LRFD Code for Ontario Bridge Substructures

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A design procedure for bridge substructure foundations and retaining walls, Load and Resistance Factor Design (LRFD), is documented in the Ontario Highway Bridge Design Code. Details of the procedure are given. Structural and geotechnical design procedures are similar and compatible. LRFD procedures help to clarify the calculation procedures used when soil and structure meet and interact. Few new technical problems result for the geotechnical engineer using LRFD; however, communication between geotechnical and structural engineers is essential to ensure that the serviceability limit and the ultimate limit are identified for structures designed using LRFD. The design process is described and evaluated. Issues relating to earth pressures, shallow and deep foundations, and code writing are discussed.

Load and Resistance Factor Design (LRFD) procedures for bridge superstructures and substructures make up part of the first edition of the Ontario Highway Bridge Design Code (OHBDC), as published in 1979 (*1*). The LRFD code came about because changes in legal truck loads during the 1970s created a need to verify that designs for structures considered these changes in design loads and superstructure analysis.

The 1979 Code addressed design of substructures and retaining walls, interaction between structure and soil, and communication and coordination between geotechnical and structural engineers. Initially, geotechnical engineers did not like or accept the new procedures because of a new terminology, an incomplete understanding of LRFD, and an attempt to codify geotechnical design procedures. Some members of the geotechnical profession believed, incorrectly, that LRFD and associated factors were based solely on statistical concepts. Their negative reaction to a new design procedure was unexpected. LRFD is really a rearrangement of factor of safety design (FSD) provisions, and it has been applied successfully in Denmark for many years (*2*).

DESIGN PROCESS

Structural design and geotechnical design, or other design connecting a structure and soil or rock, have a common objective, namely to provide an acceptable level of reliability, including a minimization of loss of function. Uncertainty exists in the design process because load (force) effects vary. In addition, there is uncertainty related to construction, material characteristics, and resistance predictions. Finally, imperfections in analysis or lack of knowledge about the structure being designed come into play.

Structural design is described as an exact science, and geotechnical design is thought to be experience based. However, in practice there is little to distinguish geotechnical design or evaluation from structural design or evaluation. Both design processes involve the recognition of uncertainty, require sound judgment, and apply historical experience.

The majority of design procedures used for foundations or structures address one or more limit states. These limit states may be defined in a design specification or as part of an office procedure. The two important limit states are

- Ultimate limit state (ULS): when a failure mechanism forms in the soil or rock, or in a structure, and
- Serviceability limit state (SLS): when loss of serviceability occurs in a structure because of deformation of the soil or rock.

A structure's or soil's reaching an ultimate limit state implies a major loss of lives or capital and damage that is not easily reparable. The collapse of a bridge, for example, may result in considerable economic loss or necessitate complete replacement.

The probability of an ULS condition occurring is about 10^{-3} to 10^{-5} (*3*). SLS occurs with a larger probability than ULS, and the damage or loss of service at SLS is reparable. For example, in a bridge foundation there may be one chance in 20 or 30 that settlement diminishes ride quality. The loss of ride quality either will be accepted, or surface repairs can be made at little or no cost.

SAFETY CONSIDERATIONS

A bridge superstructure or a pile foundation with resistance, R, subject to specified load effects, U, is considered. In design, different R values appropriate to the serviceability limit state, R_s, and the ultimate limit state, R_u, are used with a series of load effects, U, based on various combinations of specified vertical and horizontal loads. Reliability concepts are illustrated for both LRFD and Factor of Safety Design (FSD).

