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A More Rational Approach to Capacity Design of Seismic Moment Frame Columns

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Inelastic time history analyses typically indicate that the traditional sub-assembly "capacity" approach used in the design of ductile moment frames grossly underestimates the maximum moments experienced by the columns during a maximum credible earthquake. In addition, these analyses predict that the maximum column demand moments often occur near the mid-height of concrete structures, whereas a conventional elastic analysis predicts maxima at the lowest levels of these structures. Incremental displacement analyses using modal properties and displacements predicted by a maximum credible response spectrum should be used to more accurately predict the maximum anticipated column demand moments in the analysis of existing structures or the design of new structures.

INTRODUCTION

Currently, the design of ductile moment frames of both concrete and steel strive to provide a "capacity" design, or a design that attempts to maintain column strength while promoting beam plastic hinging. In steel special moment resisting space frames (SMRSFs) the sum of the columns at a joint are required to possess greater moment capacity than the sum of the adjoining beams, and in concrete SMRSFs the sum of the columns must possess six-fifths of the sum of the beam moment capacities at a joint (ICBO, 1994). The distribution of plastic beam moments to the columns is not dictated by the code, however most designers will either proportion the moments to the columns based upon the elastic distribution, particularly when "cantilevered column" effects (columns in single curvature at the lowest levels) are observed, or simply by assuming mid-height inflection points for this plastic state. The analysis of existing moment frame structures is also predicated upon the same "strong column-weak beam" philosophy, and the columns capacities are analyzed similarly to those for new design.

Unfortunately, non-linear inelastic time history analyses demonstrate that the methods previously described may underestimate the maximum column moment demands during a

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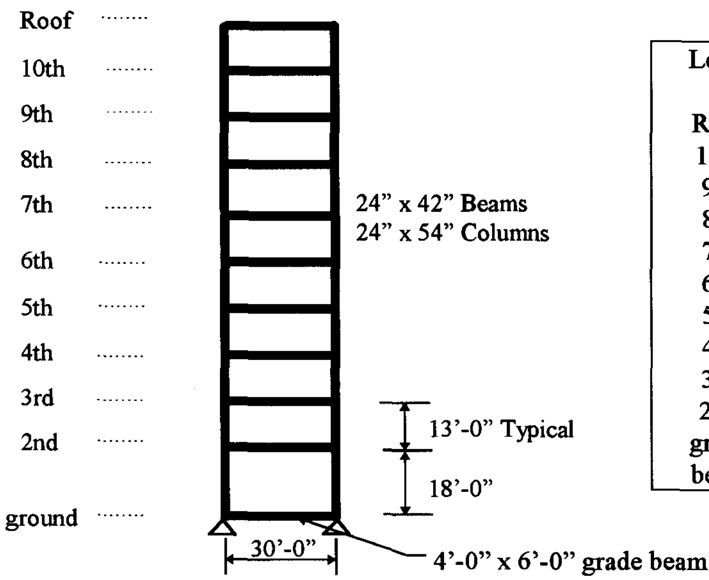
maximum credible earthquake by more than 100% in both concrete and steel structures (Park and Paulay, 1975; Bondy, 1995). The reason for the discrepancy is that the global column displacements and corresponding demands generated during the earthquake are ignored when the only column demands are assumed to be generated by the restraint of the attached beams. The increase in column moment demand is described as "dynamic magnification," and texts such as Paulay and Priestley's (1992) attempt to quantify this increase with the use of a magnification factor. The factor suggested is intended to be applied to the elastic frame moment distribution, and the factors were developed for concrete frames. While this may be practical for new construction of concrete moment frames, designers need to understand that this is not unique to reinforced concrete. Also, an engineer evaluating an existing structure may require a more building specific analysis technique in order to provide the most economically efficient retrofit solution.

