Seismic Response of Single-storey Steel Buildings

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SEISMIC RESPONSE OF SINGLE-STOREY STEEL BUILDINGS

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ABSTRACT

Nonlinear time step dynamic analyses have been performed on 24 rectangular single-storey steel framed buildings including a metal roof deck diaphragm and steel bracing bays along their exterior walls. The structures were designed according to current Canadian codes and were subjected to site specific ensembles of historical earthquake accelerograms.

The analyses indicated that larger in-plane deformations and bending moments developed in the diaphragm compared to the values expected from the equivalent lateral force procedure commonly used in design. The distribution of the shear forces in the diaphragm was also found to deviate significantly from the linear distribution assumed in design. In addition, the ductility demand in the bracing bents exceeded the amount predicted by nonlinear analyses performed on equivalent single-degree of freedom systems. Based on these results, preliminary design guidelines have been proposed for predicting the deformations, moments and shear forces in roof diaphragm as well as for confining inelastic action in the vertical bracing elements.

INTRODUCTION

The lateral load resisting system (LLRS) of single-storey steel framed buildings generally includes an horizontal metal roof deck diaphragm which transfers horizontal loads to vertical bracing elements anchored to the foundations (Fig. 1). These vertical bracing elements can be of many types among which diagonal steel bracing, moment resisting frames and masonry or cast-in-place concrete walls are the most popular. This type of construction has been widely used over the years for light and heavy industrial facilities as well as for commercial, institutional or recreational buildings.

Traditionally, the seismic design of low-rise steel frames has been performed using the well-known equivalent lateral force procedure proposed in North American building codes, wherein the dynamic effects of earthquakes on the structure are approximated by applying statically an horizontal force at the center of mass of the building. Because the area of the walls exposed to winds is relatively small for single-storey structures, it is not

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uncommon to have such prescribed seismic loads to govern the design of the LLRS (Tremblay 1993; CSSBI 1991). This is especially the case when buildings are located in seismically active regions or when a heavy roofing construction system is used.

Despite the very large number of constructions of this type and the significance of seismic induced effects on such buildings, very little attention has been devoted so far to examine the behavior of such structures to earthquake strong ground motions. This paper reports on an analytical study that has been performed within an ongoing research project on the seismic performance of low-rise steel structures with flexible roof diaphragms. The main objective of the study was to assess the validity of the equivalent force procedure used for these buildings.

Nonlinear time step dynamic analyses have been performed on 24 rectangular flat-roofed single-storey buildings subjected to historical earthquake accelerograms. All structures were located in Canada and were designed accordingly to most recent Canadian codes. The study was limited to normal buildings, that is, excluding post-disaster buildings, resting on leveled foundations on rock or firm soil. Buildings had vertical steel bracing along the four perimeter walls (Fig. 1) and were symmetrical in plan with a uniform mass, stiffness and strength. Thus, in-plane torsional effects were omitted.

Three parameters were varied: the size of the building, the building location (seismicity) and the weight of the roofing system. Among the parameters studied were the ductility demand on the vertical bracing elements, the horizontal deformations, the magnitude and distribution of the shear forces and bending moments acting in the roof diaphragm. Although the study addressed the case of steel structures with a metal roof deck diaphragm, most of the findings also apply to low-rise buildings with an in-plane roof bracing system.

CURRENT DESIGN PROCEDURE AND DYNAMIC CHARACTERISTICS

Prescribed seismic loads:

The magnitude of the prescribed seismic loads mainly depends upon the expected intensity of ground shaking at the site, the local soil conditions and the characteristics of the buildings, namely their fundamental period, damping and reacting weight. The design procedure also assumes that part of the seismic input energy is to be absorbed and dissipated through inelastic action during the design base earthquake. Therefore, the design loads also depend on the expected performance of the chosen vertical bracing elements upon successive reversals of inelastic loading.

In the National Building of Canada (NBCC) 1990 (NRCC 1990), both the expected peak horizontal acceleration (PHA) and velocity (PHV) are used for characterizing the design ground motion at a site. A dimensionless format has been adopted for these intensity-related parameters: the acceleration ratio a, which is equal to the ratio of PHA to the acceleration of the gravity, and the velocity ratio v, which is equal to PHV divided by 3.28 ft/sec (1 m/sec).

