Analysis and Design of Piles for Dynamic Loading

Analyse et conception de fondations par pieux en chargement dynamique

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ABSTRACT: With the acceptance of Eurocode 8 in Hungary a new level of seismic design is now necessary. This paper outlines some of the past history and present implementation of foundation design for seismic loading as practiced in Hungary. It shortly describes the possibilities of modeling foundations (fix support, linear elastic support, non-linear elastic support) during the design of the superstructure, and introduces the Hungarian practice. The influence of the different support's methods for the bearing forces/stresses of a superstructure is analyzed on a typical reinforced concrete office building using SAP2000 finite element software. The results of the calculation are compared through the moment of a column, the deflection and the support reaction.

RÉSUMÉ : Il était nécessaire de modifier les principes de dimensionnement des fondations par pieux, vis à vis des charges sismiques, depuis l'entrée en vigueur en Hongrie de l'Eurocode 8. L'étude donne un apercue des méthodes précédentes et actuelles du dimensionnement dynamique des fondations. Elle montre aussi les possibilités de modelisations des fondations profondes pratiqués en Hongrie (rigide, élastique linéaire, élastique non-linéaire). La comparaison des differentes techniques de fondation est réalisé par elements finis sur une structure en béton armé et à l'aide du logiciel SAP2000. La comparaison des différents résultats est montré en terme de moment, de déplacement et de force de réaction d'appuis.

KEYWORDS: pile foundation, seismic design, Eurocode-8

1 INTRODUCTION

Acceptance of Eurocode 8 in Hungary has lead to a great many changes in how engineers think about designing for seismic loading. Since Hungary experiences only moderate seismic events over several hundred years, it is difficult to implement a practical yet thorough design procedure for seismic (and other dynamic) loadings. Much of the research and development in seismic design is driven by post-earthquake evaluation and large-scale testing. For regions with lower seismicity and modest research budgets, this approach becomes problematic. A more urgent problem is that of practical implementation: designers must do something. In order to develop a reasonable and rigorous approach to the problem, the Structures and Geotechnics Department at Széchenyi István University has pursued methods to link the latest approaches in earthquake engineering to standard structural and geotechnical design practice. To that end, part of the difficulty is pile design for seismic loading. This paper describes some of the ongoing efforts to use sophisticated analysis and testing in "code-based" design practice.

2 BRIEF REVIEW OF RESEARCH

Seismic behavior of piles and their contribution to structural response has been recognized for over 50 years. However, due to the difficulty in modeling this behaviour, only very simple models could be implemented. One early approach by Penzien (1970) illustrates that the complexity of the problem was acknowledged by even the best analysts. The early methods used a subgrade reaction approach where the soil is replaced by (usually linear) springs. Dashpots may be added as well to simulate both material damping of the soil as well as radiation of energy from the foundation. Additional refinements in the approach to modelling piles included visco-elastic media and analytical methods with relaxed boundary conditions (Novak and ElSharnouby 1984, Dai and Roesset 2004). Concurrent with the development of the relaxed continuum methods, finite element approaches became more practical and required less expensive computing resources. This rather complex evolution of hardware, programming and theory resulted in what are now the de-facto analysis methods for assessing seismic behavior.

In addition to these classes of analysis, four levels of progressively "complete" SSPSI analyses can be described (Wolf, 1985). The basic level consists of a single pile kinematic seismic response analysis, normally incorporating nonlinear response and performed as a pile integrity evaluation. A pseudo-static method for pile integrity evaluation consists of transforming the horizontal profile of soil displacement (derived from a free-field site response analysis) to a curvature profile, and comparing peak values to allowable pile curvatures. This method assumes piles follow the soil perfectly, and that no inertial interaction takes place. Alternatively, a displacement time history may be applied to nodal points along the pile in a dynamic pile integrity analysis. The kinematic approaches consider the difference in stiffness between piles and surrounding soil as seismic waves travel through both. The mismatch leads to stresses generated in the pile system that become more pronounced as the soil becomes softer (the mismatch becomes greater). This behavior is indeed difficult to measure, or even qualitatively assess in the field.

