Pile-Supported Bridge Foundations Designed for Impact Loading

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Bridge piers designed for impact loading from barges or other vessels represent a large investment of highway dollars in southeastern states, particularly Florida. Recent experimental research on the behavior of groups of piles subjected to large lateral loads has provided insight into the influence of group effects (pile-soil-pile interaction) for such loading conditions. Procedures currently used in design do not reproduce the observed behavior; thus, improved design techniques are needed. Outlined is a design procedure recommended by the authors that is based on the results of recent well-instrumented, large-scale experiments. Recommendations for modification of *p-y* curves as a function of row position in a rectangular group arrangement are presented. The procedure is relatively simple and should be easily incorporated into design but should be considered an interim approach because of the limited experimental data that is available on the subject. An example use of the method is provided.

Southeastern coastal states, particularly Florida, have a large number of bridges that span navigable waterways. Current practice is to design exposed bridge supports for impact of barges or other vessels using an equivalent static lateral load applied at the waterline. In general, the foundations for these bridge piers are designed to consist of large groups of piles, and the design for this impact loading often controls the number and types of piles selected. A great deal of uncertainty exists regarding the magnitude of the applied load; however, this paper deals only with foundation design once the equivalent static lateral load is established.

Groups of piles with typical spacings of around three diameters on center will have a capacity to resist lateral load that is less than the sum of an equal number of isolated piles due to group effects (pile-soil-pile interaction). Recent experimental research on large-scale groups that were instrumented (*1–4*) indicates that the distribution of load to the piles and the individual pile response is dominated by row position within the group more than any other aspect. Analytical research using three-dimensional nonlinear finite element analyses (*5*) has indicated similar behavior and suggests the procedure outlined in this paper. Currently used design techniques that rely on either elastic pile-soil-pile interaction or treat the group as a single entity (*6*) do not account for this type of behavior and thus do not realistically model the problem.

Presented here is a procedure for design of groups of piles subjected to large lateral loading that incorporates existing techniques for analysis of individual piles subjected to lateral load along with existing techniques for structural analysis of a bridge bent. The method uses empirical factors for modification of *p-y* curves for the piles based on row position. These factors are derived from back analysis of the relatively few available large-scale experiments and analytical research and apply only to loads of large magnitude [i.e., loads large enough to produce deflections of ½ to 1 in. (12.7 to 25.4 mm) or more]. A method for conveniently incorporating the predicted pile response into a routine structural analysis is also described.

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GENERAL PROBLEM

In general, ship impact will occur on a structure similar to that shown in Figure 1. A particular bridge bent is supported by two or more columns that in turn are supported by a rectangular group of piles. The load from impact of a vessel will occur at or near the waterline, as shown in Figure 1. Clearly, a great deal of uncertainty exists regarding the kinetic energy of the impacting vessel, energy transferred to the bridge on impact, and so forth. These considerations are beyond the scope of this paper; however, the policy of the Florida Department of Transportation (DOT) is to estimate an equivalent static load using the simplified procedure outlined in a study sponsored by the Louisiana Department of Transportation and Developments (DOTD) (*7*). This strategy will at least provide a rational and consistent basis for design.

For the foundations shown in Figure 1, the major portion of the load will be resisted by the impacted pier, but some of it will be transferred through the structure to adjacent foundations. The problem is thus one in which structural analysis and foundation design are interrelated. Important parameters relative to pile selection will be shear force and moment at the top of the pile and lateral deflection under the design load condition. In general, pile-supported foundations subjected to vessel impact are not governed by rotation of the pile cap and the resulting axial loads in the piles. Batter piles tend to be efficient only for large groups in relatively deep water because of the variable direction of loading, the tendency for batter piles to produce load concentrations within the group, and the increased costs associated with installing batter piles over water. Piles are typically spaced at around three diameters on center.

