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Paper No. 1100. (quote when citing this article)
Eleventh World Conference on Earthquake Engineering
ISBN: 0 08 042822 3

INFLUENCE OF IN-PLANE FLEXIBILITY OF ROOF DIAPHRAGM ON THE SEISMIC RESPONSE OF SINGLE-STOREY BRACED FRAMES

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ABSTRACT

The paper presents the results of nonlinear dynamic analyses performed on single-storey steel buildings with flexible roof diaphragm and diagonal steel bracing. The influence of the hysteretic behaviour of the vertical bracing members and the effects of varying the design force level were examined in the study. Buildings with tension-only bracing members and those with tension-compression yielding members exhibited similar behaviour. In both cases, the shear forces and bending moments that developed in the roof diaphragm were larger than those predicted by a static analysis. The difference between static and dynamic forces was more important when lower seismic loads were used in the design. The ductility demand on the bracing members and the storey drift were also higher in buildings designed for lower seismic loads.

KEY WORDS

Single-storey building, roof diaphragm, steel deck, tension-only steel bracing.

INTRODUCTION

For single-storey steel buildings, it is common practice to use the steel roof deck as a diaphragm to transfer lateral loads to the vertical bracing elements (CSSBI 1991, Luttrell 1991). The roof diaphragm is formed by interconnecting the steel deck units (Fig. 1) and fastening them to the supporting steel framework. Upon application of wind or earthquake horizontal loads, the diaphragm deforms in its own plane, both in shear and bending.

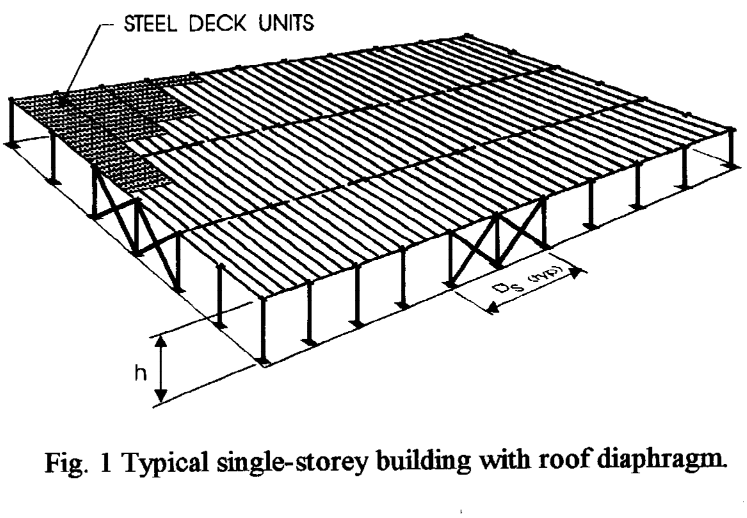

Fig. 1 Typical single-storey building with roof diaphragm.

Although in-plane flexibility of floor and roof diaphragms is known to influence significantly the response of building structures subjected to dynamic loading (Button *et al.* 1984, Jain and Jennings 1985, Dolce *et al.* 1994), an equivalent static load approach proposed in current buildings codes (e.g. NRCC 1995) is still generally used for the seismic design of single-storey buildings.

Figure 2 shows the applied load and the resulting forces in the roof diaphragm

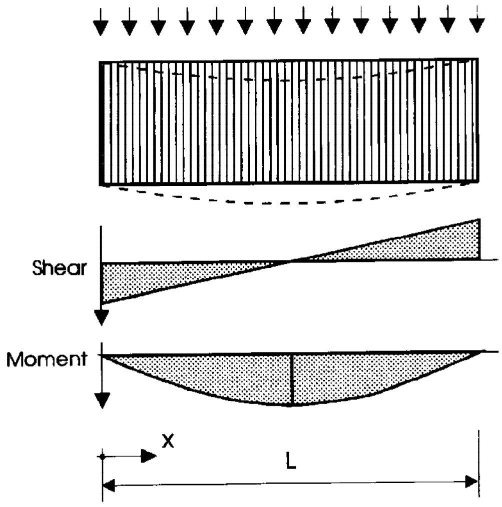

Fig. 2 Static forces in roof diaphragm.

predicted by this static approach in the case of a uniform rectangular building braced along its exterior walls. This method does not account for the dynamic response of roof diaphragms under earthquake loading and Ewing (1993) questioned its appropriateness for these structures.

