COMMENTARY ON
SOIL-STRUCTURE INTERACTION IN
U.S. SEISMIC PROVISIONS

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ABSTRACT

Soil-structure interaction (SSI) was first introduced in the 1978 ATC-3 tentative seismic provisions, which was the forerunner to the U.S. National Earthquake Hazard Reduction Program (NEHRP) seismic provisions. Except for minor technical revisions and editing, the SSI portion of these provisions has not changed since 1978, whereas, significant changes have been made to the lateral force procedures during this period. This paper identifies limitations in the way SSI is presently incorporated into the lateral-force procedure of the 2000 NEHRP seismic provisions and offers suggestions for making future improvements. A case history is presented that illustrates some of the problems and possible solutions for the design of a pile-supported building on potentially liquefiable soil.

Keywords: Soil-structure interaction, NEHRP provisions, ductility, foundation design, soil-pile interaction

INTRODUCTION

Although a tremendous amount of research on SSI has been conducted over the last 30 years, its acceptance and integration in conventional structural engineering practice has not occurred, despite efforts to introduce it into the U.S. national seismic provisions over the last two decades. The reasons for this lack of acceptance were discussed in the last U.S. Japan Workshop on SSI in 1998 (Celebi and Okawa, 1999). The purpose of this paper is not to further discuss this issue, but rather to first identify several technical problems associated with the integration of SSI into the current codes. This discussion is followed by proposals for (1) modifications to the lateral force provisions in these codes to include SSI for certain situations, and (2) long term, but focused, research to isolate the effects of SSI from superstructure ductility.

SSI AND PRESENT CODE PROVISIONS

SSI was first introduced in the ATC-3 (1978) provisions, which contained simple SSI procedures that offered the possibility of reducing the design base shear depending on the vibrational characteristics of the building-soil system. In some respects, the fact that these SSI provisions were never integrated into the Uniform Building Code (UBC) or other standard U.S. codes for buildings, was possibly a blessing because of some potentially serious limitations in the way the SSI was linked to the calculation of the design base shear. The design base shear (V) is first computed through a formula, $V = f / R$, where f is a simple function of the building importance, soil conditions, and building period, and R is the superstructure ductility. The R factor, which is typically greater than 1, allows for inelastic response of the structure in order to achieve an economic design that meets the basic life-safety performance objective of the code. The ATC-3 document and its successors, the NEHRP seismic provisions, allowed this base shear to be further reduced through the application of SSI, which was optional in these provisions. In theory, this concept has merit. However, the potential disconnect in this linkage between SSI and V is through the R factor. This factor is supposed to account for the energy-absorption capacity or global ductility of the superstructure and is only a function of the type of structure. Thus, a 2-story and 20-story building of the same structural type will be assigned the same R factor, which intuitively seems suspect, regardless of any SSI considerations. However, SSI also serves as an energy-dissipation mechanism (Crouse, 1999), and every structure experiences a certain amount of it during each earthquake because no foundation medium, whether it is soil or rock, is completely rigid. The focus of a building's good or bad earthquake performance has invariably been on the performance of the superstructure, unless of course obvious failure of the soil contributed to, or was the sole cause of, the structural damage. Consequently, the evolution of the R factors over the last two decades has been influenced by the earthquake performance of buildings during this period, as well as testing of mostly model structures and prototype elements or assemblies (e.g., shear walls, beam-column connections, etc.) in the laboratory. With respect to field performance, most buildings are relatively short- to mid-rise structures on soil sites, and for these structures, the effects of SSI will have been more pronounced than for the taller, more flexible structures. SSI would typically have a beneficial effect for these shorter structures through greater energy dissipation and lengthening of the fundamental period of the structure. Thus, if in their assignment of R factors, the code committees were influenced by the performance of the more prevalent inventory of smaller buildings on soil, then the absolute values of the R factors reflect a certain amount of SSI. Unfortunately, the portion of the R factor attributed to SSI is unknown. If it represents a significant amount (i.e., more than about 20%), then the application of these R factors to the design of structures where SSI effects are expected to be minimal (e.g., stiff structures on rock, or tall flexible structures on soil or rock) would result in a design with an implicitly lower factor of safety.

