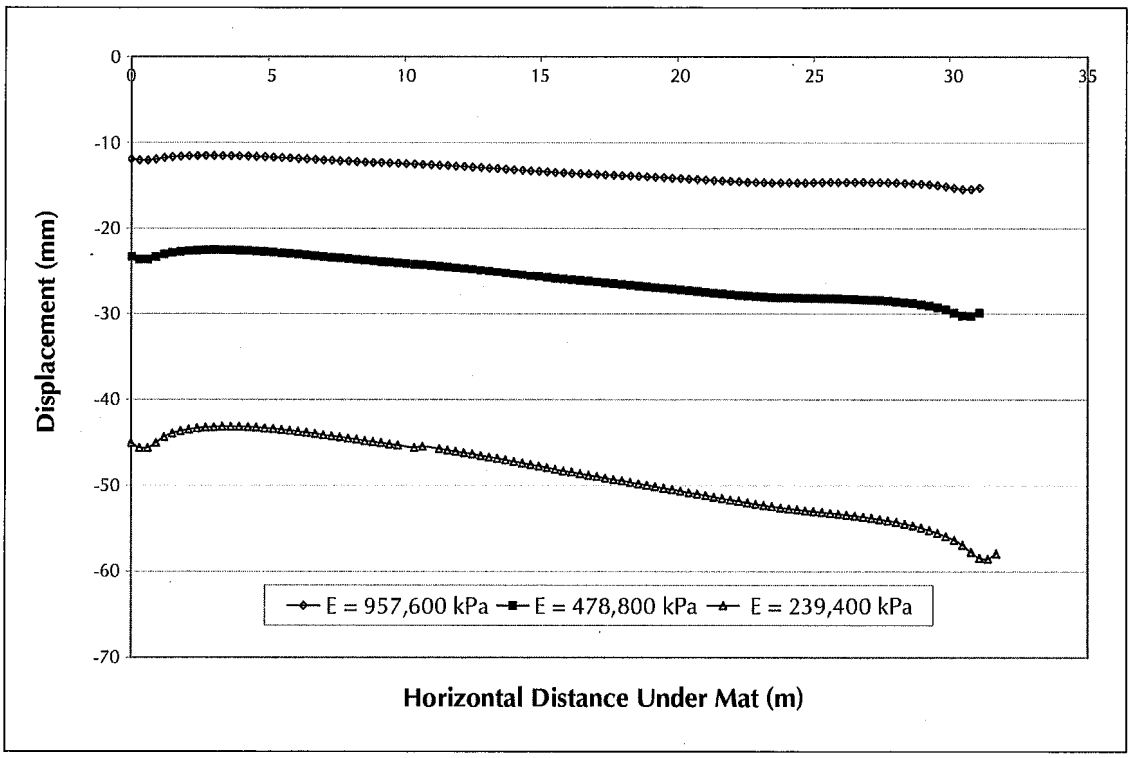


FIGURE 13. The effect of soil modulus on mat vertical displacements.