Zones delimited by contour levels on acceleration and velocity design maps were assigned an intermediate constant value of a and v (zonal a and v). These seismic zones are designated by the parameters Z_a and Z_v, which take a value ranging from 0 to 6, a higher value corresponding to a higher zonal acceleration or velocity ratio. The difference between Z_a and Z_v gives some indication on the size, location, frequency content and duration of the seismic events likely to affect the most at the site under consideration (Heidebretch *et al.* 1983). Regions with Z_a smaller than Z_v suggests that the site is mostly influenced by long duration and large earthquakes at distance, with energy concentrated in the moderate period range, whereas Z_a exceeding Z_v indicates that smaller, near-by, short duration and high frequency events dominate.

Once the design ground motion parameters are determined, an equivalent elastic base shear can be computed:

$$V_e = v \cdot S \cdot F \cdot I \cdot W \tag{1}$$

In this expression, the seismic response factor S corresponds to a smoothed design acceleration response spectrum for 5% damped Multi-Degree-of-Freedom systems subjected to earthquake accelerograms scaled to a PHV = 3.28 ft/sec (1.0 m/sec), that is, v = 1. Therefore, S is multiplied by v in equation [1] to obtain the amplification of the motion at a particular site. S varies as a function of the fundamental period of the structure T. For the long-period range (T > 0.5 sec), the S factor is given by equation [2]. For shorter periods, S also depends upon the value of Z_a relative to Z_v to account for the fact that the response of stiffer structures is mainly related to PHA rather than PHV. As shown on Fig. 3, three different values of S are specified for T smaller than or equal to 0.5 sec.

$$S = \frac{1.5}{\sqrt{T}} \tag{2}$$

The fundamental period to be used in the determination of S may be obtained from established methods of mechanics but such period must not exceed 1.2 times a prescribed limit in order to safeguard against unrealistic long periods that can be so-obtained. For braced frames, this limit is given by equation [3], where h is the total height of the frame and D_s is the width of the bracing bents in a direction parallel to the earthquake excitation.

$$T = \frac{0.05 \cdot h}{\sqrt{D_s}} \tag{3}$$

A foundation factor F and an importance factor I are included in equation [1] to account for the effects of local site conditions and to increase the safety level for post-disaster buildings. W is the so-called seismic or reacting weight of the building, which includes the dead weight of the structure plus 25% of the roof snow load, if any.

A limit state design format is used in the NBCC 1990 and since the design ground motion parameters correspond to major and accidental events, a load factor of 1.0 is specified for the seismic loads. V_e reflects the base shear a building structure would experience if it was to remain elastic during the design base earthquake. Therefore, the factored base shear V_f to be used in design is obtained by dividing V_e by a force modification factor R which accounts for the capacity of the LLRS to resist earthquakes in the inelastic range. A calibration factor U, equal to 0.6, is however introduced in the calculation which maintains the same level of protection than the one provided by previous editions of the code:

$$V_f = \left(\frac{V_e}{R}\right) \cdot U \tag{4}$$

Design of the LLRS:

Upon lateral loading, single-storey steel frames with a flexible roof diaphragm behave like a deep plate girder on flexible supports loaded in the plan of its web. This analogy is illustrated in Fig. 2 for the simple building configuration examined in this study. The web is made of the interconnected deck units attached to the roof framework whereas the supporting steel members located along the sides of the roof perpendicular to the seismic loads form its flanges. The vertical bracings elements act as the supports for the girder.

This plate girder analogy is well accepted in design (CSSBI 1991; SDI 1991). The web is considered to resist only shear forces and the shear stiffness of the girder ($G'b$), as well as its shear strength, depends upon the profile and the connections of the steel deck panels. The bending stiffness (EI) of the girder is provided by the flanges which are assumed to resist only flexural forces. Vertical bracings, having a stiffness K_B, are designed to transfer the loads from the roof down to the foundations.