SUGGESTED REVISION

Whether and to what extent the R factors are contaminated with SSI effects are debatable issues that cannot be adequately resolved without further research, which is a long-term proposition involving structural and geotechnical engineers, as well as experts in SSI that can provide the link between these two groups. This research program must necessarily involve a re-examination of the accelerogram data recorded within buildings during earthquakes in the U.S. and ideally Japan, two countries that possess the vast majority of these data. To facilitate this endeavor, a focused cooperative research program should be launched. On the U.S. side, the evolution of the R values should first be documented to the best extent possible before information is permanently lost through death or failing memories of the early developers. This technical history would indicate where future research should be directed. In the meantime, the allowance of a reduction in the base shear for SSI, as it is currently proposed in the NEHRP provisions (BSSC, 1998), should be discontinued or carefully considered in view of the above discussion.

However, some form of SSI should be mandatory for those structures founded on soils expected to fail during the design earthquakes. The potential problems in design that could result in these situations, and practical SSI methods that can assist in proper design solutions, are illustrated in the following case history.

CASE HISTORY

The illustrative example is a 5-story special steel concentrically braced frame structure founded on potentially liquefiable soil. Because of cost, a decision was made to support the structure with piles (rather than improve the soils) and accept the fact that the soils would liquefy during strong shaking. This solution was acceptable because lateral spreading of the soils was not an issue. However, this situation revealed a flaw in the preliminary foundation design through the standard code approach. The original pile-foundation concept is shown in Figure 1, which shows steel vertical H piles embedded 15 cm (6 in) in a concrete pile cap 80 cm (2.5 ft) thick. The total foundation system consisted of over 10 types of pile caps, each one with different dimensions and different number of piles. The number of caps within each type was also different, but all pile caps had the same basic connection detail shown in Figure 1. The pile caps were connected by grade beams, and a rigid floor diaphragm was connected to the pile caps and grade beams to form an integral foundation. The total number of piles was 2,600 and the plan area of the foundation was approximately 200 m x 155 m (660 ft x 510 ft).

The design base shear for the structure was computed using the formula, $V = (C_V I / RT) W$ in the 1997 UBC. A value of $R = 6.4$ was selected based on the building type, and the design could have proceeded according to the code without any modifications to the foundation. The apparent adequacy of the foundation design is illustrated in Figure 2 for three soil profiles representative of the site. This figure shows the load-deflection curves for the entire foundation during liquefaction, which was assumed to occur in each profile. The foundation, in which the top of each pile was analyzed as a free-head condition using the LPILE program (Ensoft, 1997), fails at loads above the design load, shown as the lower thick black horizontal line in the figure. Thus, the foundation performs satisfactorily in the sense that excessive displacements at the design load, possibly leading to catastrophic failure, are not produced.

Although legitimate in terms of code compliance, the foundation design and code specification of the foundation earthquake load are both flawed. Once the soils underneath the building liquefy during the design earthquake, the structure-foundation system clearly has a soft first story, where in this case, the first story is between the pile tip and pile cap. The weak link in this system is the non-ductile pile to pile-cap connection. Thus, the system as a whole deserves a much lower R factor than the 6.4 value derived for the superstructure. Beside the soil liquefaction and the lack of foundation ductility, another concern was the likelihood that during the design earthquake, the actual base shear would be much higher than the value obtained from the UBC. This phenomenon is commonly observed in response measurements made in instrumental buildings during moderate to large earthquakes (e.g., Housner and Jennings, 1982; Werner et al., 1991; Jennings, 1997), and is the basis for the allowance of nonlinear response in the codes. However, in this example, the foundation probably would have failed, and such a failure was judged to be unacceptable from a life-safety standpoint.

One approach that involved implementation of practical soil-foundation interaction software and simple statics proved useful for the design of an acceptable foundation. This approach is illustrated in the following paragraphs.

The first step was to develop a ductile foundation concept. Of the several concepts proposed, the one that was selected is shown in Figure 3. Note the cap thickness increased from 80 cm (2.5 ft) to 1.80 m (6 ft) and the pile was embedded 0.90 m (3 ft) into the cap. A steel plate is welded to the top of the pile to offer more bearing area for axial loads and the embedded portion of the pile is surrounded by a spiral rebar cage to confine the concrete around the pile. This detail provides ductility and moment resistance to limit the lateral deflections.

Next, a static soil-pile interaction analysis was performed using the LPILE program (Ensoft, 1997), a commercial software code that is used by many geotechnical firms. Pile-head load-deflection curves were computed with this code under the following assumptions: (1) the connection between the pile and pile cap was fixed and the cap was not allowed to rotate, (2) liquefaction was assumed to occur, and (3) the individual nonlinear p-y curves, which model the interaction between segments of the pile and surrounding soil, were assumed to be a small fraction of similar curves for the non-liquefaction case. Fractions (called p multipliers) of 0.1 and 0.2 were assumed. The resulting load-deflection curves are shown in Figure 4 (p = 0.1) and Figure 5 (p = 0.2).