There are many cases when a non-linear inelastic time history analysis may be impractical or impossible due to lack of data. Therefore, analysis techniques are required that will predict post-elastic demands with the use of a response spectrum, structural modal properties, and a frame analysis program. This paper will demonstrate the discrepancies between typically assumed analysis techniques for predicting column moment demands, and offer alternative approaches for both the design of new moment frame structures and the analysis of existing moment frame structures. An example ten story one-bay reinforced concrete moment frame (Figure 1) subjected to the Northridge Earthquake (Figure 2) will be analyzed. Comparisons will be made between the inelastic time history analysis and the suggested simplified techniques.

SYSTEM MODEL



Level	Beam M_p (ft-kips)
Roof	990
10th	990
9th	1260
8th	1260
7th	1635
6th	1635
5th	1880
4th	1880
3rd	1920
2nd	1920
grade beam	2520

Figure 1 Example frame

The seismic weight proportioned to the frame is 500 kips at each level. The plastic capacities of the beams were determined by using the Equivalent Static Force Procedure of the 1994 Uniform Building Code (ICBO, 1994), increased by 1.4 (the load factor), by 1.25 (strain hardening, etc.), and divided by 0.9 (the capacity reduction factor). The structure is assumed to exist in seismic Zone 4 with a site factor of 1.2. The columns are modeled with infinite yield capacities in order to identify all of the potentially over-stressed columns. This is considered practical from both an original design perspective and an existing frame analysis since in both cases we strive to prohibit the columns from reaching their full strength and therefore rotating freely in a plastic state. While column hinging at one end only does not lead to instability, most engineers would agree that it is prudent to prohibit full plasticity at all locations of the columns except at the base. An inelastic non-linear analysis was performed on the frame using the computer program DRAIN-2DX and the acceleration ground record shown in Figure 2. The response spectrum generated from this ground record is shown in Figure 3. The purpose of generating a response spectrum based upon this single earthquake record is simply to demonstrate the ability to predict inelastic dynamic effects using a relatively simple response spectrum approach, coupled with displacement analyses. Gravity loads on the frame beams are assumed negligible, which is consistent with most designs which intentionally limit the amount of tributary vertical loading to these elements.

When performing any analysis of this type, the engineer is required to make a number of modeling decisions such as predicting the element effective stiffnesses, moment-curvature relationships, actual material properties, strain hardening, damping, vertical distribution of forces, etc., as well as determining the capacity of all of the elements for the resulting demands. While these are all very important considerations, the purpose of this paper is not to dictate modeling or the interpretation of results, which is the responsibility of the evaluating engineer. The goal of the proposed procedure is to predict realistic column moment demands based upon the structural properties deemed appropriate by the responsible engineer.

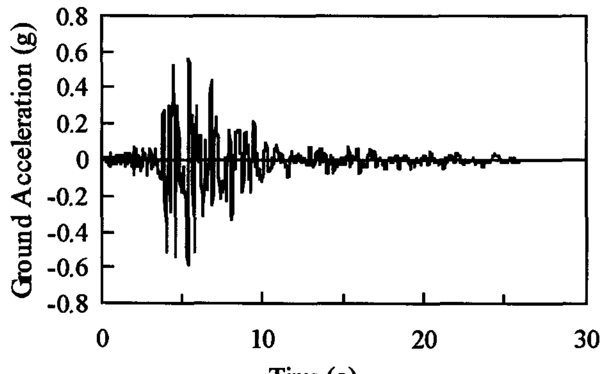


Figure 2 Ground acceleration record, Newhall - L.A. County Fire Station 90 degree component (Department of Conservation, 1994)

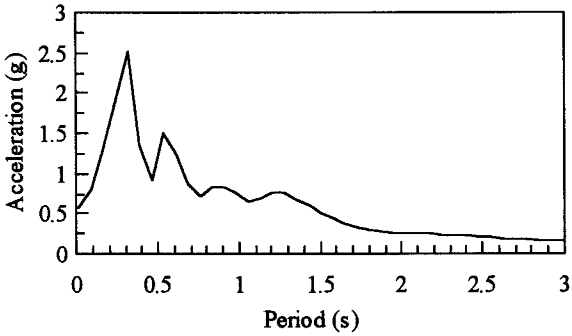


Figure 3 Response spectrum, Newhall 90 degree component, 5% damping

When the frame is subjected to the earthquake, there are three significant time steps that together envelope all of the maximum column moment response values. These steps occur at times 5.54, 8.49, and 10.47 seconds. The corresponding deflected shapes of the structure are shown in Figure 4.