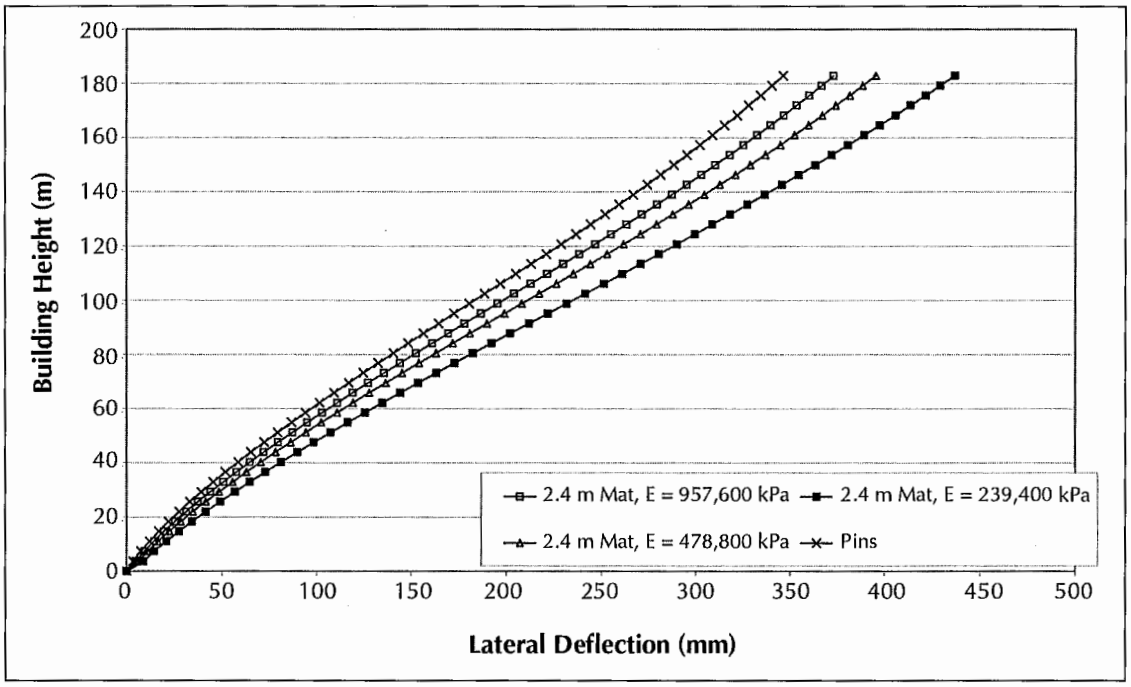


FIGURE 14. The effect of soil modulus on the lateral deflection of the building centerline.

centerline deflection with the decrease in soil modulus from 957,600 kPa (20,000 ksf) to 239,400 kPa (5,000 ksf). This figure also shows the results of an analysis in which the structure was pinned to a rigid base with no deformable soil. The resulting lateral deflection, due entirely to the deformability of the structure, underestimates the total lateral deflection that includes the SSI effects of the mat foundation and deformable soil.

Figures 15 and 16 show the bending stress distributions computed in the concrete mat at the nodes 0.3 meters (1 foot) below the mat top surface and the nodes 0.3 meters (1 foot) above the mat bottom surface, respectively. The compressive (negative) and tensile (positive) stress concentrations appearing at the top and bottom of the mat that were noted earlier (see Figures 9 and 10) are still evident. However, the concentrations become much larger with decreasing soil modulus, more than doubling for the range of soil moduli assumed. Figures 15 and 16 resemble mat bending moment diagrams superimposed with stress concentrations due to column loads.

Figure 17 shows the computed bending moments at representative members selected from the fifty-story frame shown in Figure 10, for the variations of soil modulus considered, as well as for the building frame pinned to a rigid base. It shows the bending moments in the fifty-story frame due to its self-weight, dead and live load, with wind loads. The structural behavior is asymmetrical about the centerline of the frame due to wind load. Bending moments in the exterior leeward columns (right in Figure 10) increase significantly as the foundation system softens from pins on a rigid base to soil with decreasing stiffness. Figure 17 also shows that when the wind load is applied, the bending moments in leeward exterior beams on the second floor increase considerably with decreasing foundation stiffness (shown to the right in Figure 10). When the foundation and subsoil rigidity are not modeled (pinned condition), bending moments at the exterior columns and beams can be underestimated by factors of 2 to 3. Incorrect estimates of soil modulus can lead to large errors in the bending moments computed at critical locations. Again, this increase should be considered within the context of interaction of the axial loads and bending moments, the role of other redundancies in the