Load and Resistance Factor Design

Using LRFD, specified loads are modified by multiplying the specified load by a load factor that is appropriate to the level of uncertainty associated with a given load and limit state. Values of load factor selected for OHBDC3 (*4*) are given in Table 1. Barker et al. have documented load factor values proposed for U.S. use (*5*). Several combinations of load are usually employed to determine the maximum destabilizing effect of load and thus maximize the probable resistance demands of both the soil and the structure. The design equations, for serviceability (SLS) and strength (ULS), are

$$\text{SLS: } R_s > U \quad (1)$$

$$\text{ULS: } I(\phi R_u) > \alpha u \quad (2)$$

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where

ϕ = resistance factor of either the soil or a structural component;
α = average load factor associated with combinations of specified loads U;
R_s = resistance based on a prescribed deformation, typically 25 mm or 50 mm;
R_u = predicted ultimate resistance of soil or rock due to vertical load, including the effects of ground inclination, embedment, layering, and the like;
R_u = ultimate resistance of a structural component; and
I = factor applied to the factored geotechnical resistance, ϕR_u, for load inclination, always less than 1.0.

Equations 1 and 2 apply to both structural design and geotechnical design. Different combinations of load are frequently used as part of the two design processes for the same limit state. Equation 1 is nearly identical to that used in factor of safety design, except that R is equal to R_s, a resistance based on a prescribed deformation. Design for ultimate strength is covered by Equation 2. For geotechnical design at ULS, the value of ultimate resistance is a function of the angle of inclination of the particular load combination forming U. The uncertainties covered by the design equations, Equations 2 and 3, include the following:

1. Selection of specified loads, both structural and geotechnical;
2. Method of analysis, both structural and geotechnical;
3. Choice of geotechnical parameters and resistance for a given stratigraphy; and
4. Variability in material properties and member structural resistances.

With LRFD, the geotechnical engineer normally will supply the values of R_s and R_u of the soil or rock for the design. These values must be consistent and apply to the site. Consider a medium sand supporting a footing 4.0 m in width, where R_s may be specified as 220 kPa for a vertical settlement of 25 mm and R_u as 2,000 kPa for vertical loads. The value of R_u of 2,000 kPa for a 4-m-wide footing may appear to be excessive. However, it is representative of a dry granular soil in which the angle of internal friction is about 33 to 35 degrees.

TABLE 1 Load and Resistance Factors from the Ontario Highway Bridge Design Code (Third Edition) (*4*)

Load	Load Factor	Resistance	Resistance Factor
Dead Load	1.10 to 1.50	Bearing	0.5
Live Load	0.80 to 1.25	Shear, on granular surface	0.8
Earth fill	0.80 to 1.25	Horizontal passive	0.5
Earth pressure	0.80 to 1.24	Static test, pile	0.6
Earth fill plus pressure	1.00 to 1.25	Static analysis, pile	0.4

Factor of Safety Design

A single equation applies for FSD:

$$R > U \tag{3}$$

where R is the lesser of R_s or $(I \cdot R_u)/F$, and F is the factor of safety.

For narrow footings founded on a granular soil, strength expressed by the function, $(I \cdot R_u/F)$, will control the choice of resistance, R, while serviceability, R_s, will generally control the design of wide footings. In Equation 3, all uncertainty is assigned to one function, namely, the factor of safety, F, unlike the separation expressed by Equations 1 and 2. There is little room for improvement in design when a single value covers all uncertainties associated with both load and resistance.

Factors of safety quoted for geotechnical work vary according to the function of the system. For example, factors range from as little as 1.3 for earthworks, for which the problem is almost completely geotechnical, to 3.0 for foundations. Both geotechnical and structural considerations apply in the case of foundation design, and loss of life may be a consideration in design (*6*). The calculated factor of safety for 14 embankments, all of which failed, varied from 1.0 to 1.8. This suggests that uncertainty in both analysis and the choice of the best value for a geotechnical parameter exists (*7*).

RELIABILITY CONSIDERATIONS

OHBDC includes specified permanent loads based on as-built observations of Ontario bridges as well as specified live loads that are mean maximum loads based on existing truck traffic projected over a 50-year design life (*1,4,8*). The observed loads and associated statistical distributions, uncertainty of analysis methods, professional factors, and growth are used to calculate reliability indices and load factors for design.

Various methods can all be used to predict the ultimate resistance of soil under vertical load, ϕR_u:

- Empirical values,
- Assessed values,
- Geotechnical equations,
- Partial coefficients of soil-strength factors, and
- Reliability-based resistance.