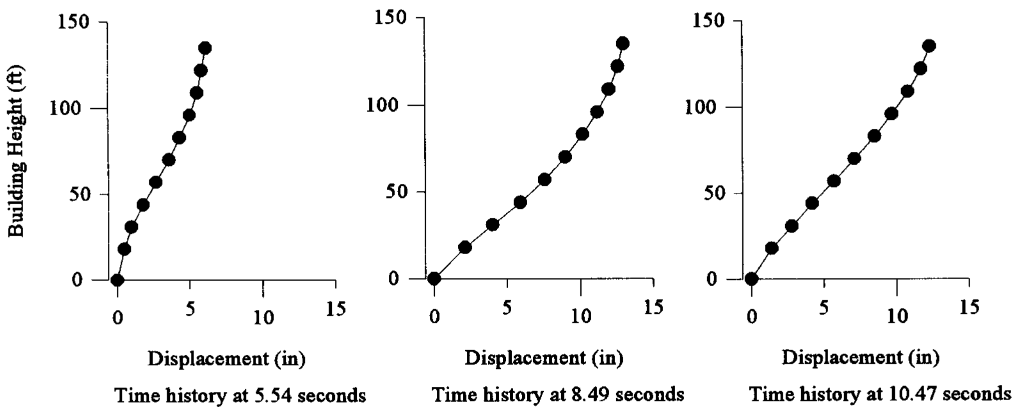


Figure 4 Deflected shapes at critical times

The column moment diagrams corresponding to these times are shown in Figure 5. The maximum values experienced at each level are shown at the corresponding time in the analysis.

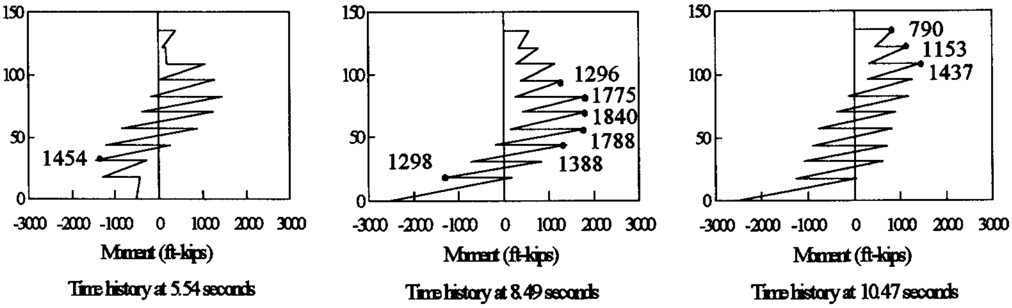


Figure 5 Column moments at critical times during the time history analysis

PREDICTING MAXIMUM COLUMN MOMENTS

To demonstrate the capability to predict the maximum column moments experienced during the time history analysis, the 6th level will be analyzed. This level corresponds to the largest moment experienced above the base, which is 1840 ft-kips at time 8.49 seconds. The shape at this time (Figure 6) is not a clear mode shape, but it definitely represents a significant first mode response. At the 6th level the beam is yielding (along with most of the other beams), and its plastic moment capacity is 1635 ft-kips. A typical "sub-assembly mechanism" analysis assuming mid-height inflection points and the beam plastic capacity would estimate that the maximum column moment at the joint face would be 703 ft-kips (1635 ft-kips divided by 2 and reduced to the joint face). This results in an error of 162%. Clearly, a more accurate method of determining the maximum column moments is required.