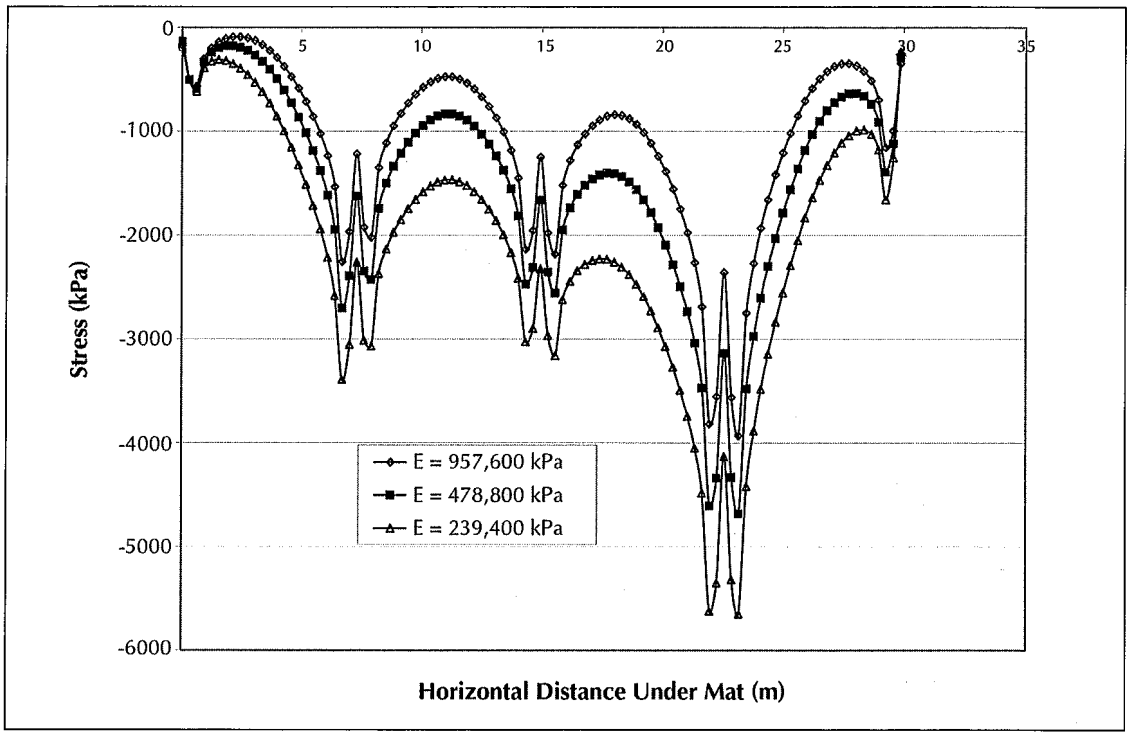


FIGURE 15. The effects of soil modulus on the bending stress 0.3 meters (1 foot) below the top of the mat foundation.

