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Earthquake-Resistant Design of Foundations

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ABSTRACT

The present-day design philosophy adopts the same types of foundations for buildings in seismic zones as for non-seismic zones with the additional consideration of seismic forces in a somewhat arbitrary manner. A commonly adopted design approach is to restrain the relative movements between footings by connecting strut-ties, grade beams, walls, or slabs. However, guidelines on design of these elements are either sketchy or not available. The code provisions are too general and often inadequate. Moreover, foundations have a low residual economic value. As a result, foundation design for seismic zones is often given a lower priority by designers than what it deserves. This exploratory paper presents a review and critique of current code provisions for seismic design of foundations in the United States and offers some suggestions for future research. This issue is timely and of particular interest since in recent years seismic hazard reduction and mitigation have assumed a renewed national urgency in the United States, Japan, and several other countries.

INTRODUCTION

Foundation design considerations are widely varied and complex in nature because of variable site and soil conditions, building configurations, different types of foundation, and a variety of structural systems. Foundations in a seismic zone have dual characteristics in terms of load transmission. These are related to the seismic forces being transmitted to the foundations directly from the ground and the inertial forces being transmitted from the building superstructure to the foundation. Critical factors influencing foundation design may be summarized as follows (Wakabayashi 1986): 1) Input and output force characteristics of foundations; 2) Strength and deformation characteristics of foundations; and 3) Strength and deformation characteristics of the ground.

The underlying notion of current design approach is to interconnect all the footing elements into a single rigid mass in order to restrain any relative movements of these elements. Detailed guidelines on this issue are limited at present either in the codes or the literature. For example, the Uniform Building Code (UBC 1991) does not provide any specific or detailed requirements for the design of strut-ties or grade beams. As a result, designers often resort to arbitrary design decisions when confronted by this issue. Other code requirements on foundation design also appear to be inadequate.

The objective of this paper is to review how the UBC-91 Code treats the foundation design issue and to point out where gaps in our present state of knowledge exist. In addition, the recommendations of the National Earthquake Hazards Reduction Program (NEHRP 1991) are also briefly reviewed. The paper also discusses some of the current thoughts on this subject and

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offers recommendations for the design of foundations in seismic zones in a more rational manner. The paper is limited to the structural design of foundations and does not address the design problems related to soil movements and characteristics, although such geotechnical considerations are of significant interest to designers in many situations. The paper assumes compliance with these requirements. It is also assumed that a foundation will be capable of withstanding all forces transmitted from the superstructure (i.e., base shear, vertical force, and overturning moment). It is generally recognized that the present UBC-91 Code requires foundations to be designed only for gravity and code level seismic loads, although in reality larger forces will develop in the structure during a major earthquake. Since foundations carry the entire structure, they should be designed conservatively and with due regard for ductility and energy dissipation phenomena. Also, underground repairs of foundations are expensive, and hence foundations should not be, as a rule, allowed to fail before the superstructure or to undergo inelastic deformations. Although the concepts presented in this paper are discussed as they primarily relate to building foundations, they are deemed to have relevance to other types of structures.

FOUNDATION TYPES

Foundations may be classified into two broad categories: 1) Shallow foundations and 2) Deep foundations. Common examples of shallow foundations are isolated or individual spread footings, combined footings, strip footings, and mats. Examples of deep foundations are pile foundations and drilled piers or caissons. Pile foundations may employ timber, steel, or concrete piles. Concrete piles may be augured (i.e., cast-in-place) or precast. Piles may be rigid or hinged at pile cap. Pile caps are required for groups of piles. A cap is often provided even for a single caisson. Basement retaining walls may also be considered in the category of shallow foundations. The role of basement walls is different from conventional shallow foundations in that they resist lateral earth pressure rather than directly transmitting vertical forces to the ground, although in some cases they may be designed to perform a dual role. In other instances they are designed to resist low soil pressure and transfer vertical loads to column foundations by way of acting as deep beams.