When the reacting weight is uniformly distributed along the span of the diaphragm, the strength and the stiffness of the girder and its supports are checked upon application of a uniformly distributed static load equal to the factored base shear divided by the span of the roof diaphragm L. The computed drift at mid-span has, however, to be multiplied by the factor R to account for the inelastic response. The NBCC limitation for the drift is equal to 2% of the building height for non post-disaster buildings.

Capacity design approach:

Although no formal capacity design approach is explicitly specified in codes for low-rise steel frames including metal roof deck diaphragms, it is desirable to prevent inelastic action to occur in the diaphragm upon strong ground shaking. Such approach is supported by the fact that the efficiency and the reliability of corrugated steel diaphragms sustaining several cycles of inelastic loading is questionable and because inspecting and repairing structural damages in a roof diaphragm is likely to be much more difficult and costly than in vertical bracings.

Avoiding inelastic response in diaphragms can be achieved by designing the vertical bracing elements to merely sustain the factored base shear and, thereafter, providing the diaphragm with sufficient strength to sustain forces corresponding to the actual capacity of the vertical bracings.

Dynamic characteristics of the structure:

For dynamic loading, the studied LLRS exhibits an infinite number of degrees of freedom. The periods and the mode shapes depend on the relative importance of the aforementioned stiffnesses, the geometry of the roof and the distribution of the mass. The first five mode shapes are shown on Fig. 2 for the case investigated herein, for which the mass of the roof per unit length, m, is constant throughout the span of the diaphragm. In such case, the mode shapes are a combination of trigonometric and hyperbolic sine and cosine functions.

Upon ground shaking, motion is introduced at the foundation level and inertia forces develop at the roof level which must be resisted by the girder and its supports. The distribution of these forces matches the mode shapes, which suggested that higher shear forces, bending moments and drift could develop in the diaphragm than those predicted by the aforementioned static design procedure.

STUDIED BUILDING STRUCTURES

Sites:

Two sites were selected for each of the three main categories of seismic regions: $Z_a = Z_v$, $Z_a > Z_v$ and $Z_a < Z_v$. These are listed in Table 1. The sites with medium and low a/v ratio are located along the Western Coast of Canada whereas the sites falling in the $Z_a > Z_v$ category, Ottawa and Quebec, are in Eastern Canada. In each three categories, the intensity of the ground shaking and the roof snow load were different between the two sites.

Site	v	a/v	Z_a	Z_v	Roof snow load (psf)	Number of records
Victoria, B.C.	0.26	1.08	5	5	20.9	21
Vancouver, B.C.	0.21	1.00	4	4	30.9	22
Ottawa, Ont.	0.098	2.04	4	2	45.1	33
Québec, Que.	0.14	1.36	4	3	65.6	18
Prince Rupert, B.C.	0.21	0.52	3	4	66.8	15
Whitehorse, Yukon	0.17	0.52	2	4	30.5	15

Table 1 Design data and number of accelerograms considered (1 kPa = 20.9 psf).

Buildings:

For each site, two different building sizes were considered: small buildings, measuring approximately 50' x 100' x 18' (15m x 30m x 5.4m), and large buildings, measuring approximately 200' x 400' x 30' (60m x 120m x 9m). For each building size, two roofing systems were considered: a light system with the membrane being on top of the insulation and a heavier system with protected membrane and ballast. The total dead load was 21 psf (1.0 kPa) and 46 psf (2.21 kPa), respectively, for each of these systems. Thus, a total of 24 buildings were designed.

All structures included an inverted-V chevron bracing bent (Fig. 1) on each exterior wall. The width D_s of the bracing bents was 24.6' (7.5m) and 49.2' (15m) for the small and large buildings, respectively. The roof framing system included a roof deck supported on open web steel joists, the latter ones being supported on girders running parallel to the long walls. For all buildings, intermediate rib 38 mm deep by 914 mm wide deck units conforming to CSSBI 101-84 (CSSBI 1984) Grade A, Fy = 33 ksi (230 MPa), were used. The thickness of the steel and the spacing of the joists varied depending upon the applied gravity and seismic loads.