Experience is the contributing feature in the selection of any resistance value based on empirical or assessed values. Geotechnical parameters such as unit weight, cohesion, and angle of internal friction are needed for the calculation of the resistance value based on geotechnical equations or reliability considerations. Many geotechnical design resistance values are based on empirical evidence suitably adjusted for historical experience. For example, allowable values for FSD are either limiting deformation or ultimate (capacity) values (factored down for safety with F equal to about 3).

A choice must be made between a global resistance, where the contribution of several parameters are lumped together, or a resistance based on individually factored geotechnical parameters. Both the Danish standard, DS 415 (*2*), and OHBDC2 (*8*) provide procedures for calculating lateral earth pressures using factored parameters. OHBDC3 publishes only resistance values based on global considerations and a system performance factor, which is less than unity (*4*). This factor includes the effects of uncertainty and can be back calculated from existing designs. Detailed knowledge of the contribution made by the friction of the cohesive components of resistance need not be known if global factors are used.

Empirical Values

ULS and SLS resistance values can be developed from empirical relationships between resistance and some indirect measure of the geotechnical parameters, such as the standard penetration test (SPT), cone penetration test (CPT), or pressuremeter data. OHBDC3 (*4*) encourages the use of empirical methods, as they are well proven. A resistance factor of 0.5 is recommended for empirical bearing resistance values in OHBDC3 (*4*) (Table 1), although the procedures used to develop the value may be method driven. The geotechnical engineer selects the empirical method according to particular site conditions; the Code does not recommend a method.

Empirical values are not identical to the presumed values published in many design handbooks. Presumed values appear to apply only to SLS and FSD design, and they cannot be modified easily to apply to an ULS situation.

Assessed Values

Data from completed investigations for one site may be of value when investigating another site, if the sites have similar stratigraphy. Thus, it may be possible to use the ultimate-resistance values from a completed investigation and an appropriate resistance factor for the new site—taken as 0.5 or 0.6 for bearing or axial resistance of piles, respectively (*4*).

Geotechnical Equations

For each design situation, there will be a suite of applicable design equations. A geotechnical engineer usually will favor one or two for resistance prediction of shallow or deep foundation design that are based on historical experience. Each equation provides a different value for the mean of the ratio of observed to calculated resistance, E, based on the geotechnical parameters chosen for the test site. Normally, the value of E should be unity in the absence of other data. The following is used for design purposes:

$$\text{Calculated factored resistance} = \phi \cdot E \cdot R_u \tag{4}$$

where R_u is a calculated resistance that is a function of the geotechnical parameters, drainage conditions, and geometry of the footing and piles.

Partial Coefficients or Soil-Strength Factors

Partial coefficients for geotechnical design appear to have been developed by Hansen (*9*). These coefficients are not based on a reliability assessment of typical soil or rock but were based on a rearrangement of FSD values. This rearrangement permits a two-part separation: namely, uncertainty due to geotechnical resistance (partial coefficients), and uncertainty due to load effects (load factors in LRFD). DS 415 provides values of partial coefficients for various safety classes (2). In OHBDC2, partial coefficients are specified and are referred to as soil-strength factors.

The factored soil strength parameters, c_f and $\tan \phi_f$, (*2*) are

$$\text{Cohesion: } c_f = cF_c \tag{5}$$

$$\text{Internal friction: } \tan \phi_f = F_\phi \tan \phi' \tag{6}$$

where F_c and F_ϕ are soil-strength factors for cohesion, c, and friction, $\tan \phi$, respectively. The specified values of F_c are 0.65 for stability and earth pressure and 0.50 for footings and piles. F_ϕ has a single value of 0.80 that applies to earth pressure and resistance calculations. Factored soil strength parameters should be used directly in Equation 4, using a resistance factor ϕ of 1.0. If the site investigation provides either SPT or CPT data, these data can be used to develop geotechnical parameters. Many Ontario engineers using OHBDC2 found that this was a very indirect treatment of geotechnical parameter data (*8*). They preferred to obtain empirical values of SLS and ULS bearing resistance directly from SPT, CPT, and pressuremeter test values, without "guessing" geotechnical parameters.