Tremblay and Stiemer (1996) recently completed a study on the seismic behaviour of single-storey steel buildings with flexible roof diaphragm. Their investigation included nonlinear dynamic analysis of 36 symmetrical rectangular buildings of different sizes subjected to site specific ensembles of earthquake records. The results showed that higher in-plane forces and deformations develop in the roof diaphragm when compared to those predicted by a static analysis. The study also revealed that the computed fundamental period of the structures was significantly longer than the value specified in codes for design purposes. Thus, all structures exhibited some level of overstrength due to their inherent flexibility.

In their study, Tremblay and Stiemer assumed ductile vertical bracing elements. Consequently, the buildings were designed for the lowest seismic load level permitted by codes and the vertical bracing was modelled using a tension-compression yielding element, which exhibited an elasto-plastic hysteretic behaviour.

In practice, however, various vertical bracing systems can be used for single-storey steel buildings, among which the tension-only bracing system with slender steel diagonal bracing members is very common (Fig. 1). This system exhibits severely pinched hysteresis under inelastic cyclic loading and is generally assigned higher seismic design loads than systems with higher energy dissipation capability (SEAOC 1990). The objective of this study was to investigate the effects of using a tension-only bracing system for single-storey buildings with flexible roof diaphragm and to propose guidelines for the seismic design of these structures.

ANALYSIS

Three typical buildings were designed according to current specifications and their seismic response was examined through a series of nonlinear dynamic analyses performed with representative earthquake accelerograms. All buildings included a tension-only bracing system. In order to grade the performance of these structures, the analyses were also carried out for comparison purposes with a bracing system exhibiting an elasto-plastic hysteretic behaviour. This second bracing system was similar to the one considered by Tremblay and Stiemer in their study. Furthermore, the analyses for both hysteretic bracing models were repeated with the amplitude of the accelerograms scaled up by a factor of 2,0. These analyses simulated the effects of using lower seismic loads in design and thus permitted to assess the appropriateness of current design load levels for these buildings.

Buildings

The buildings were rectangular in shape and the vertical bracing was symmetrically distributed along the perimeter, as shown in Fig. 1. The roof weight was assumed uniform and equal to 1,0 kPa. Three building sizes were considered: 15 m x 30 m x 4,4 m (small size), 30 m x 60 m x 6,6 m (medium size) and 60 m x 120 m x 9,0 m (large size). The spacing of the columns along the exterior walls was equal to 7,5 m. The diagonal members of the X bracing's were made of slender steel plates. The small size building had one X bracing per wall whereas two braced bays per wall were used for the larger structures.

For all buildings, the roof deck was made of intermediate rib steel deck units, 38 mm deep by 900 mm wide. The deck was assumed to be welded to the framework and metal screws were used for the sidelap connections. The spacing of the roof steel joists was equal to 1500 mm for all three structures.

Design

The structures were designed according to the NBCC 1995 (NRCC 1995) and the S16.1-94 Standard for the design of steel structures in Canada (CSA 1994). The equivalent static load approach was used in the design, but the modifications proposed by Tremblay and Stiemer were also considered. In the NBCC, the design factored base shear V_f is given by:

$$V_f = v \cdot S \cdot I \cdot F \cdot W \cdot \left(\frac{U}{R}\right) \tag{1}$$

where v is the design ground velocity for the site, S is the seismic response factor, I is the importance factor, F is the foundation factor, W is the seismic weight of the structure, U is a calibration factor (U = 0.6) and R is the force modification factor. All structures were assumed to be located in Vancouver, B.C., where the value of v is equal to 0,21m/s. The value of S varies with the fundamental period of the structure and the seismicity at the site. It is given in Fig. 3 for the site under consideration ($S = S_a/v$). The fundamental period, T, to be used in the calculation of S is equal to:

$$T = 0.09 \cdot h \cdot (D_S)^{0.5} \tag{2}$$

The parameters h and D_S are defined in Fig. 1. If the computed fundamental period of the structure exceeds the value obtained using (2), the NBCC allows to reduce S accordingly by up to 20%. For all three buildings, equation (2) yielded periods shorter than 0,25 s and S was therefore taken equal to 2,4 (= 3,0 x 80%), assuming that the computed periods would permit this reduced value. The factors I and F were set equal to 1.0 (structures of normal importance erected on stiff soil). The weight of the structure included the roof dead weight, plus 25% of the roof snow load (0,37 kPa). The R factor mainly accounts for the capacity of the system to absorb and dissipate energy during earthquakes. For tension-only bracing, it is equal to 2,0.