In a second level of analysis, pile head stiffness or impedance functions may be condensed from linear or nonlinear soil-pile analyses and assembled into a pile group stiffness matrix for use in a global response analysis. Secant stiffness values at design-level deformations are normally ascertained from nonlinear soil-pile response analyses. Third, both inertial and kinematic interaction may be evaluated from a substructuring type analysis to determine pile head impedance and foundation level input motions.

Finally, a fully-coupled SSPSI analysis may be carried out to define the complete system response.

Shake table and centrifuge models have offered a great deal of insight into the kinematic and inertial interaction between pile groups and soil, however, there remains a great deal of research yet to be done.

2.1 *SPSI*

Interaction of piles with structural components (inertial interaction) is perhaps better understood. However, simplifying assumptions to gain computational efficiencies and limit the complexity of the design process are often adopted. This is a common practice when performing structural design calculations when foundation alternatives have not been finalized. Most importantly, the design engineer wants to know what impact, if any, the foundation design will have on the seismic response of the structure.

EC-8.2 and 8.5 recommend values for applying seismic loading and estimating pile stiffness. The loadings and soil reactions are then applied to pile elements and included in the overall structural analysis/design process. These stiffness formulae are similar to those discussed by Gazetas and Dobry (1984) for low frequencies and typical ranges of pile-soil moduli.

2.2 *EC-8 Stiffness Formulae*

Eurocode 8 suggests formulae for estimating pile stiffness in seismic response analysis. One should note the effect of soil profile on behavior, especially for lateral loading. Load displacement curves based on EC-8 formulae are shown in Figure 1 for sand (E_s=20 Mpa) and clay (E_s=5 Mpa). As one moves down the legend the responses get stiffer. The lines marked with Cl or Sa are the clay and sand stiffness based on linear distribution, square root distribution, or constant with depth. E_s values were adjusted so that the average E values for a 12-m pile were equal for the respective distribution profile. Values for sand and clay are typical for Hungarian soils.

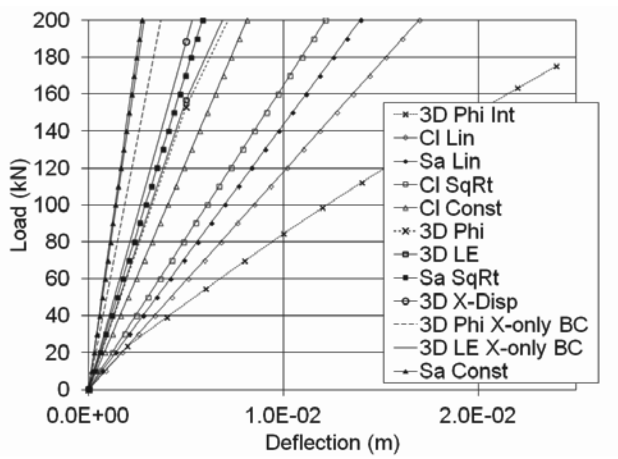


Figure 1. Typical lateral load deflection for Eurocode 8 suggested spring constants and corresponding FEA results for single piles.

These stiffness values are very high compared to what one would consider lateral load-deflection behavior for a single pile in a soil similar to the sand or clay shown. The lines marked 3D represent 3D FEA of the same pile with different conditions. The most flexible condition uses an elasto-plastic sand with a Mohr-Coulomb failure condition and E_s=20 Mpa, φ=30 deg and interface "slip" elements appropriate for the model. Progressively stiffer responses occur when the interface is removed, linear elastic (LE) behavior is used, or boundary restraints are applied. Finally, agreement is reached when the pile head is restricted to only horizontal displacement and linear elastic soil is used. Care should be taken when adapting one set of analysis or field test results for another design approach.