By comparing Figure 5 with Figure 2, note the large increase in capacity achieved by the fixed-head condition versus the free-head condition. Although the problem appears to be solved, additional checks were required because in reality the pile caps will have a tendency to rotate from the application of the base shear to the top of the caps. If the pile caps rotate, then the lateral deflection of the pile group will increase, which was easily demonstrated with the GROUP pile-group program (Ensoft, 1999), another commercial software code. Because the lateral deflections had to be kept within certain limits for structural stability reasons, the structural engineer elected to limit these deflections by redesigning the grade beams, rather than increasing the cap plan dimensions and adding more piles. For a trial reinforced concrete grade-beam section, 0.6 m wide by 0.9 m deep (2 ft x 3 ft), a grade-beam/pile-cap system was modeled as shown on Figure 6. Because the soil was liquefied, it was assumed to offer no resistance to the vertical deformation of the grade-beam. For each different pile cap comprising the foundation system, an equation was written describing the moment equilibrium about pt 0 in the free-body diagram at the bottom of Figure 6. This equation was solved for the rotation, θ . As shown to the right of the free-body diagram, the moment-rotation relationship of the pile group, M_P vs. θ , is nonlinear because the axial load-deflection behavior is nonlinear, as predicted by the APILE program (Ensoft, 1999), a companion of the LPILE software. The APILE program predicted nearly elastic-plastic behavior for each pile, and the axial deflections induced by the rotations and axial loads were assumed to be in this linear elastic range, which was verified by subsequent analyses.

From all the analyses described in the previous paragraphs, the structural/geotechnical engineering team concluded that the foundation and superstructure would perform adequately during the design earthquake.

CONCLUSIONS

Revisions to the SSI section and R factors in the current NEHRP provisions appear to be necessary based on the observations presented in this paper. However, any significant revisions will need to be supported by well-planned systematic research, which should involve our Japanese colleagues. Fruits of this research could be realized within several years after initiation; however, the use of the results to revise the seismic provisions, including the approval process among the code committees, will likely take several more years. Thus, new provisions with the SSI and R factors successfully integrated would most likely be published toward the end of the decade at the earliest. In the meantime, the codes should be revised at the first opportunity to require that: (1) some form of SSI be conducted for buildings on soils prone to failure during the design earthquake, and (2) the foundation be designed to have a ductility similar to that of the superstructure for this case.

The SSI procedures in the current NEHRP provisions that allow a reduction in the base shear, after the base shear has been reduced by an R factor, should be used with caution or ignored altogether. This recommendation does not apply to non-conventional structures, such as offshore platforms, nuclear power plants, LNG tanks, or other special structures that generally do not employ code-based ductility-factor concepts in the design. For these structures, SSI analysis is a perfectly legitimate and accepted procedure for computing seismic loads.

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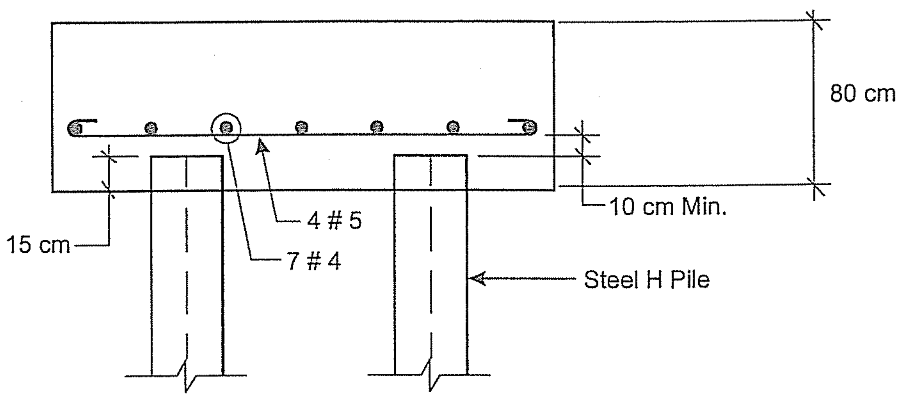

Fig. 1. Preliminary connection detail of pile to pile cap.