The vertical bracing was sized for the factored base shear V_f given by (1). Shear forces and bending moments in the diaphragm were computed assuming a static uniform load as shown in Fig. 2. The horizontal load that causes yielding of the selected bracing members, multiplied by 1,10 to account for possible impact loads induced when slack in stretched braces is rapidly taken up during an earthquake, was considered in the calculations. As proposed by Tremblay and Stiemer to account for dynamic effects, however, the shear in the diaphragm was assumed equal to the end shear over the entire length of the building and the static bending moments were amplified by 2,3.

The computed fundamental periods for the three structures were: 0,96 s, 1,24 s and 1,32 s, for the small, medium and large size buildings, respectively. The S factors computed for these periods respectively are: 1,53, 1,35 and 1,31, which is less than the assumed value of 2,4. The ratio between the design S value and the S values obtained with the actual periods can be seen as an overstrength ratio of the structures, resulting from their high flexibility. Such overstrength ranged between 1,57 and 1,83 for the three structures.

Earthquake records

The buildings were subjected to the ensemble of historical ground motions described in Table 1.

Table 1. Description of selected ground motions.

Event	Station	Component	PHA (g)[1]	PHV (m/s)[1]
1971 San Fernando, Ca	2500 Wilshire Blvd, L.A.	N61W	0.101	0.193
1971 San Fernando, CA	Hollywood Storage, L.A.	N90E	0.211	0.211
1973 East coast of Honshu, Japan	Kushiro Central Wharf	N00E	0.205	0.275
1940 Imperial Valley, Ca	El Centro, Ca	S00E	0.348	0.334

[1] PHA: Peak horizontal acceleration; PHV: peak horizontal velocity.

Except as indicated below, all accelerograms were scaled to the design ground velocity for the site (0,21 m/s). Figure 3 compares the acceleration response spectra, for 5% damping, corresponding to the mean plus one standard deviation of the ensemble of normalised ground motions to the NBCC design spectrum for the site.

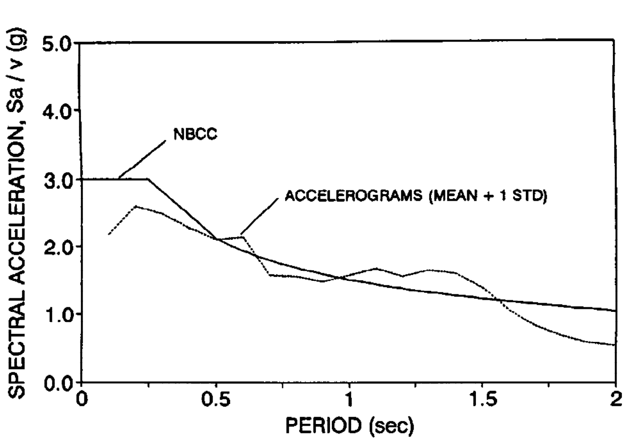

Fig. 3 Absolute response spectra for selected ground motions and NBCC, 5% damping of the

Analyses

The nonlinear time-history dynamic analyses were performed with the DRAIN-2DX computer program (Allahabadi and Powell 1988). A Newmark constant acceleration integration scheme, with a time step equal to 0,001 s, was adopted for the calculations.

The response of the structures was computed in the direction parallel to the short walls. Figure 4a illustrates the two-dimensional analytical model developed for the structures. As shown, the symmetry of the problem permitted to consider only half of the lateral load resisting system of the buildings. The roof diaphragm was modelled with 10 beam elements which were assigned the bending and shear properties of the diaphragm. The mass per unit of length of the roof, m, was lumped at the nodes. The rotation of the uppermost node in the model was prevented to simulate the zero slope condition at mid-length of the building.

The bracing members for the tension-only bracing system were modelled with a tension yielding and compression buckling element with zero buckling strength (Fig. 4b). For the ductile bracing system, tension-compression yielding elements with an elasto-plastic behaviour were used (Fig. 4c). In the latter case, the cross-sectional area of the braces was reduced by 50% to maintain the same total lateral stiffness and resistance for both vertical bracing systems. Rayleigh type damping based on 5% critical in the first two modes of the model was specified for all structures.

