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Seismic Response of Eccentrically Braced Frames Designed for Canadian Conditions

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ABSTRACT

Eccentrically braced frame structures are designed for various seismic load levels using proposed Canadian design requirements. The response of these frames to several different real ground motion records, scaled to correspond to the load level used in design, is obtained. It was found that for several ground motions, forces and deformations developed in the upper storeys of the frames exceeded those anticipated in the design process. The result of this is that the link beam ductility capacity in the uppermost one or two storeys may be inadequate, and some yielding of columns in these floors could occur. In spite of the high forces developed, there was no yielding or buckling predicted in the beams outside of the links or in the braces.

INTRODUCTION

In eccentrically braced frames (EBF) at least one end of every brace is connected so as to isolate a beam segment called the link. The design is based on the principle that when overloaded, links yield either in shear or in bending and act as ductile members in the frame, while columns, braces and beam segments outside the links are expected to respond elastically. The energy dissipation is forced to occur in the links and non-ductile modes of failure such as column and brace buckling are inhibited. This system effectively satisfies two requirements: high stiffness at working load levels and high ductility under rare but severe overloads.

This paper presents a study of a number of EBF designed for different locations in Canada according to Canadian design requirements. The objective is to examine the inelastic dynamic response of building structures when subjected to severe ground motions and thereby evaluate the validity of the design procedures to achieve the desired behaviour.

The design of the structures was carried out according to the requirements of CSA-S16.1-M89 (CSA 1989) and NBCC (1990) together with subsequent revisions. Revisions related to NBCC (1990) are described in NBCC (1993), and those related to CSA (1989), which are largely based on AISC (1992), are described by Koboevic and Redwood (1994).

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DESIGN

Structures were designed for several Canadian seismic zones. Attention is principally directed at a series of eight storey structures, but a 16-storey structure was also examined. The EBF of the eight storey structures is shown in Fig. 1. This forms part of a system with moment resistant framing in the orthogonal direction: except for the braced frame the structure is identical to that described by Lu et al (1994). For the more severe seismic zones the design procedure is to select the trial design on the basis of the ductility requirements, and subsequently to verify the resistance at the ultimate limit state. The reverse procedure may be appropriate for zone of low seismic load.

The design procedure can be summarized as follows:

1. The shear force in the link is found and the minimum beam section to resist this force is selected.

2. The brace-to-link connections are assumed to be fully moment resisting. The forces arising from 1.5 times the link resistance is assumed to act in the link, and the beam is assumed to carry a minimum of 80% of the link end moment. The beam may have to be modified to provide adequate resistance.

3. The brace is next designed as a beam-column in one of the following ways:

- elastic design: careful tuning of the brace section must be carried out to find the minium brace section with adequate stiffness, relative to that of the beam, to transfer sufficient moment from the beam, and at the same time has adequate resistance. The flexural stiffness can be significantly affected by the presence of axial load, which should be taken into account (both in tension and in compression).
- inelastic design: it is assumed that beam outside of the link may yield locally, and hence compatibility need not be satisfied. In this case the brace section is chosen so that its flexural resistance added to that of the beam outside the link exceeds the link end moment. Flexural resistance must consider the effect of axial forces.

Iteration between steps #2 and #3 is necessary.

4. Columns are designed for the forces introduced by braces and beams, based on 1.25 times the forces corresponding to link shear yield.

5. The preliminary design sections chosen above are then checked for the ultimate and serviceability limit states under all load combinations, including wind and earthquake. If link beams are modified steps 2, 3 and 4 must be repeated.

Eight storey structures were design for seismic loads based on zonal velocity ratios, v, of 0.3, 0.2, 0.15, 0.1, and 0.05. In addition a 16-storey structure was designed for $v=0.3$. This structure had braced bay width of 10 m and link length of 1.0 m. Typical storey heights were 3.7 m, and the first storey was 4.5 m.