Reliability Based Resistance

For a major structure that involves a high degree of risk, a comprehensive site investigation using continuous monitoring, for example, may be carried out. The results would yield the geotechnical parameters for soils at various locations and strata in terms of a mean and standard deviation. Such a site investigation would reduce uncertainty compred with a more limited, traditional one. Calculation details are available for factored resistance from statistical data (*3*).

Discussion of Selected Resistance

The selection of factored resistance will be a function of the quality of a site investigation and the complexity of the soil conditions at the site. More refined methods may be inappropriate for a site where the subsurface conditions are extremely variable and uncertain. OHBDC2 recommends that soil strength factors be used for ULS values for shallow and deep foundations (*4*). When test data are available for piles, a global resistance can be applied for OHBDC2 assessments. Many users found it difficult to apply soil strength factors to friction piles, as the mathematical results tended to contradict experience. In the latest version of the Ontario Code (*4*), soil strength factors were replaced by performance factors.

SHALLOW FOUNDATIONS

Ultimate bearing resistance may be based on SPT data, CPT data, or on bearing resistance (capacity) theories, if the geotechnical parameters are known. The ratio of observed to calculated ultimate bearing resistance for shallow foundations is 1.20, according to Terzaghi's theory (*10*), or 0.86 according to Meyerhof's recommendations (*11*), based on data compiled by Bowles (*12*). Even with accepted resistance-calculation procedures, questions still arise as to the adequacy and conservatism of the calculation. In addition, the final method of selecting the "best" geotechnical parameters to be used in a calculation is not always clear. The author understands that a conservatively chosen, representative mean value is often used.

In an LRFD format, a range of ultimate bearing resistance values should be provided for a shallow foundation, for both footing width and embedment. Figure 1 shows calculated bearing resistance for various widths for an ideal footing founded on the surface. The soil has an average N value of about 20 to 25, and the water table is low. Factored ultimate resistance values increase with footing width. The SLS resistance, based on a deflection of 25 mm, is approximately constant with increasing footing width. The points where a transition occurs from ULS to SLS are indicated by a and b (Figure 1). Ultimate resistances shown in Figure 1 are, in a clockwise direction, a calculated ultimate resistance, R_u, a factored resistance proposed in OHBDC3, ϕR_u, an FSD resistance, $R_u/3.0$, and a factored resistance, $I\phi R_u$, for an inclination factor for an angle equal to 21.4 degrees. In Figure 1, the factored resistance for a footing width of 4 m is nearly five times the SLS resistance.

Foundation reports made available to the author frequently quote a factored resistance that is only one-and-one-half times the SLS value for granular material. The value of 1.5 is assumed to be a back calculation from an SLS value, an F value of 3.0, and applying a resistance factor of 0.5. The FSD values shown in Figure 1 are for an SLS condition and do not include the inclination of load.

With the inclusion of the inclination factor in ULS design (Equation 2), marked changes in bearing resistance resulted for designs based on OHBDC2 (*8*). When calculations for an abutment wall footing without embedment that has a ratio of vertical to horizontal force of 0.15 are made, the calculated (vertical) ultimate resistance for granular material is reduced to approximately 60 percent of the vertical resistance. For a retaining wall that has a smaller mass than an abutment, this ratio of vertical to horizontal force increases to

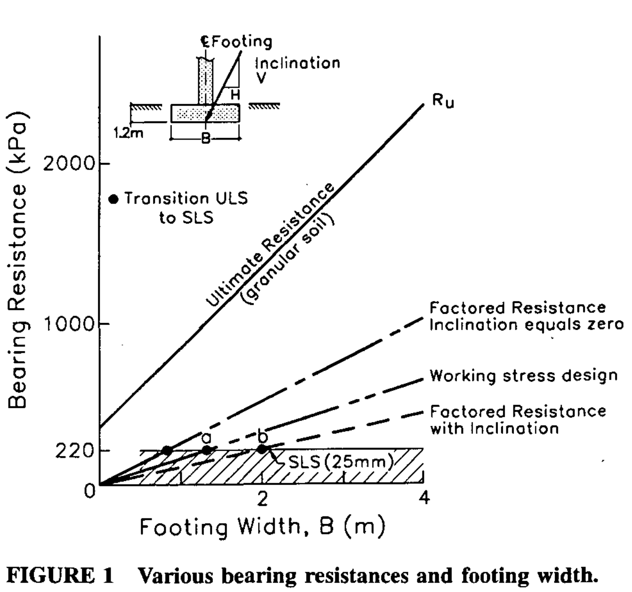

FIGURE 1 Various bearing resistances and footing width.