The critical issue that is implicitly ignored by building codes is that the elastic column (assuming that the column does remain elastic) will have moment demands independent of the moments that the beams deliver. The column will continue to displace and change the demand at an accelerated rate after the beam at that level has reached its plastic capacity. The best way demonstrate this is to place the "bare" column, unrestrained by any beams, in the same displaced shape as the structure. The moments generated become the underlying "basic" moment diagram to which the moments generated from beam plastic activity (the traditional sub-assembly mechanism moments) are added. The forces required to laterally displace the column only into the shape of the frame at time 8.49 seconds are shown in Figure 7, and the resulting column moments in Figure 8. The column properties used in this analysis were consistent with those used in the time history analysis.

When this approach is applied to the 6th level for example, the maximum demand reduced to the joint face is estimated as follows:

$$M_{max} = M(\text{basic column}) + M(\text{from plastic beam})$$

$$= 1137 \text{ ft-kips} + 703 \text{ ft-kips} = 1841 \text{ ft-kips},$$

which virtually identical to the time history moment of 1840 ft-kips at this time step (Figure 5).

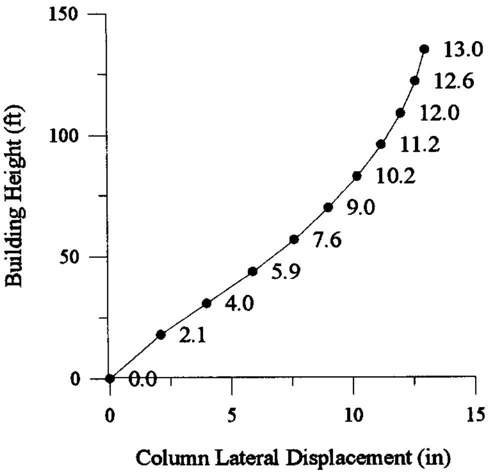


Figure 6 Floor lateral displacements at time 8.49 seconds

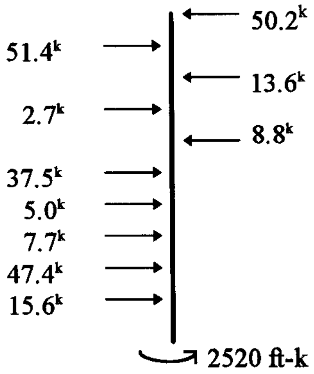


Figure 7 Forces required to generate column lateral displacements at time 8.49 seconds

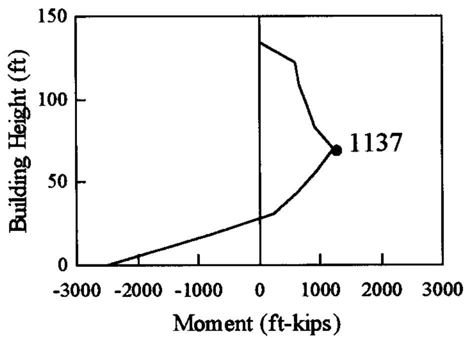


Figure 8 "Bare" column moment diagram at displacement of time 8.49 seconds

While the previous approach works well when the maximum displaced shapes of the structure are known, it does not lend itself well for use in design. A more design oriented approach is to perform incremental displacement analyses using the maximum displacement anticipated by an elastic modal analysis for each mode. It is well documented that inelastic and elastic maximum displacements will be similar for structures with fundamental periods longer than the period associated with the peak acceleration response (Englekirk, 1994), therefore they are also assumed to be the same in this analysis. The frame or structure is "pushed" out to the maximum absolute displacement at the corresponding level determined from the elastic modal analysis while accounting for all inelastic behavior. The shape that the frame or structure takes on will be a smoother version of the elastic mode shape, typically with greater displacements at levels other than the target displacement level than predicted by the elastic modal analysis.