DYNAMIC ANALYSIS

Non-linear time history analysis for four scaled earthquake records was carried out using the ANSR-1 program (Mondkar and Powell, 1975). Links were modelled with a newly formulated shear-link element (Ricles and Popov 1987,1994) which efficiently models inelastic shear deformations, as well as appropriate strain hardening for both shear and flexure. This is believed to be a reliable tool to analyze EBFs with short, shear links. The other elements of the frame were modeled as standard beam-column elements. In order to evaluate the impact of P-Δ effects on structural behaviour, analyses both with and without these effects were carried out.

Recommendations of Ricles and Popov (1987, 1994) were followed regarding the damping and modelling of the links. Non-proportional damping was employed, with no viscous damping assigned to the links, while the remaining members of the frame were assigned Rayleigh damping based on 5% of critical. This type of damping is suggested to minimize excessive viscous damping forces from developing in the links.

Earthquake accelerations used in the analysis were scaled using a factor equal to the ratio of the peak velocity of the record to the design velocity, thus the a/v ratio of the original record was preserved. The 8-storey EBF designed for v=0.3 was analyzed for all four earthquake records, both with and without second-order effects. The response of 8-storey EBFs designed for other seismic zones was studied for the El Centro scaled record with geometric stiffness included; in addition the structure designed for v=0.1 was analyzed without P-Δ effects. The 16 storey EBF designed for v=0.3 was analyzed for the scaled Taft record with P-Δ effects included.

The results presented in the following pay particular attention to the location of inelastic activity, the response of the links expressed as maximum induced shear forces and maximum deformations, and maximum forces developed in the other members of the frame, with particularly attention to columns. The effects of including geometric stiffness is also addressed.

Structural behaviour of 8 storey frames

As anticipated by the design procedure, yielding took place mostly in the links. Only the links of the structure designed for v=0.05 remained elastic. With v=0.3 most inelastic activity happened in the upper top storey links. The number yielding simultaneously frequently exceeds one, reaching as many as seven for all records except El Centro for which the maximum number was four. The time-history of link yielding for the Taft record is shown in Fig. 2. Structures designed for zones with lower velocity ratios showed more inelastic activity in the links in the lower storeys, with a maximum of 6 links yielding at the same time.

The location of the link that attained the largest level of strain hardening as well as the maximum shear force expressed in terms of the nominal shear resistance of the link, V_r', vary with zone. The normalized maximum link shear forces are illustrated in Fig. 3 for v = 0.3. For all records greater magnitudes of strain hardening developed in upper floors (7 and 8), with maximum values varying with the particular record. For the El Centro record the maximum link shear force developed was $1.5V_r$', the maximum assumed in design, while for all other records the maximum reached a value of $1.9\ V_r$'.

The link deformation envelopes of the structure with $v=0.3$ are illustrated for three earthquake records in Fig. 4. The parameter γ is the inelastic shear deformation of the link. The links in the upper floors generally developed the greatest deformations. While the design limit of 0.09 rad was not exceeded for the El Centro earthquake, and was only exceeded in the top link for the Taft earthquake, the results for the top three storeys for the Shiogama record as well as for storeys 6 and 7 for Loma Prieta are several times the design limit, with maximum values of 0.32 and 0.31 rad respectively.

For other zones, analyzed for El Centro only, the normalized maximum link shear forces are shown in Fig. 5. For $v=0.2$ maximum shear forces of 1.4 and $1.5V_r$' were observed for the links at storeys 1 and 7 respectively. For $v=0.15$ the maximum link shear developed in the bottom storey and was equal to 1.45 V_r'. When $v=0.1$, only links at storeys 1, 5 and 7 yielded with the maximum shear force of 1.4 V_r' observed in storey 7. For these less severe zones the design limit for the link deformation was not exceeded. The maximum value of γ = 0.08 rad was observed in $v=0.2$ for the link at storey 7, as shown in Fig. 6.

Yielding of all other members was examined by tracing the time-history of the bending-axial force interaction. In all cases only elastic response was observed for beam segments outside the links and for braces. However, yielding was observed in columns during three out of four records when $v=0.3$, and to a much smaller extent in Zone $v=0.2$.

Columns sections were initially selected for ductility requirements on the assumption that the links at all storeys have yielded. The probability that the average link shear under this circumstance will be lower than the maximum design value of $1.5V_r$' is accounted for by assuming an average multiple of 1.25 of the link shear resistances when computing column loads. The dynamic analysis shows that for three of the four earthquake records, as many as seven links were yielding at the same time, and also, for several of the ground motions, links in the upper storeys developed forces as high as 1.9 V_r'.