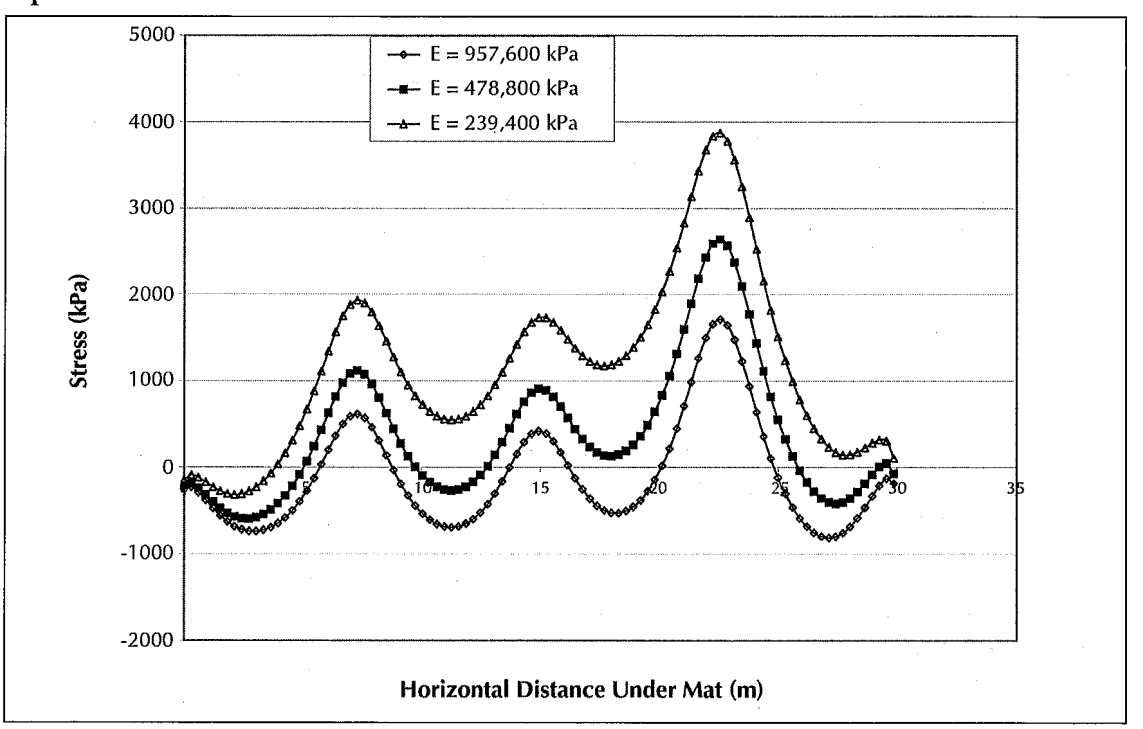


FIGURE 16. The effects of soil modulus on the bending stress 0.3 meters (1 foot) above the top of the mat foundation.

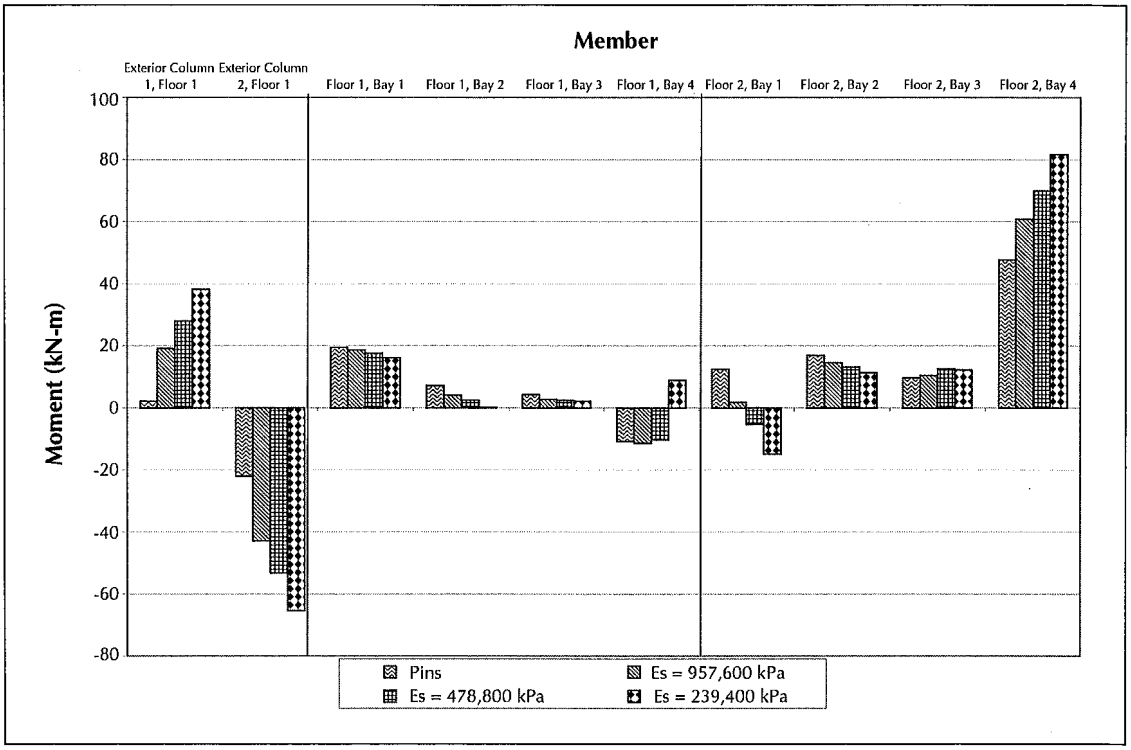


FIGURE 17. The effect of soil modulus on structural element bending moments.

structural load carrying system and considerations for the specific construction staging application of loads for the building.