THE INCREMENTAL DISPLACEMENT ANALYSES

The fundamental period of the structure is 1.96 seconds. An elastic modal analysis combined with the response spectrum of Figure 3 estimates the maximum first mode roof displacement to be 12.4 inches. The maximum first mode elastic and inelastic deflected shapes at this roof displacement are shown in Figure 9. The inelastic shape is generated by "pushing" the frame to a 12.4 inch roof displacement while accounting for all beam inelastic activity. Clearly the inelastic shape of the structure at the maximum first mode displacement is notably different than the elastic shape. The inelastic shape is "smoothed" due to the yielding of the beams and the fact that the column stiffness at the joint is the only stiffness for post-yielded displacements. This results in the inelastic column displacements generating higher column moments than would be predicted by the elastic column displacements, even for the same maximum nodal displacement (the roof in this case).

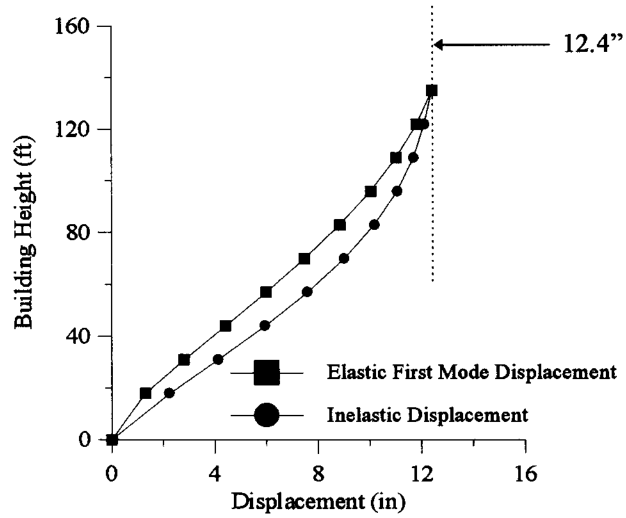
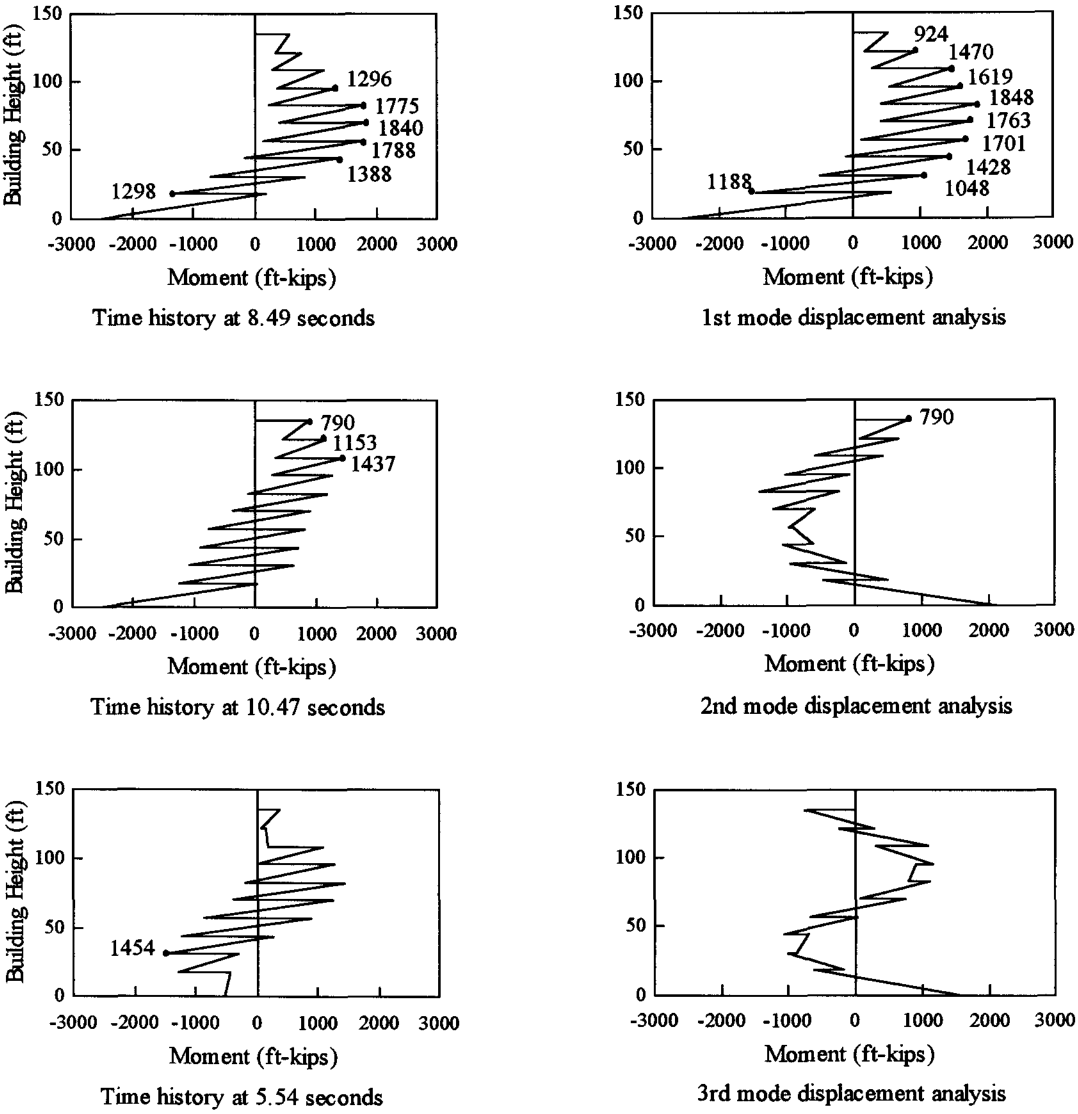