DISCUSSIONS ON UBC-91 CODE REQUIREMENTS

The UBC-91 provides a few requirements for foundation design, some of which are general in nature whereas others are more specific. For example, Section 2337(b)10 specifies the general requirement of providing strength and stiffness of the framing between the base and the foundation to be not less than that of the superstructure. This requirement is also stated in Section 1H2(k) of Structural Engineers Association of California Code (SEAOC 1988). The definition of "framing between the base and the foundation" is ambiguous at best and is open to interpretation. Does this definition include grade beams? Stiffness is certainly not synonymous with ductility. The code is not clear about the ductility issue, i.e., design and detailing requirements of the grade beams in general and with regard to ductility in particular. The special detailing requirements of Chapters 26 and 27 of the UBC-91 Code are applicable to columns supporting discontinuous lateral force-resisting elements and to special moment-resisting frame (SMRF), intermediate moment-resisting frame (IMRF), and eccentric braced frame (EBF) system elements below the base which transmit the lateral forces to the foundation. This requirement does not seem to address the grade beams, i.e., the foundation itself.

UBC-91 Code's Section 2910(a) requires that in Seismic Zones 3 and 4, the further requirements of this section are to be met for the design and construction of foundations, foundation components and the connection of superstructure elements thereto. In Section 2910(c), it is reiterated that the connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements are required to be designed. Once again, these are broad statements, without actually addressing the specific design requirements or performance criteria for foundations in seismic zones.

The specific code requirements for the various foundation types are reviewed in the following. It may be noted here that such requirements are generally very sketchy. A brief discussion follows the code requirement.

Individual Spread Footings. The individual spread footings are the most commonly used foundation system. UBC-91 Code provides the detailed regulations for foundations in Part VI, and although it does not specifically state any requirements for interconnection of spread footings or for ductility and reinforcing details of the footings in seismic zones, Section 2312 (h) 2E requires all parts of a structure to be interconnected.

It is common practice to control relative movements between spread footings by introducing connecting tie beams/grade beams, slabs, walls, etc. to tie the structure to the footings such that the entire foundation system acts as a single unit with respect to the soil. Applied Technology Council (ATC-3 1978) recommends the use of a horizontal force for the tie beams as a function of the axial force in the columns. It recommends that strut ties are designed for tension or compression for a force equal to 25 percent of the larger of the footing and column load multiplied by the effective peak velocity-related acceleration, A_v. On the basis of the ATC-3 formula, a conservative value of 0.1N where N is the axial force in the columns may be used for the horizontal force in the connecting element, in the absence of a precise value from a lateral load analysis. This is also what is directly specified by SEAOC-88 in Section 1J5b(2) in regard to the requirements for footing interconnection, i.e., to take the force in the tie as 10 percent of the larger column load. The actual design implications are discussed in the following.

The concrete cross-section and the reinforcement requirements (both longitudinal and transverse) may be determined by considering the tying element as a compression member subjected to the aforementioned axial force. Since this element may act in tension as well as in compression, the reinforcement in it should be extended into the footing by providing development length in tension. For spread footings under columns that are not part of the lateral load-resisting system, the tie beams may be designed as for the minimum requirement for column under axial compressive load.

As stated earlier, Section 2312(h) 2E of UBC-91 Code requires all parts of a structure to be interconnected. The practice of connecting the isolated spread footings conforms to this broad notion, although no detailed guidelines on the connecting elements are provided. Most designers, at any rate, interconnect the spread footings as a matter of good practice and to "play safe." When uplift is present at the columns at the foundation level and the designer wants to provide grade beams between the footings, such beams act as flexural members and should be designed as such. In SMRF and IMRF, when isolated footings are interconnected by grade beams (i.e., flexural members) some ignore these beams in the frame analysis and include the grade beams as an afterthought without recognizing that this could result in a redistribution of forces and moments. Based on the connection detail at the ends, two distinct possibilities exist: 1) grade beams are pinned at the ends; and 2) grade beams are fixed at the ends. When the grade beams are pinned at the ends, they may be designed for pure axial force (i.e., as for tie beams), and their influence on the frame analysis may not be as critical. When grade beams are fixed at the ends, they should be included in the frame analysis and should be designed for the shears, axial force, and moments in conjunction with providing sufficient ductility as for other beams in the superstructure.