about 0.40 or 21.4 degrees. The ultimate resistance is only 20 percent of that for vertical load. The reduction factors quoted are for footings founded on the surface. An increase in bearing resistance occurs with footing embedment. An additional resistance of about 700 kPa may be added to the values shown in Figure 1 if an embedment of 1.2 m is present. The embedment of 1.2 m is the design frost cover depth in much of Southern Ontario. This additional resistance due to embedment is well known and should be considered by the geotechnical engineer. The geotechnical report should be flexible enough to permit the structural engineer to make a choice between changing the footing width or increasing the embedment during design, and the choice should be made with the geotechnical engineer's knowledge.

Few tests of footings with inclined load are available. The data provided by Muhs and Weiss (*13*) for relatively large, 1-m by 3-m footings were compared with the various design proposals of Meyerhof (*11*), Vesić (*14*), and Hansen (*15*). All three theories propose conservative estimates of reduction factors for granular materials with an angle of internal friction equal to about 38 degrees. The ratio of observed to calculated reduction value was 1.4 with a standard deviation of about 0.05. The reduction factor equations of Meyerhof (*11*), assuming 1.2 m embedment, are used in OHBDC3 (*4*). If bearing-resistance values are calculated directly from geotechnical parameters and a resistance equation, reduction factor expressions should be used appropriate to the specific resistance equation that is used (*4*).

EARTH PRESSURE

The use of an equivalent fluid pressure representation for earth pressure of a free-draining, engineered backfill applying the method of Coulomb or Rankine is common in cantilever walls and abutment design (*12,16*). An active pressure condition, Ka, is assumed when the shear resistance of the retained material is mobilized at assumed lateral displacements of 0.001 of the wall height, a base rotation of 0.002, or a combination of these. For a retained soil with an angle of internal friction of 30 degrees, the horizontal pressure coefficient K_a is taken as 0.33.

When both the stem and base of the wall do not yield during the installation or compaction of the retained soil, lateral pressures in excess of at-rest pressures (K_o = 0.5, ϕ = 30 degrees) may develop [Figures 2(*a*) and 2(*b*)]. Lateral pressures from compaction will develop on the upper part of a stiff wall (*17*). The stems of most abutment walls and retaining walls are more flexible than gravity walls or culvert walls. These retaining walls translate or rotate during the installation of each layer of compacted soil. Horizontal movements will reduce locked-in compaction stresses and lead to the lateral pressure distribution given in Figure 2(*c*). The additional compaction pressures are not large for light hand-compaction equipment, and they can be calculated (*2,17*). Force effects due to the pressures from light compaction [Figure 2(*c*)] can be approximated using an equivalent fluid pressure for the total pressure due to backfill, K_b. This pressure has a value that is midway between active and at-rest pressure for a typical case.

Ministry of Transportation Ontario data (M. Devata, unpublished data, Ministry of Transportation, Ontario) indicate that the angle of internal friction, ϕ, is between 35 and 46 degrees for rock backfill and between 32 and 42 degrees for a granular backfill suitable for free-draining fill. Many design engineers use a ϕ value of 30 degrees for calculation purposes. This is conservative; lateral earth forces are overestimated.