Elastic-Plastic Analyses

The analyses of SSI described above have modeled the soil as a linear-elastic material. Some initial non-linear analyses utilized PLAXIS with a Mohr-Coulomb elastic-perfectly plastic soil failure criterion and interface elements that permit slippage between the soil and foundation wall when the interface strength is exceeded. Beam elements were used to simulate temporary excavation support and the excavation sequence was approximated. Because PLAXIS cannot model the structure in detail, the foundation-level column loads were first computed using the linear elastic ANSYS building-foundation-soil model of Figure 3 and then applied as input to the PLAXIS model. Table 4 shows the material properties for these analyses. Soil elements at the foundation-soil interface were assigned an interface strength equal to 0.7 of the soil strength. These elastic-perfectly plastic analyses do not account for time-dependent soil behaviors such as consolidation, secondary compression or creep.

TABLE 4.
Material Properties for
PLAXIS Elastic-Plastic Analyses

Material	Property	Value
Soil	γ	20.4 kN/m^3 (130 pcf)
	ν	0.25
	E	478,800 kPa (10,000 ksf)
	φ	38
	c	0
Concrete	γ	23,6 kN/m^3 (150 pcf)
	ν	0.15
	γ	27,770,400 kPa (580,000 ksf)

These initial analyses showed that the elastic-plastic vertical stresses are more uniformly distributed than the corresponding elastic values, with the greatest differences in the outer quarters of the foundation. The horizontal stresses and shear contact stresses are similarly affected. The elastic-plastic vertical displacements are about 25 percent greater than the equivalent elastic values of Figure 13. However, these initial studies (described in detail by Alonge[4]) have not investigated the effects of non-linear soil behavior on the stresses in the foundation mat or building frame.

Summary & Conclusions

A method was developed to model the effects of SSI for tall buildings using a two-dimensional finite-element model. In order to represent a three-dimensional structure in two dimensions, a unit-slice of the building was modeled. Continuous members (e.g., mat foundation, sub soils and floor slabs) required no scaling of material properties but the stiffness of discrete elements (e.g., beams and columns) was scaled over the entire bay spacing. A two-dimensional model would not be applicable for situations without symmetry in the third dimension.

This research has provided a comparison of two finite-element packages: ANSYS, a general-purpose finite-element analysis package, and PLAXIS, a package used for geotechnical engineering analysis. ANSYS provides a wide variety of options for the investigation of complex structural models and loading conditions. However, it is complicated to use, does not easily model the initial gravity body forces of geotechnical engineering problems and requires large amounts of computing power, file management and storage space. PLAXIS is much easier to use, has powerful automatic mesh generating capabilities and can easily model initial gravity body forces and non-linear soil behavior. However, PLAXIS cannot model complex structures and loadings in detail. Thus, the foundation-level column loads were first computed by ANSYS using the detailed linear elastic building-foundation-soil model (see Figure 3) and then applied as input loads to PLAXIS. The linear analyses showed little difference between the contact stresses and foundation displacements computed by ANSYS and PLAXIS.

Analyses were performed on a structural model subjected to three foundation conditions:

- pin-connected to a rigid foundation base;
- pin-connected to a mat foundation bearing on the surface of a linear elastic soil; and
- pin-connected to a mat foundation embedded within a linear elastic soil.

The linear analyses demonstrated the effects of SSI on foundation settlements, foundation contact stresses and stresses in the foundation mat and bending moments in the building frame.

Foundation Embedment. The embedded foundation (see Figure 6) settled less than the surface foundation because of the net load reduction due to the weight of the excavated soil, the smaller thickness of soil beneath the foundation and the beneficial effects of confinement. The ANSYS computations show that horizontal contact stresses were greatly reduced for the embedded foundation because of load transfer into the building's below-grade foundation walls.