RESPONSE OF PILE GROUPS

Experimental research by the first author and others has indicated that a group of piles spaced at approximately three

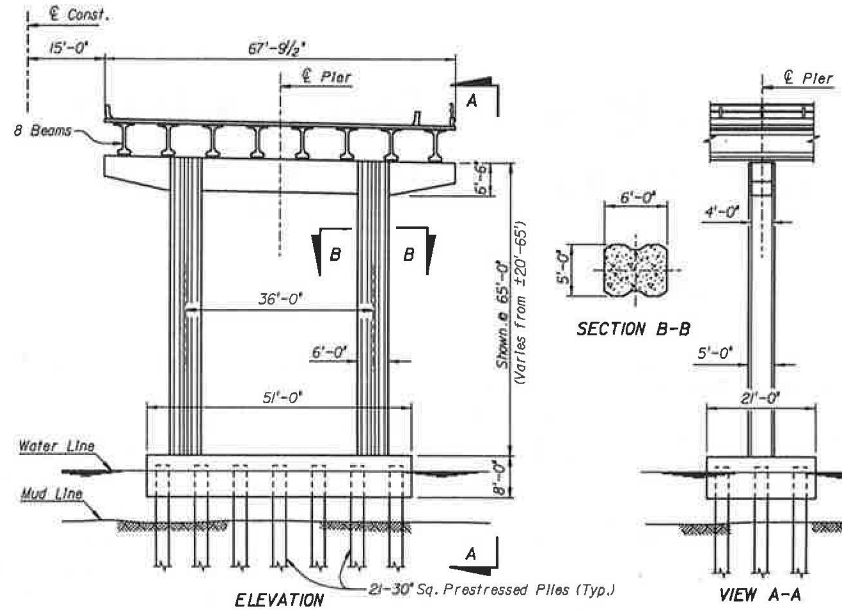

FIGURE 1 Impact load on a pile-supported bridge bent.

diameters on center can be expected to undergo significantly greater deflection at a given load than would an equal number of isolated piles. This reduced efficiency is due principally to the effect of "shadowing," in which the piles in trailing rows can mobilize only a reduced soil resistance because of the influence of the piles in the leading row. The piles in the leading row undergo a slightly reduced soil resistance due to stress overlap and superposition of strain in the soil ahead of the piles. Bending moments tend to be maximum in the piles of the front row because of the bias in load distribution at a given deflection.

One of the widely accepted approaches to the design of piles for lateral loading is the use of a Winkler model for the soil utilizing nonlinear *p-y* curves to represent the soil response. A rational approach to design of groups uses this approach, with modifications of the *p-y* curves used for an isolated single pile to account for group effects. The *p-y* curves for a single pile might be generated using correlations with soil properties, in situ pressuremeter tests, or other means.

Shown in Figure 2 is the concept of a *p*-multiplier (P_m). A reasonable method of accounting for the effect of pile-soil-pile interaction within a group is to modify the *p-y* curves for an individual pile in a group based on row position using a *p*-multiplier that has been empirically derived from experimental data (*1*).

Back analyses have been performed on the relatively few large-scale experiments for which there are rectangular groups loaded to large deflections and for which there are bending moment data with depth for at least several piles in the group. Presented in Table 1 are the backfigured P_m values for three load tests. P_m values were determined for the experiments using the following procedure.

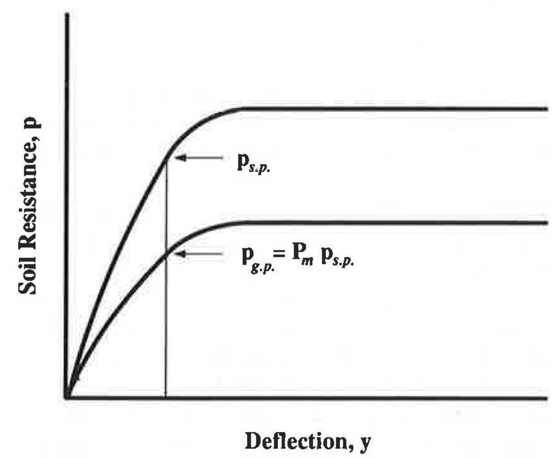

FIGURE 2 Concept of *p*-multiplier (P_m).