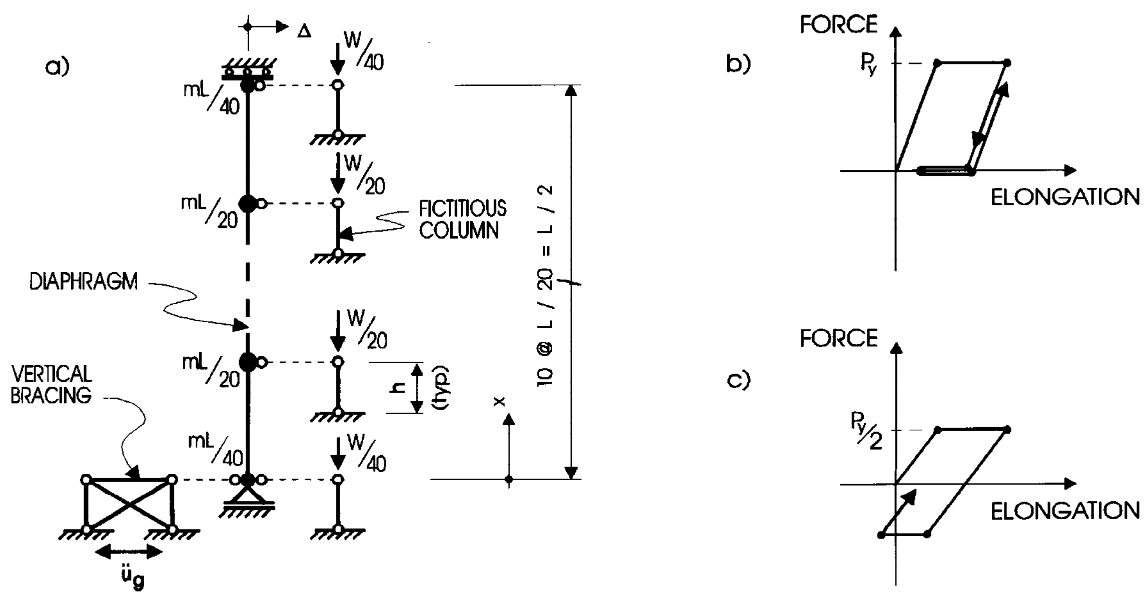

Fig. 4 Analytical model: a) 2D model of one half of the building; b) tension yielding - compression buckling brace behaviour; c) elasto-plastic tension-compression yielding brace behaviour.

The effects of gravity loads on the deformed structure, the P-Δ effects, were accounted for in the analysis by adding a fictitious column at each nodal point along the length of the diaphragm (Fig. 4a). The tributary roof gravity load of each node was applied to the corresponding column, and the lateral displacement of the upper end of each column was constrained to be identical to the lateral deformation of the roof diaphragm at that location.

Load levels

In the NBCC, the maximum allowable value for the R factor is 4,0. Its use is restricted to the most ductile systems, exhibiting stable inelastic response and high energy dissipation capacity. To examine the possibility of using such a high value of R for tension-only bracing systems, the time step analyses were repeated with the accelerograms normalised for the site conditions, as described above, and scaled up further by a factor of 2,0. So doing, the effects of varying the load level was isolated as the dynamic properties of the structures remained unchanged. This also permitted to relate the results of this study to those obtained by Tremblay and Stiemer who considered the R factor equal to 4,0 in the design of their buildings.

RESULTS

Five response parameters were examined in the study: the total storey drift at mid-span of the buildings, the ductility demand in the bracing members, the maximum diaphragm deflection, the maximum bending moment in the diaphragm, and the distribution of the shear forces in the diaphragm.

Figures 5 to 8 present the results for the first four parameters. In all cases, the mean plus one standard deviation of the results obtained for the ensemble of ground motions is presented. The results are given for each bracing system: tension-only and tension-compression yielding with an elasto-plastic response. The different buildings are designated by a letter giving their size (S, M or L for the small, medium and large size, respectively), followed by a number (2 or 4) which indicates the R factor considered for the seismic design loads. In Fig. 7 and 8, the maximum deflection and bending moment in the diaphragm are normalised with respect to the values obtained from a static analysis (Fig. 2) under the uniform load causing the yielding of the vertical bracing members.