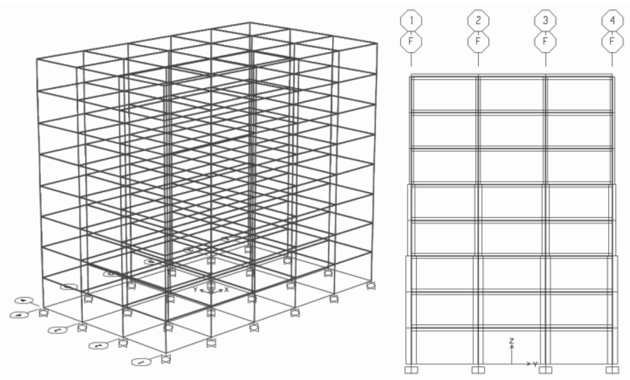
While dynamic conditions may influence the value of stiffness, other authors have pointed out that the first approximation to pile stiffness, especially in lower frequency ranges, is the static displacement profile (Gazetas and Dobry 1984; Blaney et al 1976; Novak and Nogami 1977). Damping will also reduce response, but to a lesser extent and it primarily shifts the pile response out of phase with the driving (earthquake) forces.

3 APPLICATION TO BUILDING DESIGN

The primary focus of research has been to establish more precisely the effects of SPSI on structural design considerations. To that end, an example design is used as a basis for study.

3.1 *Example Structure*

The structure is a 5-bay by 3-bay reinforced concrete frame with a ground floor, seven floors supported by columns, and a roof slab. Floor slabs are separated 3.2m c-c and columns are spaced uniformly 6.0 m c-c. Structural elements were dimensioned according to EC-2 using factors from the Hungarian National Annex. While this is a common design, one unique, and problematic feature of Hungarian designs is the integration of continuous floor slabs with floor beams. This makes it more difficult to model beam-column connections and properly account for stiffness distributions throughout each floor system. Columns are dimensioned 40x65 cm at the base and taper to 40x40 at the top while beams are uniformly 40x90cm and slab thicknesses are 20cm. Perspective and profile views are shown in Figure 2 a, b respectively.



(a) (b)

Figure 2. Perspective and section view of RC frame. Note elastic supports added to base of columns.

3.2 *Seismic Loading*

Hungary is located in a region of moderate seismicity. Recurrence and intensities of earthquakes have been estimated by the National Seismological Observatory, Hungarian Academy of Science. A comprehensive discussion of seismicity as it applies to engineering design can be found in Tóth et al (2006). Based on the observatory's studies, maximum horizontal accelerations one would expect with 10 percent probability of exceedance over 50 years are shown in Figure 3. Design values for seismic loading in EC-8 Annex are presently being completed; however horizontal accelerations of 0.1-0.15g can be expected in some areas.

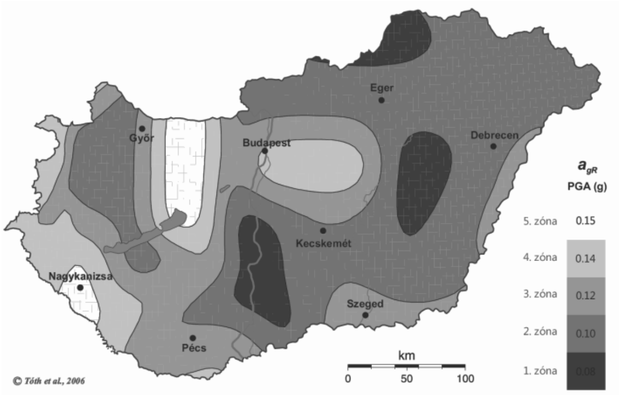


Figure 3. Five Seismic zones in Hungary showing contours of peak ground acceleration, PGA = 0.15,0.14,0.12,0.10,0.08 Tóth et al, (2006).

