DETAILING REINFORCED CONCRETE COLUMNS FOR SEISMIC PERFORMANCE

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SUMMARY

Reinforced concrete columns require special design and detailing considerations if they are to perform adequately in seismic exposure. Code requirements such as those presently in use in the United States provide a good start, but additional considerations are necessary for proper performance. Design related factors include the reserve capacity in columns as affected by recent code changes for non-seismic loadings, the effects of unreinforced cover and simultaneous orthogonal loading. Detailing effects evaluated include the need for more ties in the midheight of columns and providing adequate shear reinforcement from higher than anticipated moments, as well as the need for cyclic laboratory testing to improve understanding of the details of tie reinforcement.

INTRODUCTION

Columns exist in virtually all buildings and maintenance of their integrity in earthquakes is essential if good building performance is to be achieved in a seismic event. Failures and collapses of reinforced concrete buildings in significant earthquakes give continuing evidence that concrete columns must be designed, detailed and constructed in a manner which will enable them to resist extremely high overloads into the inelastic range without significant failure. Columns must support the building's weight. If they fail, building collapse becomes likely.

Traditionally, the capacity and detailing of reinforced concrete columns for seismic exposure has been very strongly influenced by non-seismic considerations. In the United States the basic building code provisions for designing and detailing reinforced concrete columns ignore seismic considerations, with additional provisions added for areas of highest seismicity. However, these added provisions are at best patchwork and do not properly insure that columns will provide sufficient strength and ductility for adequate seismic performance. A similar situation undoubtedly exists in the code provisions of other countries with seismic exposure.

This paper attempts to explore some of the factors affecting the design and detailing of reinforced concrete columns for improved seismic performance. Factors affecting the design loads or calculated strength are presented first followed by a discussion of detailing practices and important considerations in that area.

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INFLUENCE OF LOADS AND CAPACITY

Reinforced concrete structures are generally designed in accordance with a Building Code or set of regulations adopted by governmental authorities. In regions of high seismicity in the United States, the code provisions generally applicable are contained in the current edition of the Uniform Building Code (Ref. 1). That Code incorporates basic provisions for reinforced concrete construction from the Building Code of the American Concrete Institute (Ref. 2) and seismic design loads and additional details from the Recommended Lateral Force Provisions of the Structural Engineers Association of California (Ref. 3).

The earthquake design forces for structural design are based on experience and anticipate some level of ductility, redundancy and uncalculated strength. It is well documented that the code seismic design forces will be exceeded by a large factor in a major earthquake. Thus, the balance between design loads and design capacity is very important as is their relationship with the actual anticipated structural response and the ductility provided by the detailing of reinforcement.

In recent years the design capacity or strength or allowable load of reinforced concrete columns has been drastically increased without change in size, reinforcement or concrete strength, as a direct result of research and codification of ultimate strength design procedures. The 1963 Edition of ACI 318, which introduced strength design to the United States, permitted column capacities nearly double those calculated under the previous codes, since the former allowable stresses were drastically increased to provide better correlation with the results of ultimate strength design procedures. The 1971 Edition permitted another 6 to 7 percent increase when load factors were slightly reduced. While these changes appear totally justifiable for gravity load situations or design for routine wind exposures, their impact on seismic design has been significant and in the author's opinion has never been adequately considered. It is difficult for many engineers to justify that a change in calculated strength or in the load factors for dead and live loads has an impact on seismic performance. Many feel it improper to consider capactiy for these better known loads to accomplish reserve capacity for seismic loadings. Yet, these factors do affect the total strength provided; the newer codes provide less reserve capacity or uncalculated strength which historically has been beneficial in seismic resistant design.

In calculating the strength of a reinforced concrete column, we follow procedures developed for design situations where inelastic action is not expected. Thus, these procedures allow capacity for reinforcement, concrete within the column core and concrete cover over the reinforcement. This is a proper procedure for the non-seismic design situation. Yet, we expect that the concrete cover will spall in a major earthquake leaving only the column core to provide the strength at critical regions. Ideally, we should use only the concrete within the core for seismic design capacity, neglecting the concrete cover. However, the lack of design aids and tables for that condition has prevented the idea from advancing beyond an unpopular suggestion. The most obvious examples of the effect of concrete cover is the fact that our codes permit a higher capacity for a square column with spiral reinforcement than for a round column with identical spiral reinforcement. Our codes certainly should not permit this increased capacity for additional unreinforced concrete cover in seismic exposures. Realistically, practical design using available aids requires the inclusion of minimum cover in our design calculations, but this effect must be kept in mind when discussing design loads and details for ductile performance.