For buildings designed with R equal to 2,0, both types of bracing system yielded very similar results. The storey drift and the ductility demand are slightly lower for the tension-compression system whereas roof deflections and bending moments are nearly the same for the two systems.

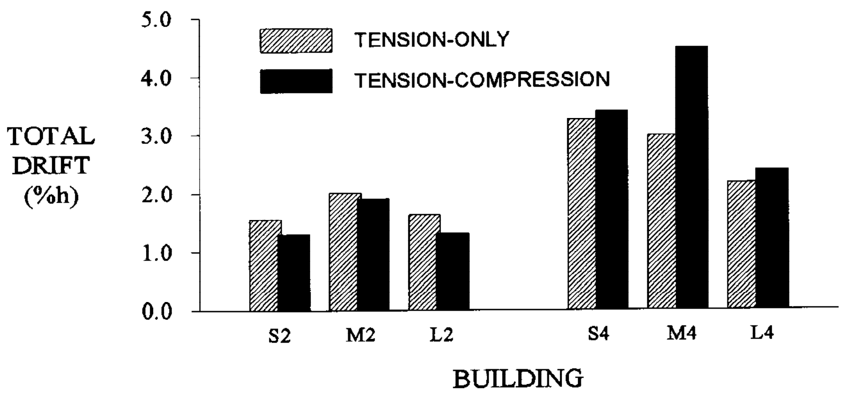

Fig. 5 Total storey drift at mid-length of building.

At that load level, the four response parameters do not vary significantly with the size of the buildings and the response of the tension-only bracing system appears to be satisfactory in all cases. The total storey drift ranged between 1,5 and 2,0% of the building height, which is within the limit prescribed in the NBCC (2%). The computed ductility demand in the bracing members varied between 2,7 and 2,9. This amount of inelastic axial deformation can be easily accommodated with common structural steels, for which strain hardening takes place at ductility's higher than approximately 10 and fracture typically occurs at strains exceeding 100 times the strain at yield.

As observed by previous researchers, in-plane roof deflection and bending moments were amplified with respect to the value obtained assuming a static behaviour. In both cases, the computed amplification varies between 1,4 and 1,5.

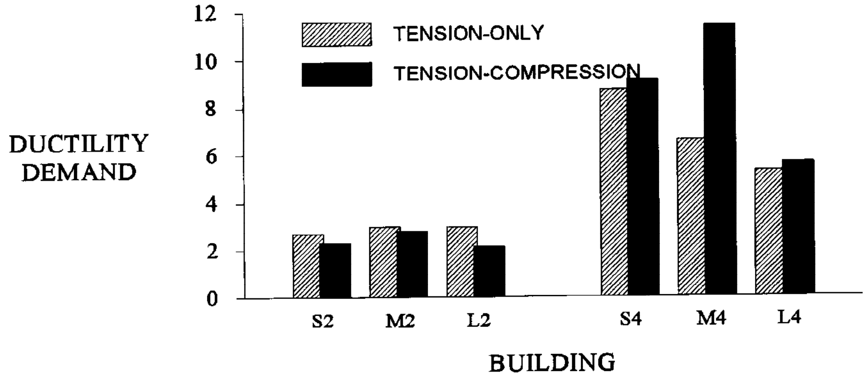

Fig. 6 Ductility demand in the vertical bracing.

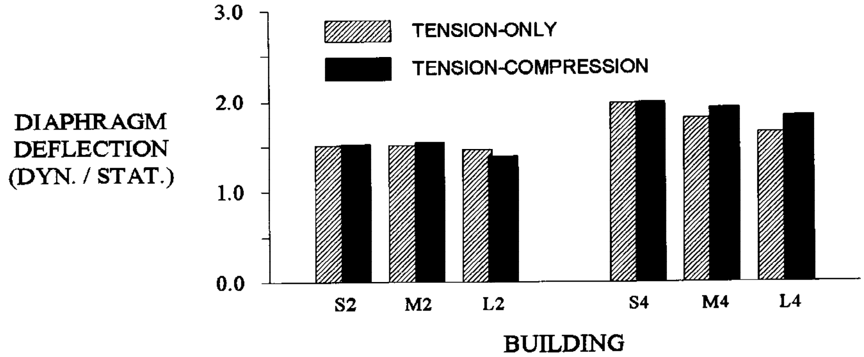

Fig. 7 in-plane deformation of roof diaphragm.