Another important consideration, and as alluded to earlier, is the question of uplift of foundations. This is not clearly addressed by the code although it could be quite critical for EBFs, concentric braced frames (CBF), and shear wall systems, and as a result uplift of spread footings under forces greater than those specified by the code is generally accepted to be tolerable. This seems illogical since uplift under seismic conditions (even for CBFs and EBFs) should not be encouraged and when uplift is at all found to occur, grade beams should be designed to provide the required resistance by way of transferring the uplift forces to adjacent footings. This is particularly important for SMRF and EBF structures where ductility of the superstructure can be exploited only if the foundation has enough strength and does not undergo any uplift, i.e., the design philosophy that ductile structural failure shall precede foundation failure is implemented. The grade beams should still be detailed for ductility to avoid any brittle type of failure, i.e., the prescribed ultimate strength should be reached in the beams in a ductile manner. The prescribed ultimate strength of foundations for SMRF and EBF structures should also be higher than that of ordinary moment-resisting frames (OMRF) and CBF structures. Foundation strengths for OMRF and CBF structures could be based on simply code specified forces. For example, for SMRF, EBF, and dual systems with an R_w value of say 9 or greater, foundation could be designed to resist higher levels of seismic forces for uplift, sliding, bending, shear, etc. Grade beams should always be provided for tying action for all systems regardless of whether they are utilized to resist uplift forces or not. The strength of the foundation in such cases may be determined from considerations of the strength of the superstructure elements assuming material overstrengths, maximum forces that would occur in a fully yielded structural system or 3 (R_w/8) times the design seismic forces in the superstructure elements, etc.

When grade beams are included in the framing system and a rigorous analysis is done, they should be designed for strength and ductility exactly as for other beams in the superstructure. However, if no detailed frame analysis is done and the grade beams are introduced and designed to resist vertical uplift forces only, the minimum steel requirements (for both longitudinal and transverse reinforcement) for a flexural element should be met. In a practical situation a grade beam, even though designed for flexure, will also act as a strut-tie to resist horizontal forces and as such the minimum reinforcing requirements should cover both conditions of flexure and axial compression. For grade beams with large cross-sections and subjected to a small axial force, if 0.5 percent steel as required by the ACI Code for columns is provided as longitudinal reinforcement, it may appear high at first sight compared with a steel ratio of $200/f_y$, where f_y is the yield strength of reinforcing steel, or the steel ratio computed when the ultimate moment capacity provided is greater than or equal to the factored ultimate moment multiplied by 1.33 as per ACI-10.5.2. However, 0.5 percent of steel is applied on the gross concrete section and hence top and bottom reinforcement will each comprise 0.25 percent of steel for a compression member, whereas 0.33 percent for f_y = 60 ksi is applied on the effective cross section (bd) and has to be provided at both top and bottom faces of the beam resulting actually in 0.66 percent of steel. It would be logical then to select the larger of the two values which, in most cases, will be 0.33 percent for each face of beam. Similar comments apply to the selection of transverse reinforcement.

Another question that the code does not address is whether spread footings themselves need to be ductile. A spread footing could be subjected to a large lateral load and a strong overturning moment during an earthquake. With the alternating and repeated nature of such action, the soil under the footing edges is compressed and the foundation may rock violently resulting even in tension at the top of the footing, because of the tendency of the footing to rotate and to lift up one edge, particularly if the footing is a deep one. In a deep footing, the center of rotation is on the center line of the footing between the ground surface and the bottom of the footing, whereas in a shallow footing, its location approaches the toe of footing, resulting in a rotation of the entire footing about this point (Ambrose and Vergun 1990). It is possible that during a seismic event, such complex effects that are not amenable to simple mathematical analysis may result in a brittle type of failure, if there is no top steel. For footings that are subjected to uplift due to combined dead and seismic loads, it may be possible that the weight of soil overburden will eliminate this uplift although the footing may still develop tension at the top, since it will push against soil at the top. A combination of footing uplift and rotation will only aggravate the problem. Considering all these factors, provision of top steel in the footing will not only eliminate the possibility of tensile failure at the top of the footing, but will also increase the ductility of the footing. Of course, provision of stiff grade beams at the level of the footings will alleviate the problems of rocking, rotation, and uplift of footings. However, adding some top bars in footings may be advisable until more knowledge and information are available on this issue. Evidently, there will be resistance from the practicing profession at the thought of providing top bars in footings which will incur additional cost. To begin with, the footings along the perimeter of the structure where seismic effects are expected to be more pronounced may be considered as candidates where nominal top steel could be provided.