A number of pressure distributions (Figures 2 and 3) may exist following installation of the backfill. Figure 3(*a*) shows surcharge and active pressures acting on a wall. Such a pressure distribution may exist following movement of the base, even though the destabilizing effects of the earth forces are resisted by the soil beneath the footing base. This soil is assumed to mobilize its factored resistance and to deform sufficiently to cause an active pressure condition, K_a, to develop in the retained soil. The situation when the soil is beneath the footing has not reached limiting equilibrium (a factored resistance) is illustrated in Figure 2(*c*) and Figure 3(*b*). The earth pressures acting on the wall are not an active pressure, as movement of the base is small. A backfill pressure, K_b, which includes compaction pressures and associated surcharge, is present. This backfill condition occurs when the bearing is competent and non-yielding during compaction of the fill.

The wall shown in Figure 3 should be designed to resist the forces from both pressure conditions [Figure 3(*a*) and Figure 3(*b*)], that is, both K_a and K_b plus any surcharge. Two separate designs are necessary, one in which the base width is selected (active conditions control) and a second in which the structural size of the wall is calculated (backfill pressure conditions control). The design philosophy is to identify the worst case for the design of the stem, toe, and heel, and the worse case for the footing width.

The provisions of DS 415 include a load factor of 1.0 for all vertical and horizontal earth forces (*2*). The procedure was not followed in the Ontario design documents (*1*,*8*) wherein a load factor of 1.25 was applied to earth pressures that already included an allowance for uncertainty through the use of soil-strength factors. Double counting of the safety provisions in the first two editions, coupled with reduction factors for inclined load, resulted in some footing widths being 50 percent larger than might be obtained using FSD. These proportions were questioned by design engineers. OHBDC3 attempts to rectify the double counting by using one load factor to handle uncertainty in the calculation of active or backfill pressure effects based on unfactored values (*4*). The load factor chosen is 1.25. The uncertainty associated with the horizontal forces and moments due to lateral earth pressure for geotechnical design (Figures 2 and 3) can be managed using either soil strength factors or load factors but not both.

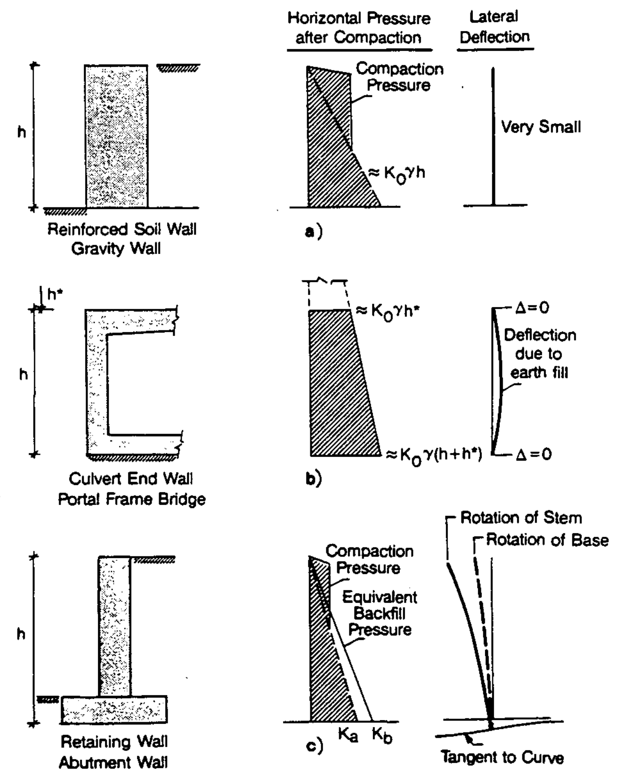

FIGURE 2 Various earth pressure conditions

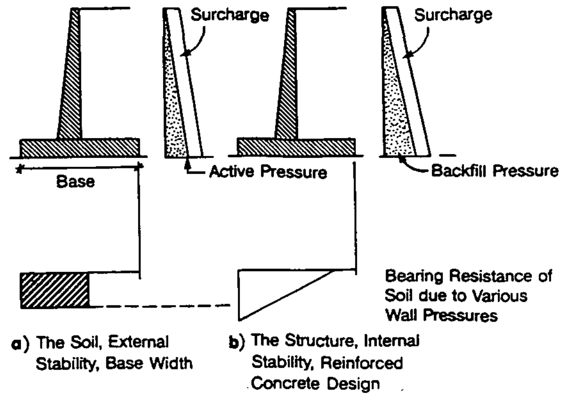

FIGURE 3 Earth pressures and bearing resistances.