Acceptance of seismic code requirements by the structural design community in Hungary has been mixed; mainly due to perceptions that seismic requirements present an unnecessary financial burden on the builder. However, much of the design work, when properly executed, result in very little increase in materials and workmanship. Much of the difficulty lies in resorting to overly-conservative design assumptions and pseudo-static approaches that contribute to substantial increases in materials used in standard framing design. Of course, when viewed from this perspective, seismic design is certainly expensive. However, if new designs are carefully applied in a more sophisticated over-all approach, seismic requirements can be met with less difficulty. An added benefit is that the static design is more robust as well.

4 IMPACT OF ELASTIC PILE RESPONSE ON STRUCTURAL DESIGN

A more flexible foundation system will generally offer lower base shear at the cost of larger lateral deflections. What is more difficult to quantify is the re-distribution of member forces due to the greater component flexibility. That is, the relative stiffness of members changes when introducing a flexible foundation system.

4.1 *Response Spectra Method*

For this analysis, one of the methods used was elastic response spectrum. The spectrum approved for use in Hungary is the Type I spectrum (Figure 4). The two spectra are shown with respective soil profile values. Note that the Type II lines are the left-most of the plot lines.

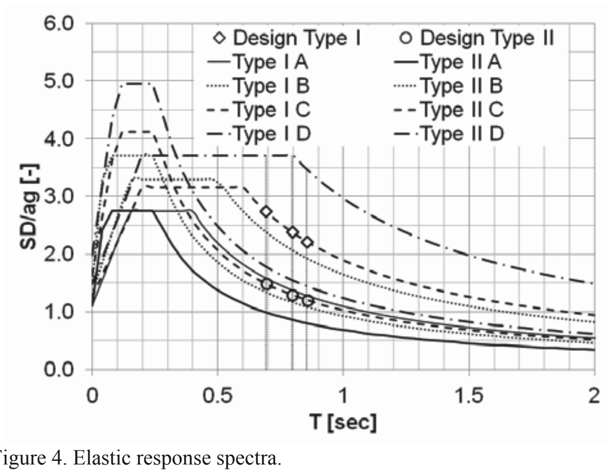


Figure 4. Elastic response spectra.

Building periods were computed from EC-8 formulae (T=0.85 sec) and modal analysis assuming fixed-base (T=0.69 sec) or spring-base (T=0.79 sec). In this range, the design spectral values for a Type I spectra (diamond symbols) are much higher than for Type II (circles). It is ironic that one of the reasons for adopting a Type I curve was the perception that it would yield lower factors and lower cost. However, for a large percentage of buildings in Hungary, the opposite is true, as just shown. Once the spectral factors are known, appropriate lateral loads may be computed for simulating seismic forces.

4.2 *Computing Structural Response*

Load combinations were applied to the structure through SAP2000 analysis program. Loads were based on floor and frame masses and distributed throughout the structure. As suggested in the software documentation (Wilson 2002) lateral load combinations consisted of 100% lateral load parallel to the direction of study and 30% perpendicular to it. This helps to determine the effects of slightly non-symmetrical geometry. Without the perpendicular loading, a feeling of false confidence in building resistance is possible. Three loading cases were examined based on the fundamental periods mentioned earlier: Eurocode-8 (EC-8), SAP fixed base (SapFB), and SAP spring base (SapSB). Since this study has a field component, most attention was focused on the site conditions with Type C soil.

In order to directly compare the difference between rigid and spring base conditions, the structure is first analyzed under a lateral load that was based on the distributed load from EC-8 section 4.3.3.2.3, equation 4.11. For this structure, load increases linearly with height from 8 to 54 kN/m. These values change with changing base shear forces computed from the spectral values discussed earlier. There are a total of six load/base fixity combinations presented here: 3 spectral values; 2 base fixities. While the overall maximum and minimum joint forces changed by moderate amounts, the re-distribution of forces was significant. Table 1 lists joint forces at the base of the structure for 100% force applied on the long side (parallel to the plane of Figure 2b, the y-direction in the model) direction and 30% applied perpendicular. Average percent changes in joint forces, related to the fixed-base condition are also shown.