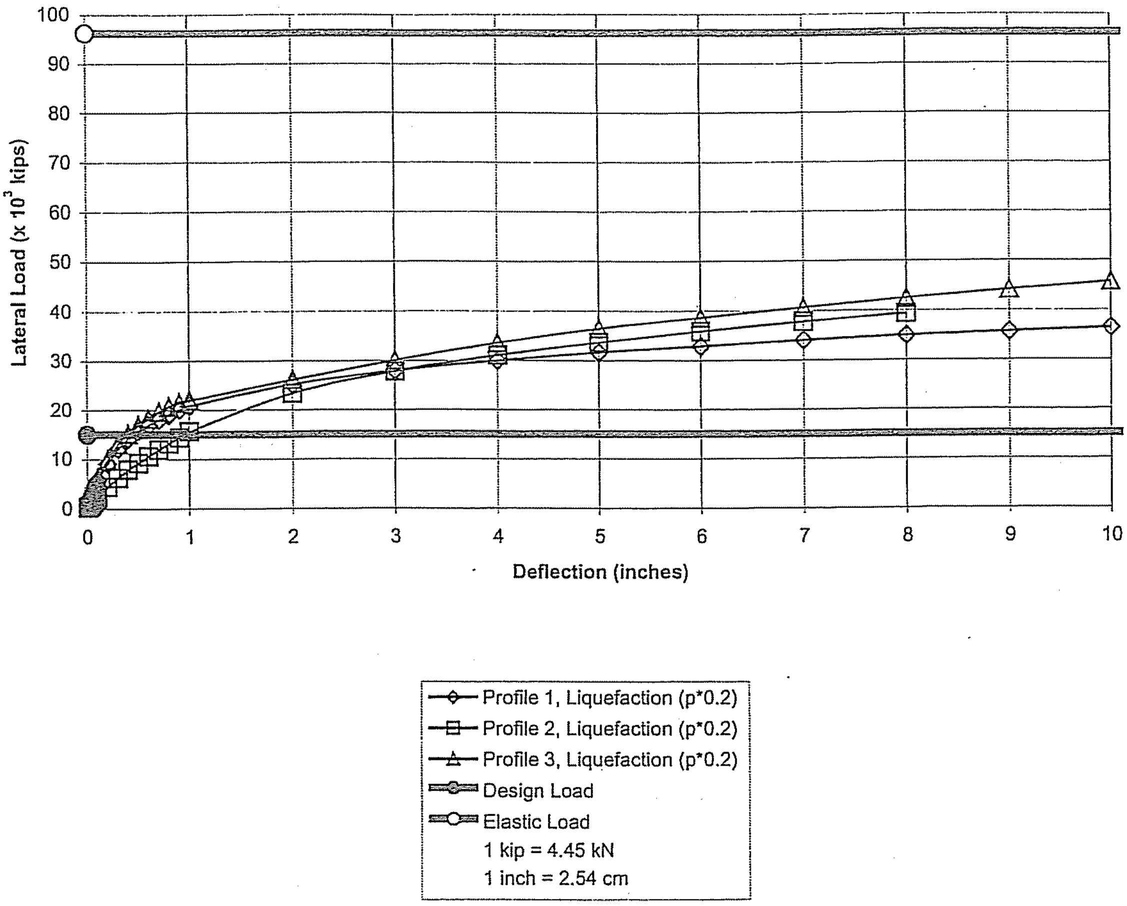

Fig. 2. Pile load vs. deflection with soil liquefaction (p multiplier of 0.2) for assumed free head condition.