Traditionally, we design a structure for lateral forces in one direction at a time, determining the critical loads for each member from the critical direction. This follows the traditional approach in wind loadings where the wind blows in a single direction at an instance of time. The critical directions are routinely selected as the orthogonal axes of the building except for simple structures like the classical case of the four-legged elevated water tank covered in all elementary textbooks. However, maximum seismic response of a structure will occur in both directions simultaneously, so to be correct we should consider simultaneous response in design. In framed structures, beams are no problem as they work for only one direction of loading. However, columns must sustain the biaxial effects from both directions of loading. Traditionally, it has been rationalized that the structure and its columns have adequate reserve strength to accommodate the biaxial effects even when designed for only one direction of earthquake motion at a time. However, some proposed code provisions such as ATC-3 (Ref. 4), are recommending simultaneous loading in both directions. This is undoubtedly influenced by the reduced capacity of today's column designed by current code procedures. While considering both directions simultaneously may be technically correct, it greatly complicates analysis and design procedures and the simplified procedure of using increased load factors could be accepted as an alternative approach in simple structures. The important item is to provide sufficient capacity for biaxial considerations. This includes axial forces resulting from overturning moment in both directions as well as biaxial bending.

Another factor worthy of examination is the ratio of gravity loads to earthquake loads on a concrete column. Seismic design adopted the 1/3 stress increase for wind loads years ago, and this basic principal has carried over into strength design as expressed in load factors. A column designed to carry predominately gravity loads with small design seismic loads can resist a high percentage of seismic overload before inelastic performance is required. However, a column designed to carry nominal gravity loads and large seismic loads will reach its ultimate capacity at a relatively small percentage of seismic overload. The code ignores the difference in these two situations, but the earthquake doesn't. As designs become more daring and structural redundancy is reduced, adequate seismic performance is threatened. Tentative Provisions for the Development of Seismic Regulations for Buildings, otherwise known as ATC-3 (Ref. 4), has attempted to codify redundancy requirements, but such provisions cannot substitute for professional judgment necessary to provide reasonable gravity to seismic load member ratios and redundant structural systems.

Summarizing the influence of design loads and calculated capacities, it appears that the seismic capacity of concrete columns has been significantly reduced in recent years as more efficient non-seismic design provisions have been developed. Considering the negligible effect of a few extra inches of concrete or additional reinforcing bars in concrete columns on the total cost of a concrete structure, a tight design to current code provisions has saved nothing and jeopardized adequate seismic performance. It would appear that increased load factors of about 2.5 for all loads in column design or reduced strength reduction factors, $\emptyset$, in the 0.4 range for concrete column design would increase safety at costs so small they will never be distinguished within the accuracy of the competitive bidding system while reducing extensive calculations. In addition, only minimal concrete cover should be permitted in design calculations in seismic regions.

DETAILING CONSIDERATIONS

In addition to the need to have appropriate strength in a reinforced concrete column for adequate seismic performance, the proper detailing of reinforcement to provide ductility is even more important in seismic areas. Detailing practices of concrete columns cannot be overemphasized; they determine the eventual performance of the column.

Numerous papers have illustrated the difference in ductile performance of spiral and tied columns. The most graphic example was the performance of the Olive View Hospital in San Fernando, California in the February 1971 earthquake. Whereas nominally tied columns completely shattered, spirally reinforced columns in the same story maintained integrity of their cores and provided stability against collapse of the structure even with two feet (60 cm) of story displacement. However, the circular pattern of column reinforcement often interferes with beam reinforcement and tied columns continue to be the most widely used in the seismic regions of the United States. The following paragraphs discuss detailing considerations of both types of columns.

Little research has been performed recently with spiral columns in the United States, probably because their use is almost solely limited to the seismic areas of the West Coast. Recent earthquakes have clearly illustrated the need to install the spiral full length of the column and in the joint region when a deep beam frames into the column. Recent work in New Zealand (Ref. 5) has compared various spiral percentages with axial load and concluded that spiral requirements of ACI 318-77 are generally conservative but they underestimate the moment capacity, resulting in unconservative design for shear, particularly in the higher axial load ranges. Columns are designed in seismic regions to preclude shear failure, but when moment capacities are considerably greater than anticipated, this design rule is not achieved. The confinement of the core obviously increases the moment capacity. The ductility of spiral columns is well established and the fact that the full length of the column core is confined by the spiral is certainly a factor in that reputation. More work needs to be done in defining the moment and shear capacities of these members, as well as the effects of varying amounts of spiral reinforcement.

Tied columns represent the vast majority of columns constructed in the United States even in regions of high seismicity. The performance of the tied columns in the inelastic range, so essential in resisting strong ground shaking, depends on the details of the lateral reinforcement - the ties. The ties have three major purposes: to provide shear strength together with the concrete, to prevent the longitudinal column reinforcement from buckling, and to confine the concrete in the column's core at regions of maximum compressive stress from combined axial load and flexure.