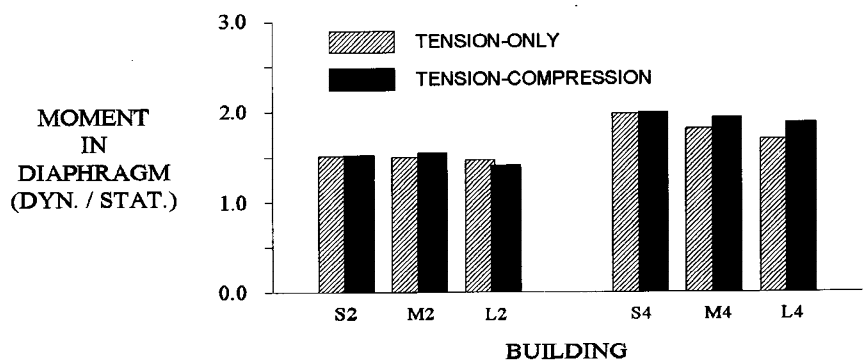

Fig. 8 Bending moment acting in roof diaphragm.

As expected, the structures designed for lower seismic loads (R = 4,0) exhibited larger storey drifts and higher ductility demand. The dynamic amplification of roof deformations and bending moments also increased when reducing the magnitude of the design loads.

Significant differences were anticipated between the two bracing systems at that lower design load level since more numerous and extensive inelastic excursions took place in the vertical bracing. Surprisingly, the two bracing systems performed very similarly in spite of their different hysteretic behaviour. In all cases, the buildings with tension-only bracing even exhibited a lower response than the more desirable tension-compression system. The more remarkable difference observed for building M4 is due to the very intense response of that building with the tension-compression system under the 1973 East coast of Honshu, Japan earthquake. For the three other ground motions, the results for both bracing systems were comparable.

The relative good performance of tension-only braced frames can be attributed to the isolation mechanism that develops in that system during strong earthquakes. Once the braces have stretched after the first few large acceleration pulses, the vertical bracing exhibits nearly zero lateral stiffness around the undeformed position and the roof becomes uncoupled from the ground motion. This mechanism attenuated significantly the dynamic response of the buildings for the remaining part of the accelerogram.

The storey drifts computed for the buildings with tension-only bracing designed with R equal to 4,0 varied between 2,2 and 3,3% of the building height, which is beyond the NBCC limit. Conversely, the ductility demand obtained for these buildings (between 5,3 and 8,7) is still well within the safe range for structural steels. The large deformations observed in this study may not be acceptable for some buildings. However, from a structural point of view, building sway in typical single-storey buildings is not as critical as in higher structures and the possibility of relaxing the allowable drift under earthquake loading should be examined in view of the satisfactory ductility levels attained. It is also believed that the drifts predicted by the simple model used in this study are conservative. For instance, the partial rotational restraint of the connections between the steel members or at the base of the columns, the axial stiffness of intermediate roof joists and beams in the calculation of the stiffness of the diaphragm, and the nonlinear behaviour of diaphragm subjected to shear forces are factors not included in the model which could reduce the deflections.

Contrary to the situation with R = 2,0, the ductility demand and the storey drift for tension-only bracing designed with R = 4,0 generally diminished as the size of the buildings was increased. With R = 4,0, the influence of the overstrength level was likely more pronounced due to the higher degree of inelastic action, and larger buildings experienced lower response. This suggests that increasing the stiffness of a structure to meet stringent drift limitations would likely result in a higher inelastic demand on its bracing system.

The computed amplification of the roof deflection and bending moment varied between 1,6 and 2,0 for the tension-only bracing and R equal to 4,0. Tremblay and Stiemer suggested an amplification factor of 2,3 for these two response parameters. This value represented an upper bound of the numerous results obtained in their study for buildings with elasto-plastic bracing designed with R = 4,0. This value should also be adopted for buildings with tension-only bracing, considering the similar response of the two bracing systems.