Combined Footings. There is no specific reference to combined footings in the UBC-91 Code. The conventional design of a rectangular combined footing assures a linear soil pressure distribution under a rigid member and involves the determination of the location of the center of the footing area. Normally, negative reinforcement is required at the top of the footing between the columns. For reasons of erratic dynamic behavior of the footing during a major or severe earthquake, provision of nominal number of top bars throughout the length and width of the footing appears to be logical. This will also assure added ductility of the footing. The minimum steel ratio throughout the footing in both directions may be taken as $200/f_y$. Further, grade beams connecting combined footings with other footings should be provided as usual.

Strip Footings. Strip footings under walls are not addressed in the code. Using the same concept as before, top and bottom transverse reinforcements are desirable in such a footing. This may, however, involve a large quantity of additional steel. As such top steel may be limited perhaps in areas where uplift is a possibility, i.e., in corner areas of the buildings, where overturning effect coupled with lesser magnitude of dead loads from floors and roofs in these areas may result in a reduction in uplift resistance of the wall and loss of contact with soil. The "dishing out" effect of the footing due to normal settlement will also result in an aggravation of the problem at the ends of the footing. In the absence of a detailed analysis, it is recommended that nominal top transverse reinforcement be provided over at least 10 percent of the length of the wall in the corner regions.

Mats. The code does not make any reference to mat foundations in seismic zones. Mat foundations are ideally suited for buildings in seismic zones because of their continuity and rigidity. Just like any other foundation, adequate strength and ductility should be provided for mats and both top and bottom reinforcement should be continuously provided. However, since this may involve substantial amounts of additional steel reinforcing, further studies are required on this topic. In the interim, it is recommended that continuous top and bottom reinforcements are generously provided along the periphery of the mat in both directions, particularly when the slab cantilevers out from the exterior column line.

Basement Walls. The structural design of basement walls (either as a cantilever retaining wall or as a retaining wall laterally supported at the top) is a special problem. It is often assumed that the soil mass behind these walls have the potential of delivering an additional lateral seismic force if there is a slip plane. This force should be included in the lateral seismic forces on the wall that are delivered from the superstructure as well as generated by the acceleration of the mass of the wall. More research is required in this area particularly in terms of the soil failure mechanism and the location of the slip plane to get a better understanding of the magnitude of the lateral seismic force caused by the earth-mass behind the wall. For further discussion, the interested reader is referred to Seed and Whitman (1970) and Naeim (1989).

Pile Foundations. UBC-91 Code specifically requires interconnection of pile caps and caissons in Section 2908 (b), and it is also required by SEAOC-88 in Section 1J5. The special requirements for piles and caissons for foundations in Seismic Zones 3 and 4 are included in Section 2910 (e) of the UBC-91 Code. It calls for designing piles and caissons for flexure wherever the tops of such members will be displaced by earthquake motions. The criteria and detailing requirements of Section 2625(e) for concrete and Section 2710(e) for steel (i.e., columns under combined axial load and bending) shall apply to a length of these members equal to 120 percent of the flexural length, which is defined as the length from the point of fixity to pile cap. Full details of pile design are presented in Sections 2908-2910.

Because of the low magnitude of the lateral resistance of pile foundations, UBC-91 makes it mandatory to connect the individual pile caps and caissons, and transfer the lateral forces to the other stable parts of the building with the help of ties, grade beams, etc. Such tying action also provides lateral stability of pile foundations under vertical loads. For the design of ties or grade beams, a similar procedure as discussed before for spread footings could be followed. UBC-91 requires in Section 2908(b) that ties shall be capable of resisting, in tension or compression, a minimum horizontal force of 10 percent of the larger column vertical load. Other approved methods may be used when it can be shown that equivalent restraint can be provided. Friction piles have generally considerable resistance to uplift forces, although such resistance is lower for tapered piles. For structures where the lateral forces cannot be transferred to other building parts, battered piles are often used to resist lateral forces. Many designers, however, prefer to design foundations with vertical piles provided with moment resistance. Battered piles result in unequal amounts of flexural and shear reinforcement in the piles caused by the axial load eccentricity.