Figure 9 Elastic and inelastic deflected shapes at the target displacement of 12.4 inches

In addition to the first mode, incremental displacement analyses were performed for the second and third modes, which have corresponding periods of 0.60 seconds and 0.31 seconds respectively. These modes represent the three translational modes with periods that lie at or to the right of the peak acceleration response value, which typically are the modes that may produce noticeable non-linear effects. The critical column moment diagrams determined from the time history analysis from Figure 5 are re-plotted next to the envelope moment diagrams generated by the individual modal displacement analyses (Figure 10). Notice that there is dominant 1st, 2nd and 3rd mode response at times 8.49, 10.47 and 5.54 seconds, respectively. However, there is also significant first mode response in each of the critical time history plots. This helps explain why the 1st mode displacement analysis closely predicts most of the moment demands from the time history analysis.

It is also important to realize that the greatest column moments at the lowest levels of the structure may not occur concurrently with the maximum roof displacement during the first mode displacement analysis. The entire column moment diagram tends to "slide" in the direction of the displacement of the structure as beam yielding occurs, actually decreasing the demands at the lower level columns. For instance, in Figure 10 (b) the 1188 ft-kip moment shown in the 1st mode displacement analysis occurred at approximately the same instant that the first lower level beam yielded. At the end of the analysis the column moment at this same location was significantly less due to the shift in the moment diagram. This is particularly important to document in structures that demonstrate "cantilevered column" action under the elastic distribution of forces.



(a) From time history analysis

(b) Envelope moments from displacement analyses

Figure 10 Column moment demand comparison from the time history and displacement analyses

Observe the column moment envelope plots of Figure 11. This diagram shows the maximum column moment demands predicted by the UBC, the inelastic time history analysis, and the displacement analyses. The time history and displacement analyses agree very well at most levels. Further, notice how drastically the code estimated demand underestimates the actual demands, particularly at the middle levels of the structure.