DEEP FOUNDATIONS

The design of deep foundations requires a knowledge of the axial and lateral resistances of a pile or group of piles. A calculation procedure whereby the forces acting on a pile due to external actions can be calculated is also required. The geotechnical engineer normally will supply values of ultimate resistance for axial load and may provide lateral resistance values at the ultimate state. The structural engineer will determine the number and the arrangement of the piles, based on the calculation of forces.

Figure 4 shows typical load-deflection data for vertical and horizontal load tests completed in Ontario. The steel piles were driven into fine sand (top 3 m) and then silty clay. ULS and SLS values for vertical load are easily identified [Figure 4(*a*)]. The

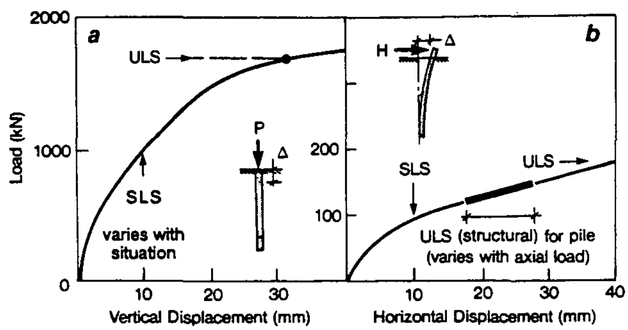

FIGURE 4 Load and deflection test data for both vertical and horizontal loading.

ULS value for the pile is associated with a limiting vertical deflection. The SLS value may be based on a limiting stress or a limiting deflection for a single pile or a group of piles, shown as 10 mm in Figure 4(*a*). Not shown in Figure 4 is an SLS value based on down-drag effects and structural resistance of the pile as well as soil properties.

Figure 4(*b*) illustrates three main design features for horizontal effects. The first is an assumed SLS value based on a lateral movement of the pile of 10 mm (arbitrarily chosen). The other two are ULS resistance values, one based on the soil's passive resistance and the second controlled by the structural resistance of the pile, including lateral load and axial stresses. From simulations made using the procedures of Reese (*18*), the ULS resistance of a pile subjected to horizontal load was found to be controlled by structural rather than geotechnical considerations, except for short piles.

A number of expressions for the axial prediction of pile resistance exist. Many are empirical relationships. Briaud and Tucker (*19*) developed ratios of observed to calculated values for 98 pile tests using 13 methods of calculation and including piles driven in sand or clay as well as in layered soil. Of the 13 methods, only 3 yielded a ratio of observed to calculated greater than unity. This is not a safety problem if an appropriate value of E and the resistance factor ϕ are used for each analytical method in Equation 4, or if conservative values of the geotechnical parameters are used.

The structural engineer requires both simple and detailed methods for the preliminary and final analysis of pile foundations. Soil-structure interaction solutions are available whereby the final designs can be verified (*18*). There does not appear to be a universally accepted method of analysis for the forces in piles. Calculation methods that permit the analysis of pile footings with the very simple geometry given in Figure 5 and consider the interaction between vertical and horizontal forces and associated resistance are required. OHBDC2 provided a limit equilibrium solution for the analysis of vertical load on a pile group (*4*). The code was silent as to how the analysis for vertical load should include horizontal effects, however. The force in individual piles within a pile group is a function of the applied axial load, moment, and the horizontal load applied to the footing of a pile group. Any method of analysis should consider all three load effects concurrently, especially if deformations of the footings and hence superstructure are of import. Analyses that combine the interaction equation for forces due to eccentric load on a footing with a simple, graphic static solution, and include the interaction of vertical and horizontal load, are available (*20*). Even though compatibility of deformation between the structure, piles, and the soil is not considered, this procedure is perhaps the simplest of any for preliminary design. An example of the method is shown in Figure 5.