Stresses in the Foundation Mat. Bending stresses are proportional to the general shape of the mat foundation's bending moment diagram. Significant stress concentrations were computed at column locations in the concrete mat, compressive near the top of the mat (see Figure 8) and tensile near the bottom of the mat (see Figure 9). Compared to a 2.44-meter (8-foot) thick mat, a 1.83-meter (6-foot) mat produced slightly less uniform contact stresses (see Figure 7) and significantly greater stress concentrations (see Figures 8 and 9), but almost identical lateral deflection of the building centerline.[4]

Bending Moments in the Building Frame. Bending moments were computed at critical locations in lower floors of the building frame shown by Figure 10. In general, the moments in the structural members are not very different for the 2.44-meter (8-foot) and 1.83-meter (6-foot) mats. Figure 11 shows that an analysis in which the structure was

pinned to a rigid base underestimates the bending moments at the exterior columns of the first floor by a factor of 2 compared to the analysis with SSI.

Effects of Soil Modulus, A decrease in soil modulus from 957,600 kPa (20,000 ksf) to 239,400 kPa (5,000 ksf) produced more uniform vertical stress distributions with less pronounced stress concentrations beneath the columns (see Figure 12), proportional increases in building settlements (see Figure 13) and proportional increases in the building centerline deflection (see Figure 14). The compressive and tensile stress concentrations at the top and bottom of the concrete mat (see Figures 15 and 16) more than doubled with decreasing soil modulus. When the foundation and subsoil rigidity are not modeled (pinned condition) with the building, bending moments at the exterior columns and beams can be underestimated by factors of 2 to 3. Incorrect estimates of soil modulus can lead to large errors in the bending moments computed at critical locations.

Effects of Non-Linear Soil Behavior. The importance of modeling soil with a non-linear elastic-plastic failure criterion such as the Mohr-Coulomb elastic-perfectly plastic model was investigated. The elastic-perfectly plastic vertical stresses are more uniformly distributed than the corresponding elastic values and the elastic-plastic vertical displacements are about 25 percent greater than the equivalent elastic values. However, these initial studies have not investigated the effects of non-linear soil behavior on the stresses in the foundation mat or building frame.

Importance of SSI. This research shows that for the type of structure studied, it is important to include the effects of SSI in computing building response to the applied loads. In this example, the analyses of the building frame model pinned to a rigid base greatly underestimated the bending moments at the lower floors in the frame. The linear soil modulus will also greatly affect the stresses in the foundation mat and at column locations in the frame. The effects of non-linear soil behavior on the building structural response may also be important but they have not been quantified in the preliminary non-linear computations. These results show that it is important to model the foundation and deformable subsoils with the building structure in order to capture the effects of SSI on the building structural response. This requirement implies a need for improved coordination of both geotechnical and structural engineering research and design studies, including advances in finite-element software packages such as ANSYS and PLAXIS. Further studies are warranted to examine impact of construction staging as part of the SSI analysis. The net sum of this work is a better understanding of building behavior in comparison to traditional design methods, leading to more efficient structural designs.

NOTES — *The finite-element software packages used for this study are ANSYS 5.5[3] and PLAXIS 7.1.1[7]. ANSYS, a general-purpose two- and three-dimensional finite-element program developed to analyze mechanical, thermal and structural problems, offers numerous elements and loading conditions. However, ANSYS does not automatically generate an initial geostatic stress-zero displacement condition. Rather, the initial stresses are calculated as the first of a number of load sequences and the displacements caused by the initial stresses must then be subtracted from the displacements computed for subsequent loadings in order to determine the net displacements caused by the non-geostatic loading. ANSYS was used to model the detailed structural response of the model fifty-story building to applied loads in a linear elastic analysis. PLAXIS, a two-dimensional (axisymmetric or plane strain) finite-element program, was developed for geotechnical applications. The program calculates initial geostatic stresses, considers the effects of gravity body forces and drainage conditions, and incorporates a number of non-linear stress strain models. PLAXIS operates through an easy-to-use graphical user interface and creates the finite-element model according to user-specified requirements. However, the element selection and loading conditions are too limited for the detailed analysis of building structures. PLAXIS was used for verification of the ANSYS computations and for an initial investigation on the effects of non-linear soil behavior on building structural response.*

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Enjoying the View

Beauty is, indeed, in the eye of the beholder. And, of course, many times it is not so apparent upon first glance.