Fig. 7 compares the maximum response axial force for the Taft record for $v=0.3$ with the axial force used in the preliminary design. These forces are almost identical. However, column bending moments were assumed zero in the preliminary design whereas response values of column moments reach as much as 38% of the pure bending resistance in the top storey, dropping to 10-20% in other storeys. Although these maximum moments were unlikely to occur simultaneously with the maximum axial forces they are sufficiently large to explain the yielding. In general, column sections satisfying ductility requirements had to be slightly increased in section to satisfy the ultimate limit state condition, and for this the moments due to column continuity were considered.

In other zones the maximum response column forces approached very closely the column design loads, particularly in upper storeys. The magnitude of end moments in the columns was under 20% of the pure bending resistance in all cases except in upper two storeys when $v=0.2$, and this was the only case in which yielding took place.

With the exception of one storey in one structure under one ground motion ($v=0.3$ with the Loma Prieta record), the maximum inter-storey drifts were well below Code limits. This may partially explain why P-Δ effects did not affect the response of the structure significantly. The effect was much less than is suggested by their impact on design forces.

Summary of structural behaviour of 16-storey frame

Sixteen storey EBF designed for Zone v=0.3 was analyzed for the scaled Taft record with geometric stiffness accounted for. Maximum link shears reached $1.75V_r$' in the top storey, and dropped in the mid-height of the frame, such that at one level the link remained elastic. In the bottom two storeys, maximum shears were about $1.5V_r$', a much higher value than in the eight-storey frames at this level. Link deformations were consistent with these shear magnitudes, exceeding the design limit in the upper two storeys by a significant margin. Again, neither braces nor beams outside of the links yielded, but some column yielding occurred in the uppermost column tier. The number of yield excursions was similar to those in the eight-storey frames. It was found that the preliminary design for ductility produced a flexible structure which required changes to some braces and to the lower column tiers in order to satisfy the inter-storey plastic drift requirements (a mass increase of 15% resulted). This added capacity probably avoided yield of these columns.

CONCLUSIONS

The following conclusions may be drawn from this study:

(i) In the design process any beam shear over-strength in the link region must be minimized. At the same time the beam section must be selected so that it can carry, outside the link, the probable forces arising from the strain-hardened link. In severe seismic zones the ductility design should precede the strength design, whereas in lower risk zones the sequence should be reversed.

(ii) Response magnitudes of link shear forces and deformations, as well as column forces, are considerably higher than anticipated in the uppermost storeys. While this study is limited to one frame configuration, and only four ground motions are considered, similar observations have been made for concentrically braced frames. This raises the question of the adequacy of the specified value of upper storey lateral loading.

(iii) In spite of high link shears, the braces and beams outside the links remain elastic in all cases.

(iv) Even if link forces in the upper storeys are constrained not to exceed $1.5V_r$', the upper columns must be assumed to be loaded by forces greater than $1.25V_r$'. It is also desirable that moments be estimated for inclusion in the column selection in the ductility design stage.

ACKNOWLEDGEMENTS

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REFERENCES

AISC. 1992. Seismic provisions for structural steel buildings. American Institute of Steel Construction, Chicago, IL.

Canadian Standards Association (CSA). 1989. Limit states design of steel structures, CAN/CSA-S16.1-M89. Rexdale, Ontario.Koboevic, S. and Redwood, R.G. 1994. Design and seismic response of eccentrically braced frames. Proc. 1994 Ann. Cong. Vol. II, CSCE, Winnipeg, Manitoba. pp. 309-318.

Mondkar, D.P. and Powell, G.H. 1975. ANSR-I General purpose computer program for analysis of non-linear structural response. Report No. UCB/EERC-75/37. University of California, Berkeley.

NBCC. 1990. National Building Code of Canada, 1990. Associate Committee on the National Building Code, National Research Council of Canada, Ottawa.

NBCC. 1993. National Building Code of Canada - Second series of proposed changes, August 1993, National Research Council of Canada, Ottawa.