It is evidently necessary to clearly stipulate in the code the design requirements for piles and the details for anchorage at the pile heads. More guidelines are needed on pilecap reinforcing, spacing of piles, ductility of pile caps, and pile confinement reinforcing. Such guidelines are expected only from further research.

REVIEW OF NEHRP-91 PROVISIONS

The NEHRP provisions (NEHRP 1991) prepared by the Building Seismic Safety Council (BSSC) for the Federal Emergency Management Agency (FEMA) and based on ATC-3, constitute a major step forward in the development of a comprehensive seismic code. The two codes, i.e., UBC-91 and NEHRP-91 attempt to provide minimum standards for the design and detailing of the spread footings and pile foundations--the two most common types of foundation. As for ATC-3 requirement, Section 7.5.2 of the NEHRP-91 Code requires strut ties or the equivalent to interconnect the individual spread footings, in tension or compression, for a force equal to the effective peak velocity-related acceleration (A_v) divided by 4, unless the footings are founded directly on rock, as defined for Soil Profile Type S_1 (NEHRP 1991). Ties are not required in Seismic Performance categories A, B, and C.

As for spread footings, NEHRP-91 requires in Section 7.4.3 all individual pile caps, drilled piers, or caissons to be interconnected by ties. However, ties are not required in Seismic Performance Categories A and B. Reasons for providing the ties are explained in NEHRP-93 Part 2: Commentary. Other special pile requirements are presented in Sections 7.4.4 and 7.5.2 for Seismic Performance Categories C, D, and E, respectively. Detailed explanations of these requirements are given in the Commentary.

SUMMARY AND CONCLUSIONS

The paper has discussed the provisions of UBC-91 and reviewed the requirements of NEHRP-91 for the seismic design of foundation. Many of the recommendations provided in this paper need further investigations. Although the currently prevailing design practice underestimates the importance of foundations in seismic zones, there is, however, a growing recognition that foundations should be designed to perform better than the structure they support. The SEAOC ad-hoc Foundation Design Committee is currently working towards the goal of formulating rational and practical code provisions for foundation design with this realization.

The ad-hoc committee has developed a matrix of structural versus foundation systems that represents a systemization of the function and structural integrity of foundations in seismic zones. Based on the three basic performance requirements of strength, stiffness, and ductility, the importance of each foundation system is ranked into categories of A (very important), B (somewhat important), and C (less important). The matrix, which is preliminary in nature, covers a wide range of foundation and structural systems. This matrix and several other important issues and criteria are included in the committee's progress report entitled "Towards More Rational Code Provisions for Foundation Design." The committee expects to develop specific provisions for the SEAOC Blue Book and the 1997 edition of UBC.

The topic of earthquake-resistant design of foundations is an open area of research. There are too many concerns that could be addressed. However, broadly speaking, the following issues warrant immediate attention of researchers.

- Ductility of foundation systems, i.e., how it influences the overall structural response and how it could be ensured.
- Investigation of foundation failures in buildings caused by actual earthquakes (past investigations have often been limited to superstructures and geotechnical aspects rather than structural aspects of foundations).

- Foundation behavior in different types of buildings, e.g., high-rise vs. low-rise, massive vs. lightweight, moment frames vs. box-type structures, etc.
- Analytical modeling of foundation systems to predict accurately strength and stiffness characteristics of foundation systems and components.
- Experimental simulation (e.g., shake-table tests) of foundation behavior on reduced structural models.
- Detailed study of possible failure modes of foundations in seismic zones.
- Dynamic response of foundations subjected to seismic excitation.
- Studies on connections and detailing.
- Development of rational and practical code provisions.

Recognizing the fact that seismic foundation design provisions are lagging far behind other seismic requirements of the code and that research efforts in this area are highly inadequate, the author hopes this paper has served to bring this matter to the attention of those who are interested in seismic design.

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