BRIAN BRENNER

Over the years, the scenery adjacent to the Southeast Expressway has been spruced up quite a bit, to the point where it's actually one of the more aesthetically pleasing highway rides. Driving north from the Braintree Split (at the southern portal entrance), you pass beneath the new East Milton deck, which has reconnected the village with a park and pine trees. Then it's on to a pleasant tidal marsh, and across the Neponset River to a ride next to the new Pope John Paul II Park. This site used to contain a landfill and drive-in movie theater but is now a beautiful, grassy space adjacent to the river and estuary. Further north, you see sailboats and sparkling sunlight reflecting off Boston Harbor, with views of tidal flats and sand beaches near Marina Bay. Even the KeySpan gas tank, seemingly an industrial eyesore, has been transformed to an abstract work of art, with its rainbow painting by artist Corita Kent. Beyond that, north of Columbia Road, the decrepit garbage plant with its massive smoke stacks has been demolished. At the northern terminus of the expressway, the decaying Mass Pike interchange has been completely rebuilt by the Central Artery/Tunnel Project into a series of soaring, sculptured concrete bridges. As part of this work, an abandoned building at the old Broadway Bridge was demolished, and the crumbling bridge itself has been replaced by a nicely detailed concrete viaduct.

Remarkably, almost every mile along the expressway has been redone. Through a variety of projects, old eyesores and decaying structures have been demolished and replaced. Visitors to the city from the south get a much different and much improved visual perspective of the city, especially with the ugly garbage plant at Mass Ave demolished. It's a good thing that the sites are so much better to look at, since viewers get a close, long look with speeds of 20 mph or less during "rush" hour. Watch the seagulls soaring over Malibu Beach in South Boston and you can imagine for a moment (or much longer than a moment) that you're on Cape Cod.

Landmarks New & Old

Over the years, radio traffic reporters have developed a list of markers and shorthand comments to convey just how bad the traffic is. If, for example, the expressway is described as a parking lot from Furnace Brook, then it's best just to stay home. Probably the most prominent landmark along the route is the gas tank, an appropriate icon for the traffic reports. The tank is filled with fuel, and the

cars sitting in the traffic jam burn it off, in a sort of commuting equilibrium (yes, and I know it's liquid natural gas in the tank and gasoline in the cars, but literary latitude is being taken here). So it was a bit startling in May 2005 to see new competition for icon status when a windmill popped up along that linear shrine to fossil fuel consumption. The windmill was built by the International Brotherhood of Electrical Workers (IBEW) Local 103. It provides electric power to some adjacent buildings.

The windmill project is ingenious on many levels. For provision of power, it's placed at a good site next to windy Boston Harbor. For educational purposes, the project is a winner. Instead of talking about renewable energy, IBEW went out and built a windmill. As an addition to the scenery, the windmill is quite beautiful. It is a sleek, stark kinetic sculpture, with a simple cylindrical shaft and white blades. Recently, blue illumination was added at night and the turning blades glow. Funding for the project was provided, in part, by the Massachusetts Renewable Energy Trust, which in turn collects funds from electricity rate payers. In a sense, the fossil fuels are helping to subsidize development of renewable energy sources.

Dressing Up, or Dressing Down?

The IBEW windmill was planned and built during debate over another, much larger windmill project proposed for Nantucket Sound. For this project, the developer has proposed to build a large swath of windmills in the shallow waters north of Nantucket. According to the developer, the wind farm would provide upwards of three-quarters of the electric power needed for Cape Cod, Nantucket and Martha's Vineyard. The proposed farm would be a non-polluting, renewable energy source that would greatly reduce demand for fossil fuels. What's not to love?