The statistics of the shear forces acting along the diaphragm have been computed for each bracing system and design load level combination, that is, for ensembles of results obtained for the three building sizes and the four earthquake ground motions (12 analyses). The mean plus one standard deviation of these forces is given in Fig. 9. In this figure, the shear forces have been normalised to the shear force acting at the end of the diaphragm. In all cases the distribution of shear deviates significantly from the one predicted by a static analysis. This difference is more accentuated for buildings designed with an R factor of 4,0 but almost the same distribution was obtained for the two bracing systems. These results suggest that the shear capacity of the entire roof diaphragm exceeds the shear forces acting at its ends.

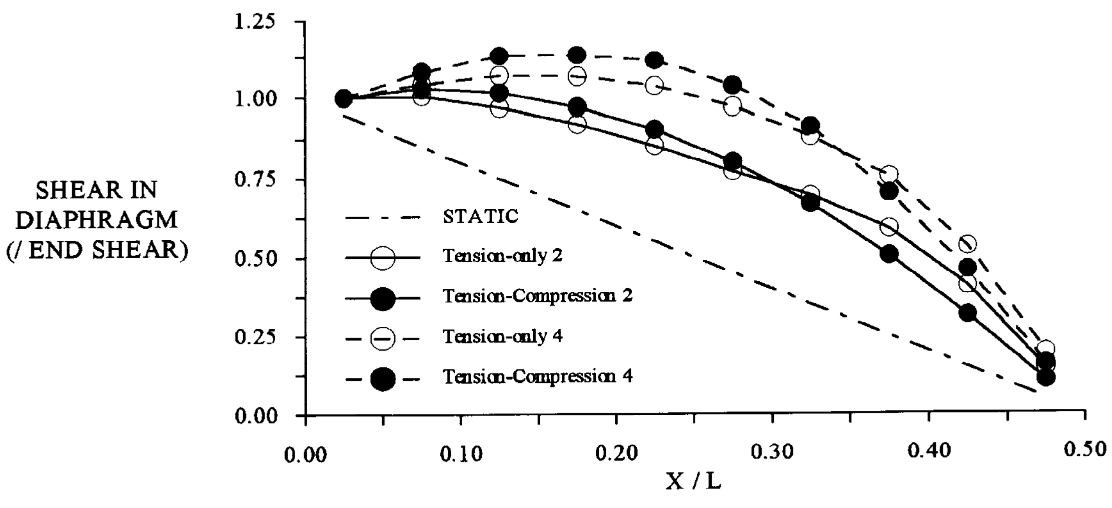

Fig. 9 Distribution of shear forces in roof diaphragm.

CONCLUSION

The seismic performance of single-storey steel buildings with flexible roof diaphragms was examined through a series of nonlinear time step dynamic analyses carried out on three typical buildings. Two bracing systems and two design load levels were considered.

The study clearly showed that tension-only bracing performs very similarly to a tension-compression yielding system exhibiting the same resistance and stiffness. This similitude was observed for structures designed with R equal to 2,0 and 4,0, which indicates that the need to specify higher design seismic loads for tension-only bracing systems to compensate for their pinched hysteretic behaviour may not be appropriate and should be re-examined. However, the buildings designed with the R factor of 4,0 experienced excessive storey drifts which could preclude the use of such low loads for their design. Further research is needed to investigate this aspect.

In order to avoid structural damage to the roof diaphragm, the design forces acting in the diaphragm should be based on the actual resistance of the vertical bracing system. Furthermore, in-plane diaphragm forces and deflections as obtained from a static analysis should be modified to account for the dynamic response of the structure under earthquake ground motions. The shear force should be assumed constant and equal to the end shear over the entire span of the diaphragm, and the maximum bending moment and the deflection should be multiplied by a factor of 2,3 for buildings designed with R equal to 4,0.

Further research is also needed to investigate the possibility of using the actual period of the buildings when computing the design seismic loads. In this process, guidelines should be developed to obtain realistic values of the periods including the stiffening effects of typical non structural elements.

ACKNOWLEDGEMENTS

The financial support of the Natural Sciences and Engineering Research Council of Canada and the Fonds pour la formation des chercheurs et l'aide à la recherche of Quebec, Canada, is gratefully acknowledged. The help received in this research from Roger Bélair, undergraduate student at the Department of Civil Engineering of École Polytechnique of Montréal is appreciated.

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