Design:

The seismic base shear was determined with the procedure described earlier with the value of v as given in Table 1. The fundamental period was taken equal to 1.2 times the value given by equation [3], that is, 0.21 and 0.25 sec for the small and large buildings, respectively. Therefore, the prescribed loads varied significantly depending on the relative values of Z_a and Z_v. The foundation factor F and the importance factor I in [1] were set equal to 1.0. All buildings were designed with a R factor equal to 4.0, as this value was to lead to the most significant amount of inelastic deformation.

The LLRS was designed accordingly to the aforementioned capacity design approach. The actual capacity of the bracing bent was assumed to be equal to 1.5 times the force produced by the design factored load. The expected inelastic drift, that is, R times the computed drift at mid-span of the diaphragm, was kept within the prescribed limit.

The design charts published by the Steel Deck Institute (SDI 1991) were used for assessing the shear strength and stiffness (G') of the deck. Arc spot welding (16 mm welds) was assumed for fastening the deck to the framework. For the sidelap (stitch) connections, #10 screws were specified for 22 ga. (0.76 mm thick) material whereas spot welds were assumed for 20 ga. (0.91 mm) and thicker material. In case either the strength or the stiffness had to be increased, the following changes, in order, were made to: spacing of the stitch fasteners (sidelap), spacing of the frame fasteners, spacing of the joists and thickness of the deck. In the large buildings, the characteristics of the steel deck (fasteners, steel thickness) were varied along the length in order to make a better use of the material. A more detailed description of the structures can be found in Tremblay (1993).

Structural characteristics:

According to the classification generally accepted in practice (SDI), the roof diaphragms of the small buildings were of the flexible category with G' varying between 11.1 and 13.8 K/in (1.95 and 2.41 kN/mm). The roof deck in the large buildings generally fell in the semi-flexible and semi-rigid categories as G' varied between 20.0 and 131 K/in (3.51 and 22.9 kN/mm). The flexibility of the diaphragms can also be visualized in terms of the relative contribution of the diaphragm deformation to the total drift at mid-span, as computed under the prescribed static seismic load. That contribution varied between 83% and 91% and between 51% to 70% when the load was applied in the direction parallel and perpendicular to the short walls, respectively.

This flexibility can also be observed by comparing the fundamental periods of the buildings as computed considering and neglecting the diaphragm deformations (Fig. 4). In the short direction (loads parallel to the long walls), the ratio between the two periods is about 1.5. That ratio ranges between 2.0 and 3.0 in the other direction. It is worth nothing that these computed periods were significantly longer than the ones used in design.

ANALYSIS

The response of the building structures to earthquake ground motions was obtained analytically using the computer program DRAIN-2DX developed by Allahabadi and Powell (1988). That program can perform structural dynamic analyses of systems with material and geometric nonlinearities. A Newmark constant acceleration integration scheme, with a time step of 0.001 sec, was used throughout the study.

Each building was subjected to an ensemble of site-specific earthquake accelerograms. In addition, the performance of the structures was investigated successively in each of the two principal directions: parallel and perpendicular to the short walls. Therefore, 48 series of analyses were carried out. Thereafter, statistics of the desired response parameters were computed for examination. The mean plus one standard deviation are presented herein.

Earthquake records:

The earthquake records were chosen from an ensemble of 45 historical rock site accelerograms selected by Naumoski *et al.* (1988) for structural analysis. For a given site, the decision to consider a record was based on the likelihood of occurrence of such ground motion at the site, as established from a study of the seismicity of the site. The number of selected accelerograms for each of the sites is given in Table 1. A total of 1 000 nonlinear dynamic analyses were performed in the study.

The records were scaled to match the design parameters v unless the a/v ratio of the record exceeded the one at the site. In such case, the accelerogram was scaled to the peak acceleration.

Analytical model:

It was assumed that both bracing bents in the direction parallel to the shaking were struck at the same time by the same ground motion. Therefore, making use of the symmetry of the structures, only half of the LLRS of the buildings, in the direction perpendicular to the ground motion, needed to be included in the analytical model. So doing, only the contribution of the symmetrical modes (1st, 3rd, etc.) was included in the response of the structures. The model shown in Fig. 5 was employed for all buildings, for both directions of loading. Only the span of the diaphragm, the reacting weight and the structural properties of the components were changed from one case to another.