Apparently, there's plenty not to love. A citizen's group, the Alliance to Protect Nantucket Sound, was formed to protest the project. The crux of the protesters' argument appears to be that the windmills would convert pristine Nantucket Sound into essentially an industrial wasteland. On an earlier version of the group's website (*www.saveoursound.org*), a black and white artist's rendering pulled no punches in its depiction of a despoiled seascape, with sailboats menaced by the sinister-looking windmills, beneath a gray, smoky sky that probably contains clouds but was drawn to suggest industrial waste.

There are many challenging issues to be debated by a proposal to build a wind farm in the Horseshoe Shoals area of Nantucket Sound. The issues include who is entitled to build in and profit from what is public land (or in this case, water), how wildlife (in particular, birds) is protected, how facilities like the wind farm should be regulated and what arrangements would be in place to ensure that the windmills are properly maintained and removed, if necessary.

But of all the issues to be debated, the one that seems to most energize opponents is the concern for the visual pollution and the potential industrialization of Nantucket Sound. This concern seems slight and unintentionally ironic. It is a slight argument, because windmills are dramatic and quite beautiful, at least in many viewers' eyes. For some, myself included, they can be less visually intrusive than the behemoth yachts that currently ply the waters of the sound. It is ironic, because many of the people protesting the wind farm are perhaps some of the highest per-capita consumers of energy in the world. I'm not aware of a study to document this, but if you visit Nantucket, you will see very large homes, very large vehicles on the cobblestoned streets and enormous yachts cluttering the harbor. Cape and Island residents use a lot of energy, almost all of it provided by non-renewable sources. Using wind power to satisfy these needs would seem to be a good thing in that context. Deepening the irony, remember that one of New England's greatest environmental disasters occurred in 2003 when a barge dumped oil in Buzzards Bay, despoiling significant areas of the coastline. The barge was en route to the Canal Electric Generating Station in Bourne, which generates electricity for Cape Cod.

For alternatives to the proposed wind farm project, the Alliance to Protect Nantucket Sound states on its website:

There are other far more appropriate ways to achieve the same emissions reductions as the Cape Wind project. In fact, given that we do not have a current need for additional power, we should start with an aggressive energy efficient and conservation program. We should also explore land-based wind options prior to going offshore due to economics, risk, and regulatory process.

This argument is a curious one. It seems to be a combination of not-in-my-backward, there's no need for it anyway and use of conservation. Even if it were true that there's no need for additional power, it would still be good to replace current fossil fuel consumption with wind power. However, opponents to the windmill farm have a good point in emphasizing the need for aggressive energy conservation. But for now, the SUVs filling the narrow Cape and Island streets keep getting bigger and bigger. A good start would be to replace these vehicles with compact cars and smaller sedans. Some more modest sailboats and dinghies instead of five-story yachts would also be helpful.

Back along the expressway, the white blades from the new IBEW windmill lazily turn in the sunlight, spinning a bit faster than the long lines of traffic moving in fits and starts. A news radio helicopter hovers above, broadcasting a dire prognosis for commuters. This morning, it's another slow roll to Columbia Road. But now there's something new and nice to look at along the way.

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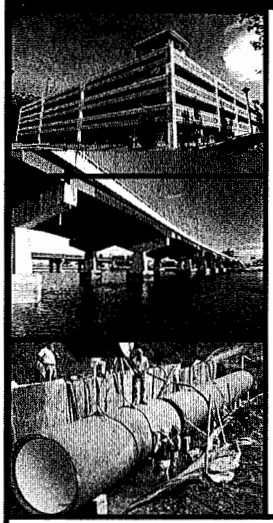

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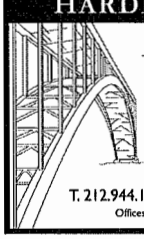

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IBEW Local 103's wind turbine spins in the breeze
over commuters headed south on I-93.
Photo by Ben Thayer.