Redwood, R.G., Feng Lu, Bouchard, G., and Paultre, P. 1991. Seismic response of concentrically braced steel frames. Can. J. Civ. Eng., 18:6. pp. 1062-1077. December.

Ricles, J.M. and Popov, E.P. 1987. Dynamic analysis of seismically resistant eccentrically braced frames. Report No. UCB/EERC-78/07. University of California, Berkeley, Calif.

Ricles, J.M. and Popov, E.P. 1994. Inelastic link element for EBF seismic analysis. ASCE. Journal of Structural Engineering, Vol 120, No. 2, Feb.,pp.441-463.

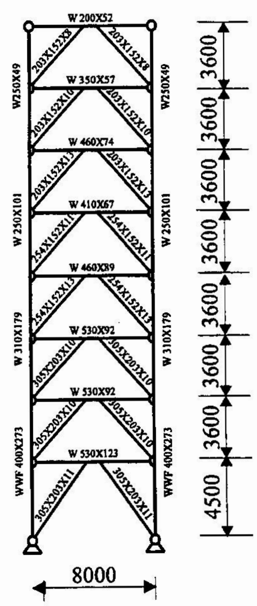

Fig. 1. Elevation of the frame and member sizes for v = 0.3

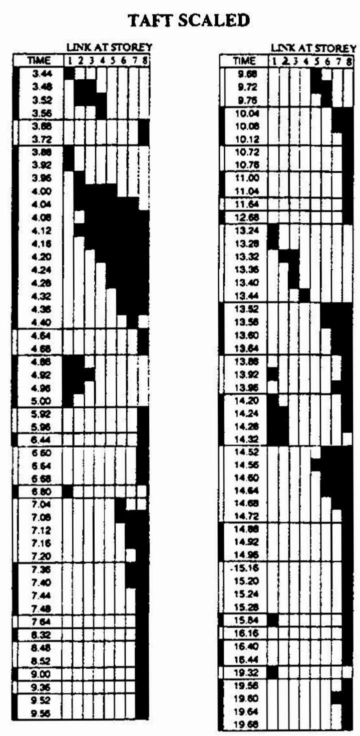

Fig. 2. Time-history of link yielding : Taft record with v=0.3

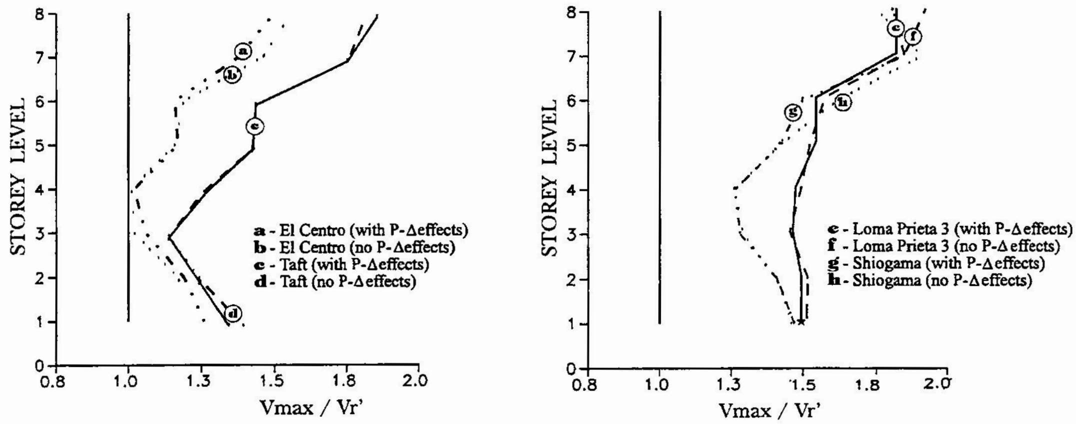

Fig. 3. Normalized maximum link shear forces for v = 0.3

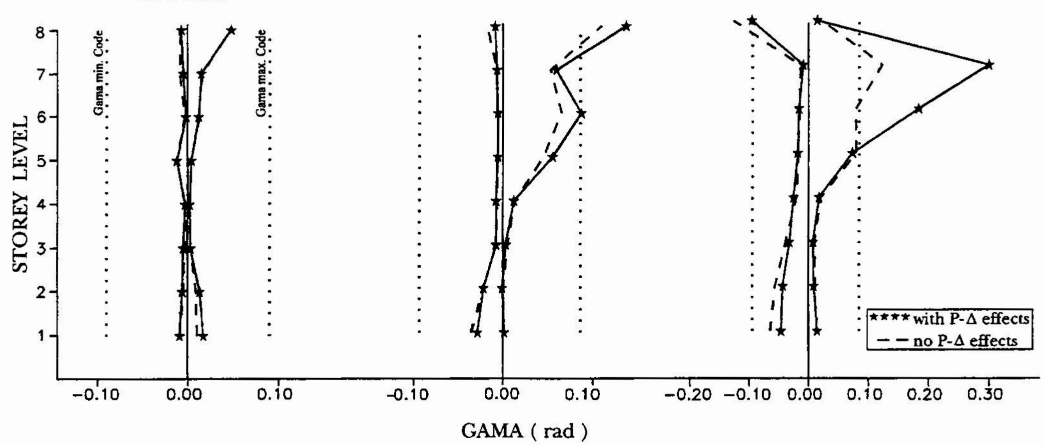

Fig. 4. Link deformation envelopes for v = 0.3

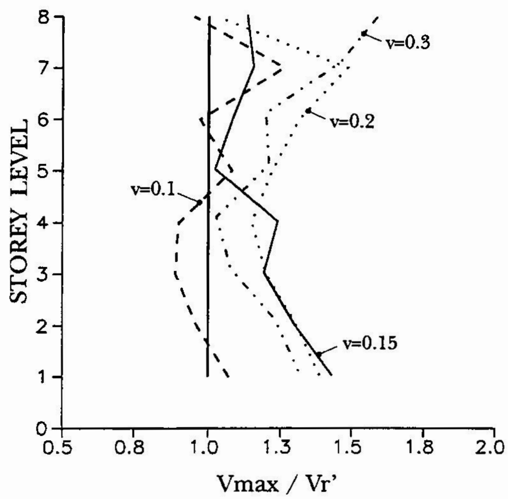

Fig. 5. Normalizes maximum shear forces for El Centro record

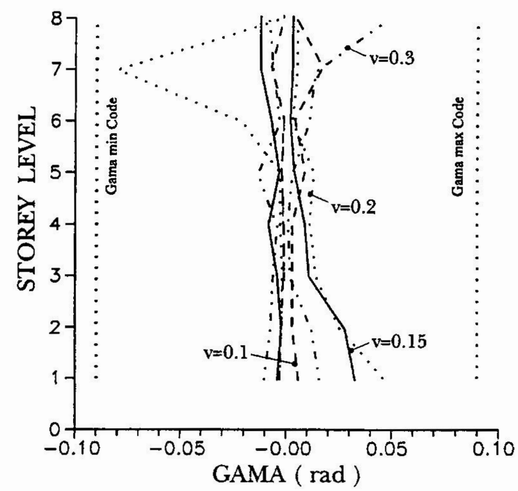

Fig. 6. Link deformation envelopes for El Centro record

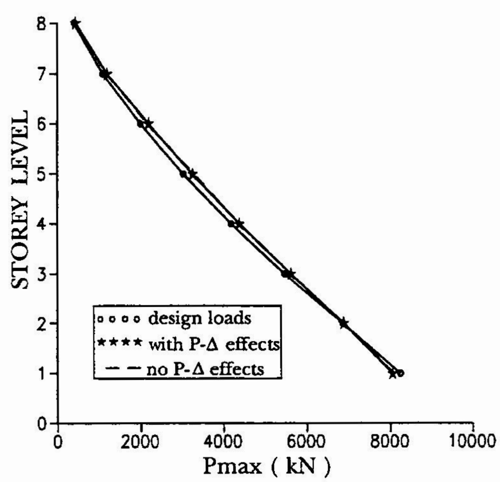

Fig. 7. Design and maximum response axial forces in columns: Taft with v = 0.3