Tie spacing or amount of tie reinforcement is the first critical factor to discuss. Code requirements in the United States require transverse reinforcement of

$$A_{sh} = 0.30 s_h h_c \frac{f'_c}{f_{yh}} \left(\frac{A_g}{A_{ch}} - 1 \right)$$

$$\text{or} \quad 0.12 s_h h_c \frac{f'_c}{f_{yh}}$$

where s_h is the hoop or closed tie spacing and h_c is the greatest core dimension. This transverse reinforcement is required only at the ends of the typical columns and through the beam-column joints in certain conditions. The midheight of the column needs to have ties designed only for shear strength, and the result is generally ties at large spacings approximating nominal code requirements for non-seismic regions. This tie formulation also contains some arbitrary judgmental factors which possibly could be improved by extensive laboratory testing. However, added ties at the ends of the columns have long been recognized in California construction as a beneficial detail, although only recently codified. A graphic example of this beneficial effect was observed in the Lima, Peru earthquake of October 3, 1974 at the Agricultural University where two, one-story classroom buildings had dramatic column failures, but additional ties at 8 cm (3 inch) spacing prevented failure at the column end, as seen in Figure 1. The failure was located at the beginning of the nominal tie spacing of the midheight of the column.

The recent column failures of the County Services Building in El Centro, California in the earthquake of October 15, 1979, further illustrated this concern. The concrete frames were designed for ductility with transverse reinforcement (ties at 2 to 3 inches or 5 to 8 cm on center) in the joints and for about 2 feet (60 cm) at the ends of each column. Although overturning forces from discontinuous shear wall above undoubtedly contributed significantly to the failure, the columns at one end of the building all failed in the midheight region immediately above the closely spaced ties which began at the top of the pile cap (Figure 2). Whereas we can evaluate relative member strengths at a typical frame joint, we enter an uncertain condition at the foundation where footings, pile caps or mats undoubtedly provide high degrees of fixity despite the analyst's assumptions. As the column will always be weaker than the footing, we appear to be building in column hinges above the base which may not be properly detailed. We certainly need more ties in these lowest columns to control failures like this recent El Centro example.

It is the author's opinion, that the area of greater concern in the present United States codification of ductile moment resistant space frames is the midheight of the column which results in minimal ties being provided when code provisions are routinely followed. The rational for the present code assumes a column member with uniform properties, but the author suggests that the effects of the confinement reinforcement at the ends of the column should be evaluated for the increase in stiffness and strength of that portion of the column. Thus, the column becomes like a haunched member with greater stiffness at its ends, and the critical section by analysis may in fact be at the end of the code required transverse reinforcement where minimal ties are installed. Research and testing is needed to investigate this vulnerable area. Tie reinforcement in this midheight in many circumstances might be less than the transverse reinforcement required at column ends, but more than present nominal design for column shear alone. Perhaps a set of ties at 6 diameters of the longitudinal bars as suggested by Bresler (Ref. 6) is an adequate compromise. The 6 diameters is based on revisions to Bresler's formulation for tie spacing (Ref. 7) to prevent bar buckling with improved data on the tangent modulus of reinforcement under cyclic loading (Ref. 8). Even this may be inadequate immediately above the foundation. Ties should be sufficient to prevent compression or shear failure in the concrete should a hinge form. The potential problem with increased tie spacing is the hinge may be prevented where ties are close, but forced to occur where confinement has been reduced.

Another factor affecting the performance of tie reinforcement is the details of the ties themselves. The most obvious factor is providing anchorage for the ends of the tie in other than the column cover which will spall in severe ground motion. This detail is currently satisfied in the United States by requiring ties in seismic regions to terminate with 135° hooks with a 10 diameter extension into the column core. Cross-ties or interior ties likewise should be adequately anchored into the column core at their ends to prevent premature failure. This provision becomes difficult to construct when ties are closely spaced and systems of alternate cross ties with 90° hooks have been used but have not been laboratory tested under cyclic loading conditions.

The discussion of intermediate cross ties raises another interesting point. The ACI Code prior to 1963 required every longitudinal bar to be enclosed in the angle of a tie. The provisions were relaxed in 1963 to alternate bars within 6 inches (150 mm) of a tied bar based on monotonic, non-cyclic tests. Since 1963, column ties in seismic regions have been specified by the same new provision, although no cyclic testing of this condition has been performed to compare with specimens where every bar is tied. Certainly, there is some difference, but we are currently relying on engineering judgment, not competent research, to justify this provision. It may be that the improved concrete placement resulting from less ties overweighs this concern, but it should be verified in the laboratory where the consequence of failure is less severe than in real structures.