TABLE 1 P_m VALUES FROM SELECTED LARGE-SCALE EXPERIMENTS

Reference	Deflection (in.)	Front Row	2nd Row	3rd Row
Brown et al, 1988; Sand,	1.0 to 1.5	0.8	0.4	0.3
Brown et al, 1987; Stiff Clay,	1.2	0.7	0.6	0.5
Stiff Clay,	2.0	0.7	0.5	0.4
Meimon et al, 1986; Soft Silty Clay	0.6	0.9	0.5	-

1. Using either the actual *p-y* curves from a single pile experiment at the site or analytical *p-y* curves fitted to the single pile experimental results, perform an analysis of a single laterally loaded pile using COM624 (*7*) to confirm that the predictive model is appropriate for the specific research site.

2. Perform a number of analyses using COM624 with *p-y* curves that have been modified using different P_m values; all of the *p-y* curves are adjusted using the same (constant) value of P_m.

3. For piles representative of each row in a group, select the value of P_m that provides the best agreement in terms of pilehead load versus deflection in the ½ to 1 in. deflection range (12.7 to 25.4 mm) and in terms of depth to and magnitude of the maximum moment in the pile. P_m is thus not directly backcalculated but selected based on agreement between analytical and experimental results.

Note that because these back analyses have been performed using *p-y* curves fitted to actual data from a single pile load test on the same site, it may reasonably be concluded that the observed effects are due to group action and not simply errors in the site-specific *p-y* curve formulation. Although P_m need not be constant with depth, the P_m values shown in Table 1 were derived as constants; given the limited experimental data, there appears to be little justification in attempting greater precision by varying P_m.

The P_m values shown were found to provide good agreement with both measured bending moments in the piles as well as pilehead load-deflection relationships. The group effects were larger with increasing deflection (load level), which may account for the relatively larger values of P_m derived for the test piles described by Meimon et al. (*3*); these piles were loaded to static deflections of only about 0.6 in. (15 mm), which were not as large as those of the other test cases.

PROPOSED DESIGN PROCEDURE

The proposed design procedure uses the *p*-multiplier concept described previously, along with a relatively simple way of incorporating this approach into a conventional structural analysis. The procedure is briefly described here.

1. Develop equivalent static loads for barge impact following the Louisiana DOTD guidelines (*7*).
2. Develop *p-y* curves for an isolated individual pile using the best available means; this might be done using available published correlations with strength and other soil properties, using in situ data from a pressuremeter or other device, or using a site-specific load test.
3. Perform analyses using a code such as COM624 (*7*) to develop load-deflection and load-moment data for piles using the *p-y* curves developed in Step 2, modified using the P_m values appropriate for a given row position. In general, and until further research provides more complete design guidance, it is suggested that the soil resistance values for the isolated pile be multiplied by $P_m = 0.8$ for the front row, $P_m = 0.4$ for the second row, and $P_m = 0.3$ for the third and all subsequent rows. These values are considered to be reasonable for pile spacing of about 3 diameters on center and for deflections at the groundline of about 10 to 15 percent of the pile diameter; they may be somewhat conservative for larger spacing or smaller deflections. For most cases in which the piles in the group are embedded into a thick concrete cap, it will be appropriate to assume a top boundary condition in which the piles are fixed against rotation and subjected to a specified shear or deflection. Once a shear load versus deflection relationship is established for piles in each row, it is possible to estimate deflections of the group for a given lateral load. One may generally assume the cap to be rigid, so that the piles all undergo the same deflection.
4. After an initial estimate of deflection and pilehead shear is obtained for the piles of each row, an equivalent length of fixity (L_{eq}) is estimated for the piles in each row using the relationship

$$L_{eq} = [12EI/(P/Y_t)]^{1/3}$$

where

P = Lateral load at top of pile,
Y_t = Deflection at top of pile,
E = Young's modulus of pile material, and
I = Pile bending moment of inertia.