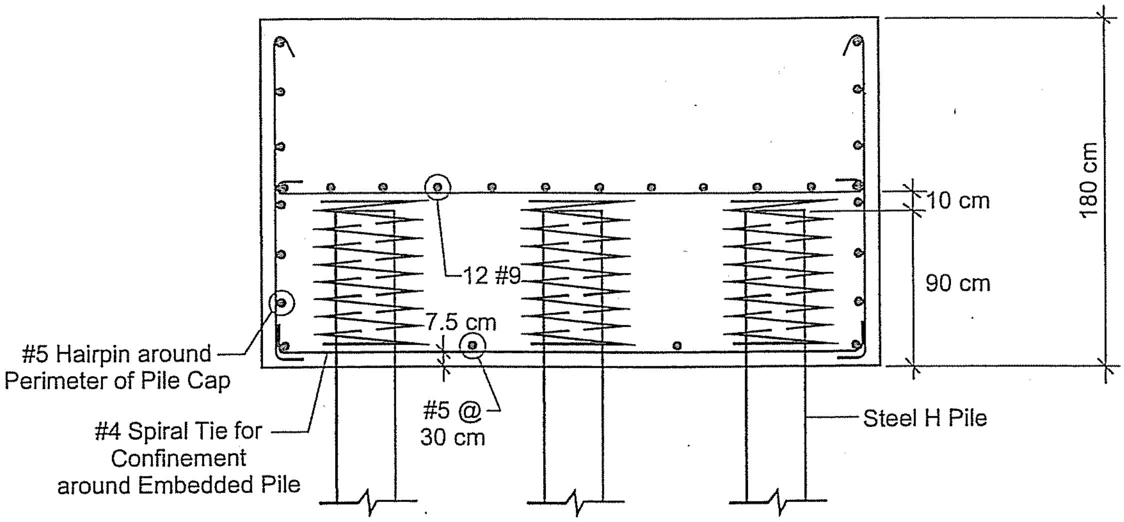

Fig. 3. Ductile moment resisting connection between piles and pile cap.

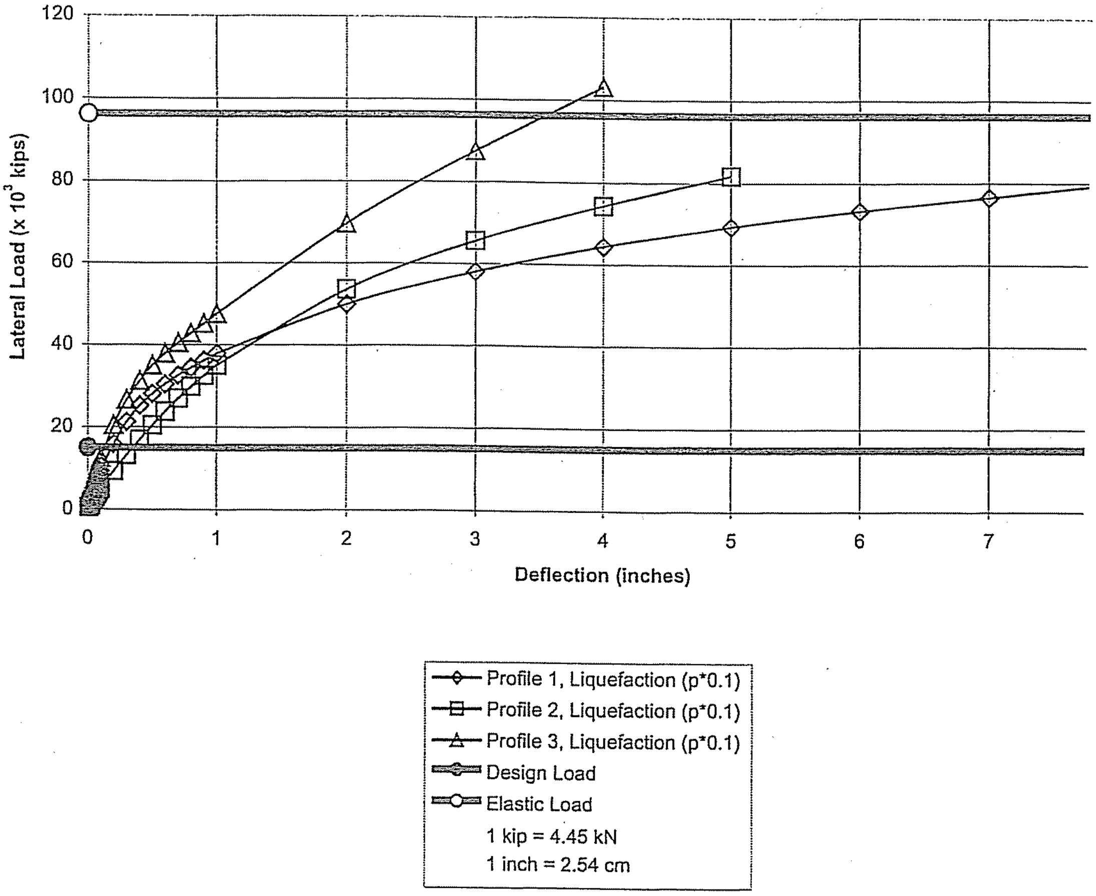

Fig. 4. Pile load vs. deflection with soil liquefaction (p multiplier of 0.1) for assumed fixed head condition.