Another factor affecting tied column performance is the pattern of longitudinal column reinforcement. Recent monotonic tests at the University of Toronto (Ref. 9) showed that columns with many longitudinal bars around its perimeter had superior performance in the inelastic range when compared to columns with fewer but larger bars giving similar percentages. These longitudinal bars, in effect, basket the core and provide improved confinement. However, it should be noted that the referenced test had ties at every bar, not alternate bars as permitted by the ACI Code.

Concrete columns need to be properly detailed if they are to provide satisfactory performance in significant earthquakes. The current United States code provisions adopted for use (References 1 and 3) are a good beginning. However, it is the author's opinion that certain details as required by the code will not always provide the desired or required ductility. Specifically, the midheight of the column should be detailed with more ties and research is needed to define the effectiveness of certain tie details under cyclic loading.

CONCLUSIONS

The current practice of design of reinforced concrete columns in seismic exposures has been discussed and several concerns expressed regarding these procedures. They may be summarized in the following conclusions:

1. Code requirements in the United States for calculating column capacity have changed over the past twenty years, greatly increasing the apparent capacity of reinforced concrete columns. These increased allowable loads have resulted in smaller columns for the same load, reducing the reserve capacity available to resist earthquake forces.

2. Columns should be designed for effects of earthquake loading along orthogonal axes considering overturning effects for both directions as well as biaxial bending.

3. Concrete columns in seismic regions should be designed ignoring the concrete cover.

4. If concrete column design in seismic regions continues to permit the calculation of column strength including the concrete cover, then only a minimum cover should be allowed in determining strength. Thus, the square column with a round spiral would have the same design capacity as a round column with identical spiral size and reinforcement.

5. The confinement effects of spiral and closely spaced tie reinforcement increases moment capacity with the result that shear forces are underestimated. If shear failures are to be prevented in concrete columns, moment capacities must be properly evaluated and the design shear must be determined on the basis of the moment capacities at the column ends.

6. The high degree of fixity of foundations and pile caps must be considered in columns in lower stories of framed structures. Confining reinforcement to provide ductility must be provided in the midheight of these columns where inelastic behavior will eventually cause a hinge.

7. Increased tie reinforcement appears necessary in the midheight of tied columns. The closely spaced ties only at column ends creates a haunch-like effect causing a discontinuity in member properties. A minimum tie spacing of 6 diameters of the longitudinal bars is suggested, although closer spacing may be necessary in some columns.

8. Laboratory research of cyclic loading is suggested to define various tie details, such as tying alternate bars, tie anchorage.

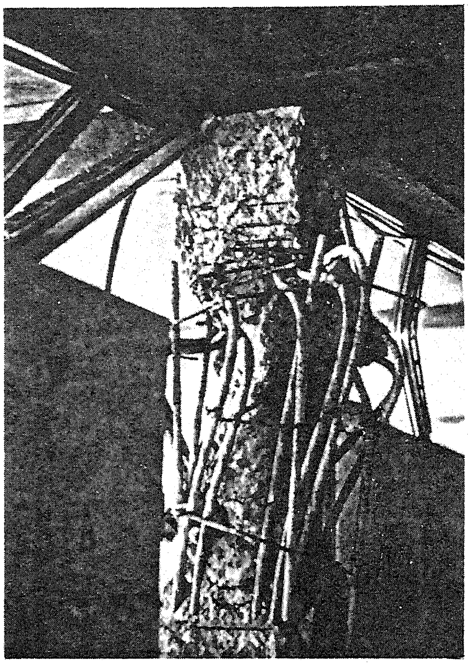

Figure 1. Column failure in Classroom Building of Agricultural University, Lima, Peru, in October 1974. Note beneficial performance from closely spaced ties at top of column.

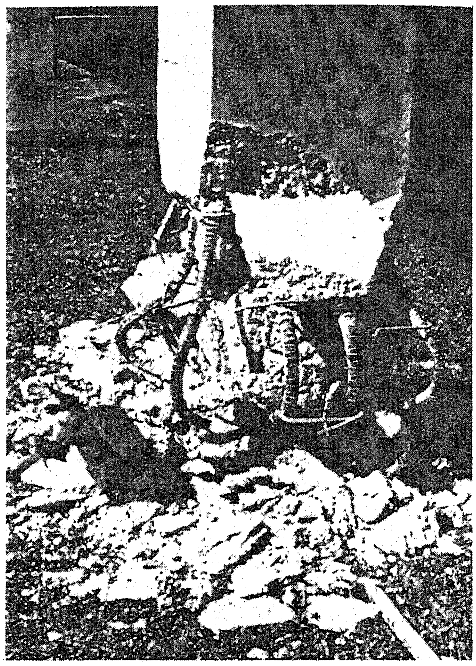

Figure 2. Column failure in Imperial County Services Building in El Centro in October 1979. Column had closely spaced ties immediately below failure.

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