This equivalent length will allow the piles in the group to be modeled in the structural analysis as cantilevered beams, fixed at the base, in order to give an appropriate load-deflection response from the piles for the structural analysis. Note, however, that the equivalent length used is only appropriate for a limited range of deflection because the true load-deflection response of each pile is nonlinear. It would also be possible to model the foundation by replacing the piles with springs of different equivalent stiffness; the equivalent length approach is primarily used for convenience.

5. Perform the structural analysis of the frame with the piles in the group modeled as cantilevered beams. Note that the cantilevered beam model is only to match the lateral load-deflection response at the pilecap, and the computed bending moments at the point of fixity are irrelevant. Likewise, if significant axial loads are associated with the ship impact, the axial load-deformation response of the piles using this model may not be appropriate. A two-dimensional analysis is typically performed, with each cantilevered beam representing an actual row of piles. As a result of the structural analysis, the shear at the top of each pile and pile group deflection (Δ) is computed.
6. Because some of the load is typically transferred through the frame to other parts of the structure, Δ is usually slightly less than the deflection estimated in Step 3. If these deflections are not within a few percent, return to Step 4 for a revised estimated L_{eq} based on a deflection equal to Δ, and iterate to convergence.
7. Once the lateral load and deflection at the top of each pile is established, check the structural adequacy of the piles using the bending moments computed in Step 3 for that load. Because the piles in the front row attract the most shear load, the front row piles usually govern the structural design of the piles.

Because the procedure is specifically for two-dimensional analysis of rectangular groups, impact loads that are skewed are resolved into two orthogonal load components that are analyzed separately. The row positions of each pile will be different for the two orthogonal load cases, and the L_{eq} values used for each row will likely differ. Stresses and displacements are summed using superposition. Although this approach to a skewed loading condition is a simplification of the actual three-dimensional problem, the effects of shadowing in a skewed loading are likely to be less severe, and thus, the approach is a rational simplification. No experimental data exist for such a condition to provide guidance for design, so there does not seem to be sufficient justification for a more complicated procedure.

EXAMPLE

To illustrate the proposed procedure, a simple example is provided. A 4 × 5 pile group of prestressed concrete piles is to be analyzed, as shown in Figure 3. An equivalent static lateral load of 1,600 kips (7,120 kN) is used to represent the impact load. The computer code COM624 is used to analyze the piles in each row using the criteria for sand proposed by Reese et al. (*8*), modified by f_m values of 0.8, 0.4, 0.3, and 0.3 for each row, front to back, respectively. COM624 generates the *p-y* curves internally, and the addition of the P_m modification to the code is relatively simple. The load-deflection and moment-deflection curves resulting from these analyses are shown in Figure 4. Note also that the average

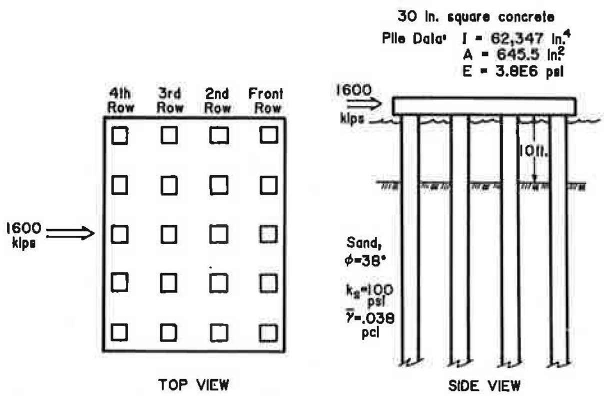

FIGURE 3 Design example (1 in. = 25.4 mm, 1 kip = 4.45 kN, 1 psi = 0.145 kPa).