Table 1. Joint reactions in rigid- and spring-base models

	Spring Base		Fixed Base		
Joint Rxn	Max (kN)	Min (kN)	Max (kN)	Min (kN)	Avg % Chg
Y-direction Reaction	430	278	443	255	20
Z-direction Reaction	3004	556	3165	-8	30

A comparison of forces and moments in selected members shows that the overall seismic effect is reduced, however for particular members, forces and moments may increase. Figure 5 presents interior column bending moments for the loading cases discussed previously. Spring-base conditions reduce bending moments near the base by about 30%. For higher portions of the column, the reduction is less. The same is true to a slightly lesser degree for the exterior columns. Since the EC8 spectral ordinate value is the smallest, lower moments are produced. As the fundamental period decreased for the other cases, spectral ordinate values, and moments, increased.

The other difference in behavior is increased lateral deflection due to the increased flexibility of the spring-base condition. Figure 6 shows lateral displacement of the same column and for the same loading conditions. Note that the foundation flexibility doubles the lateral displacement for almost all soil profiles. The slightly wavy nature of the deflection profiles is due to the abrupt cross-section changes in the column with increasing height.

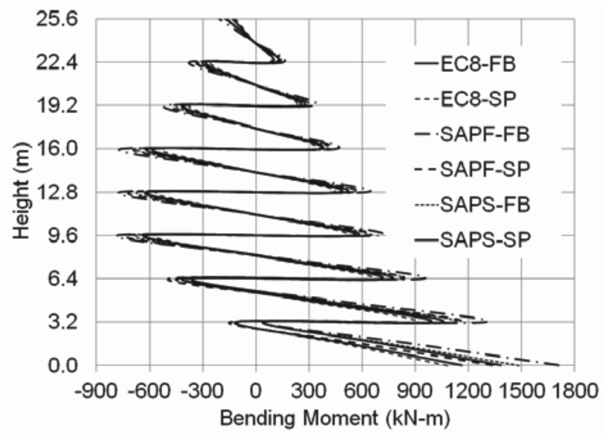


Figure 5. Column bending moments for rigid- and spring-base conditions.

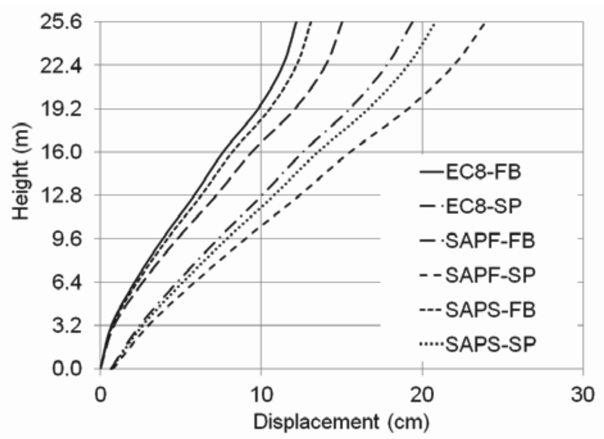


Figure 6. Lateral displacement vs. height for an interior column.

5 CONCLUSIONS AND FURTHER STUDY

The engineering community in Hungary has started to put the seismic provisions of Eurocode-8 into practice. Some parts of the provisions have been easy to deliver, while others require more experience and constructive feedback on their best application. The fact that Hungary has moderate seismicity makes delivery of the code a challenge: seismicity is strong enough to require substantial changes to design practice, but is not obvious to the public and professionals since no major earthquakes have been witnessed in recent history.

Implementing a useful but rigorous methodology for pile design for seismic loading is equally challenging. One of the major efforts in the author's department is to develop a continuously-improving system of analysis, design, testing and assessment of foundations and structures for seismic and other dynamic forces.

The authors are working on integrating sophisticated foundation modeling into the structural design work-flow. This includes methods to evaluate field data, laboratory testing, geotechnical modeling, and structural design. The final goal is a more seamless movement from site characterization to final design. Being able to bridge between geotechnical and structural analysis software is part of this process.

6 ACKNOWLEDGEMENTS

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