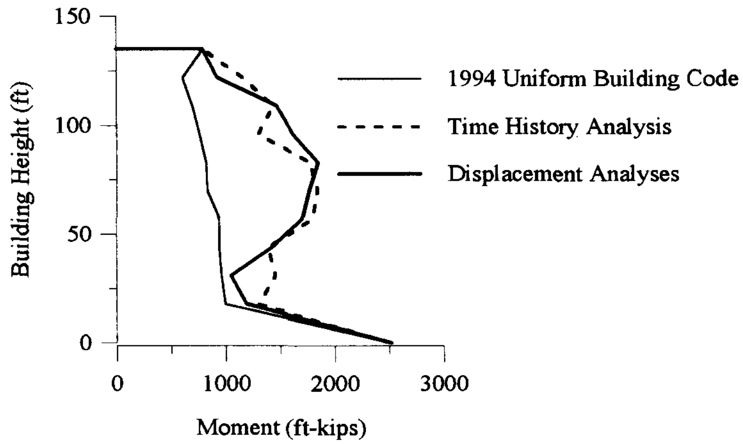


Figure 11 Column moment envelopes

Observe also that although excellent correlation exists between the time history analysis and the incremental displacement analyses, some demands are underestimated by the displacement analyses. A conservative solution to enveloping the maximum column demands would be to use a modal combination technique, such as the square-root-sum-squares approach. This would, for instance, better capture the 3rd level moment that is currently underestimated in Figure 11.

The January 17, 1995 Kobe Earthquake demonstrated the vulnerability of mid-rise structures to middle level collapse. This is often attributed to the apparent change in construction material type at these levels. While this is most likely a significant factor, the corresponding demands relative to capacities should also be verified before any conclusions are drawn. Based upon the analysis and behavior demonstrated here, there is reason to believe that the mid-level demand on these structures may have been at least that of the lowest levels, and the column capacities less.

STEEL FRAMES AND MULTI-BAY FRAMES

Steel frames are not exempt from the same type of behavior illustrated here (Bondy, 1995). The significant difference for steel frames is that the column moment of inertia is usually much less than that of the beams at a joint, whereas in concrete frame structures the opposite is true. In both cases the columns display a significant elastic response independent of the attached beams. The increased column moments in steel frame structures are typically more pronounced at the upper levels, whereas in concrete frames the mid-section as well as the upper levels experience this phenomenon.

The fundamental lesson to be remembered is that the stiffer the column is, the more load will be generated by placing it through deflected shapes. These demands must be viewed as additive to the loads delivered by the attached beams. Concrete columns are typically stiffer than steel columns, however the flexibility of steel structures usually requires larger displacements and a greater number of displaced shapes to generate the design column moment envelope.

Multi-bay frames will also exhibit the same type of behavior demonstrated here (Bondy and Allred, 1996). The anticipated displaced shapes of multi-bay frame structures are not significantly different than those constructed of single-bay frames, and the effects of incrementally displacing these structures into various shapes will be similar. Once again, the columns will continue increase their moment demands at an accelerated rate after the beams at the joint have yielded while the structure continues to displace.

NON-SEISMIC ELEMENTS

All elements of the structure that connect floors must be shown to be able to withstand these displacements while continuing to function in their capacity. For non-seismic columns this simply means that the same displacements experienced by the moment frames must be placed upon these columns. The columns must be able to demonstrate an ability to maintain vertical load carrying capacity under these displacements, either elastically or inelastically.

Once again, even if the columns do not have beams attached capable of transferring moments (as in precast structures), there will be significant moments and shears generated in these elements as they displace with the structure through various shapes.

CONCLUSIONS

The state-of-practice "capacity" design of flexible moment frames has been shown by inelastic non-linear dynamic analyses and possibly by actual earthquakes to significantly underestimate the moment demands on the columns. The philosophy of "capacity design" must be revised to include the significant effects of the elastic column deformations in addition to the plastic action of the attached beams. Estimates of the maximum column moment demands can be made for both concrete and steel structures using relatively simple analyses of the significant modes displaced incrementally to the displacements predicted by an elastic maximum credible response spectrum. The envelope moments found from the inelastic displacement analyses can be used to reasonably estimate the maximum anticipated moments during a maximum credible earthquake.

Additionally, the non-seismic elements of the structure should also be analyzed for their capacity to achieve the same maximum displacements while continuing to perform in their primary functions.

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