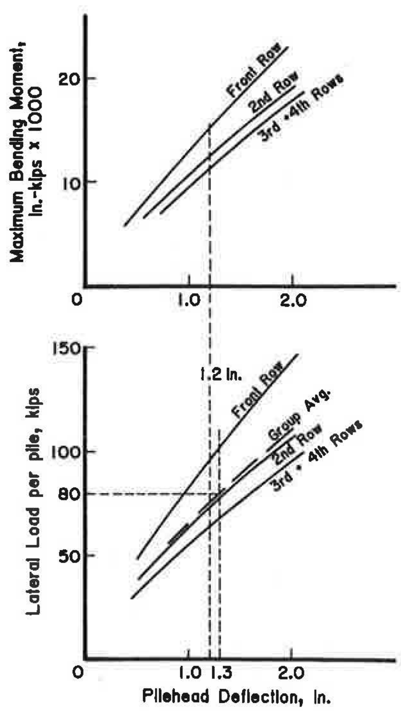

FIGURE 4 Plots of load versus deflection and moment versus deflection for design example (1 in. = 25.4 mm, 1 kip = 4.45 kN).

TABLE 2 COMPUTED L_{eq} VALUES AT 1.3 in. (33.0 mm) DEFLECTION

Position	Shear (kips)	L_{eq} (feet)
Front Row	103.	27.5
Second Row	79.	30.0
Third Row	69.	31.4
Fourth Row	69.	31.4

TABLE 3 COMPUTED L_{eq} VALUES AT 1.2 in. (30.5 mm) DEFLECTION

Position	Shear (kips)	L_{eq} (feet)
Front Row	98.	27.2
Second Row	74.	29.7
Third Row	64.	31.1
Fourth Row	64.	31.1

pile load-deflection curve is plotted by averaging the load at a given deflection from the piles in all rows. Using this average pile curve, at an average load of 80 kips (350 kN) per pile, a deflection of 1.3 in. (33.0 mm) is computed.

For the piles in each row at 1.3 in. (33.0 mm) of deflection, the lateral shear load at the pilehead can be determined from Figure 4, and the equivalent length for the piles in each row computed as presented in Table 2. A structural code such as STRUDL is used to analyze the pier-superstructure-pile interaction with the piles modeled as cantilevered beams fixed at the bottom of length L_{eq}, and a pilecap deflection of 1.2 in. (30.5 mm) is computed. Another iteration is performed using the equivalent lengths computed at a 1.2-in. (30.5-mm) deflection (shown in Table 3), and convergence is achieved. Note that the resulting shear force on the group is now 1,500 kips (6,675 kN); 100 kips (445 kN) has been transmitted to other parts of the structure. Maximum bending moments in the piles are determined from the moment-deflection diagram in Figure 4. Note that the results of a COM624 analysis for the entire length of pile can be produced using the final pilehead shears or deflections, but normally the pile reinforcement is not varied to adjust for changes in computed bending moments.

For conceptual design and initial sizing of the group, engineers at Florida DOT have developed design charts for some typical soil conditions by plotting curves of equivalent length of fixity below the mudline versus lateral load for selected piles. Limiting values of lateral load are indicated for selected piles. In this manner, a group can be sized relatively quickly for further analysis using the procedure identified here.

Note also that for concrete piles or shafts, the stiffness in bending (*EI*) will vary with the level of bending stress. If *EI* is to be a constant, a value of *EI* that is representative of the pile or shaft behavior at the computed bending stresses should be chosen.

CONCLUSION

Data from large-scale experiments on groups of piles subjected to lateral loading have been analyzed for insight into design of pile groups for ship impact loading. A rational design procedure is proposed that includes the most important aspect of pile-soil-pile interaction—"shadowing," in which piles in trailing rows are subject to reduced soil resistance. The design procedure proposed incorporates existing techniques for design of piles for lateral loading as well as existing techniques for structural analysis of bridges. The procedure is limited to analysis of rectangular pile group arrangements subjected to relatively large loads and deflections. Experiments indicate that group effects are less significant at lesser load levels. The procedure is based on a center-to-center pile spacing of about three pile diameters; experimental data are insufficient to extrapolate to other arrangements or spacings. The limited amount of experimental data available to guide designers in this area emphasizes the need for additional research.

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