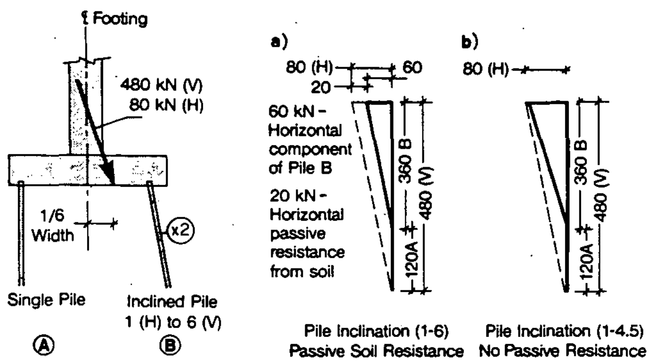

FIGURE 5 Pile force analysis.

In Figure 5 the point of application of the vertical load is chosen to induce equal vertical loads in the single rear pile and each of the two inclined piles. For the loading cases and geometry shown in Figure 5, the two inclined (1 to 6) piles only resist 75 percent of the applied horizontal load of 80 kN. A horizontal passive resistance of 20 kN should be provided by the soil to maintain equilibrium. If all the horizontal resistance is assigned to the inclined piles with none provided by the soil, a design inclination of 1 to 4.5 would be required for the front piles. This design inclination will only be effective for a single-load case. As the ratio of horizontal to vertical load changes, passive resistance would be required from the soil. Conservatively chosen, factored horizontal passive-resistance values are required for design, even if simple manual methods of analysis are used in the absence of the *p-y* compatibility conditions outlined by Reese (*18*).

The example of Figure 5 combined with Huntington's analyses (*20*) suggests a simple method for designing the preliminary proportioning of pile footings that minimizes both rotation and horizontal displacement. The method, which is given elsewhere (*4*), is

1. Select the most common SLS loading condition for the footing; typically this would be the dead load plus any permanent horizontal load.
2. Choose a pile arrangement that results in equal axial load in all piles.
3. Check this pile arrangement to ensure that all other SLS and ULS load combinations are satisfied.
4. If number 3 above is not satisfied, the number of piles (per m run) chosen in step 1 should be increased without changing the centroid of the piles.

Final checks might include 10 to 15 load combinations and would consider the passive horizontal resistance at the pile-soil interface specified by the geotechnical engineer.

DISCUSSION OF RESULTS

LRFD requires the use of little or no new technology for either the structural or the geotechnical engineer. However, LRFD does require cooperation between structural and the geotechnical engineers; a complex project may demand several discussions. Site investigation procedures can remain unchanged. New technology will be used in the future and will reduce uncertainty regarding the identification of the stratigraphy and the soil parameters. Complex structures still demand a high level of investigation. Results from detailed investigations may provide geotechnical data of the quality and quantity necessary for reliability-based predictions of resistance.

A repackaging of the design information developed for FSD design is required for LRFD, as it addresses SLS and ULS as two separate, specific design states (*4,5,21*). The geotechnical engineer should no longer provide a single bearing value for shallow or deep foundations based on the more conservative of either SLS or ULS resistance. Both resistance values are required for structural design. For structures with components that interact with soil, the serviceability limit may control design aspects involving the soil, whereas the ultimate strength limit may control structural design. Different combinations of load may apply in the proportioning of a footing width (geotechnical) or in selecting a footing depth and the reinforcing steel for that footing. The concept is not new; the process permits design for extreme values and combinations of load. Although some additional computational effort may be required, it is not a problem if design spreadsheets are used.

LRFD procedures demand full understanding of the interaction of soils and structures, and the design process for using these components. The LRFD method leads to complete designs and permits the use of new data in both design and evaluation.

CONCLUSION

LRFD is an appropriate procedure for resolving design problems where interaction between soils and structures is present. Designs evolve where either serviceability or ultimate limits control the final design, thus providing a linkage between FSD and ultimate-strength design. No new technology is required for LRFD. However, a reassessment of current design processes is required.

ACKNOWLEDGMENT

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