As shown in Fig. 5a, the roof diaphragm was modeled with 10 beam elements for which the bending and shear stiffnesses of the diaphragm were specified. The rotation of the node at mid-length of the building (uppermost node in the model) was prevented to simulate the zero slope condition at that location imposed by the symmetry of the problem. A truss element was used for the bracing bents. One end of this element was attached to the ground (fixed support) whereas the second was connected to the diaphragm. For the sake of simplicity, an elastic-perfectly plastic storey shear-storey drift relationship was adopted for the vertical bracing (Fig. 5b). The yield load was set equal to the force acting in the bracing bents upon application of the prescribed factored base shear.

As shown, the roof mass and weight were lumped at the nodes along the diaphragm. Gravity load effects were introduced by adding vertical truss elements (fictitious columns) along the length of the structure. The length of these elements was set equal to the building height. At the top, the horizontal deformation of these elements was constrained to be equal to the one of the corresponding nodes of the roof diaphragm. The base of the vertical members was attached to the ground (fixed support). Mass and stiffness proportional (Rayleigh) damping was included, with 5% of the critical damping considered in the first and third modes of the structures.

RESULTS

Typical behavior:

The typical seismic response of the studied buildings is illustrated in Fig. 6 wherein a portion of the time history of the drift of a particular building (no. LSH2) subjected to ground motion no. IAV13 is shown. The computed drift is given at two locations along the span of the diaphragm as well as at the bracing bent. For that building, the period in the first and third modes was 1.22 and 0.41 sec, respectively.

Despite the fact that most of the energy of the accelerogram IAV13 was concentrated in the period range between 0.15 to 0.6 sec, the figure clearly shows that the structure mainly responded in its first mode, as indicated by the fact that the drift at quarter- and mid-span are almost perfectly in phase. Such behavior was observed in all other buildings, which indicates that the fundamental period of the structure remains a critical parameter for characterizing this type of structures. The figure also reveals that the inelastic action only took place in the bracing bents and that the diaphragm oscillated about the deformed position of the vertical bracings.

Peak ductility:

Fig. 7 presents the peak ductility demand, which corresponds to the peak deformation divided by the deformation at yield, on the vertical bracing. As expected, larger values were observed in the short period range and for buildings located in the regions with $Z_a < Z_v$. As shown, very high values were indeed calculated (up to 85) when these two conditions were met simultaneously. The computed COV for this parameter varied between 0.5 and 1.0, indicating a rather large scatter in the results.

In Fig. 8, the computed peak ductility for each building is compared to that of a single degree of freedom (SDOF) system whose period was the same than the one of the building. The SDOF systems also had the same strength and damping ratio than the corresponding buildings and were subjected to the same ensemble of accelerograms. As shown, the ratio of the computed ductility to that predicted by the analysis of the SDOF systems varies significantly from one structure to another. In all cases but one, that ratio exceeds unity and values up to 5 were obtained, regardless of the period and the seismic region.

Peak horizontal deformations:

Despite very significant peak ductilities were observed, the peak storey drift at mid-span of the buildings remained within the applicable limit. As shown in Fig. 9, the computed drifts were found to generally increase with the period and larger drifts were experienced by the structures located in the regions with $Z_a < Z_v$.

Fig. 10 presents the ratio of the computed peak in-plane deformations of the diaphragms under the ground motions to the deformations obtained by statically applying the prescribed factored seismic loads. As shown, the dynamically induced deformations exceeded the static ones in all buildings, the amplification ranging between 1.5 and 2.3. For that parameter, the scatter was very low: the maximum value of the COV was 0.4 with most of the values being between 0.1 and 0.2.

In-plane forces in roof diaphragms:

Fig. 11 indicates that similar dynamic amplification was observed for the maximum bending moment acting at mid-span of the diaphragms. The scatter for that parameter was also very similar to the one observed for the diaphragm deformations. Partial examination of the computed envelopes of bending moment seems to indicate that multiplying the static bending moment diagram by the amplification factor given in Fig. 11 results in a conservative estimate of the moment along the span of the diaphragm. This observation still needs to be validated by further analysis.

As expected, the shear force that developed at the end of the diaphragm reached the yield strength assigned to the bracing bents in all buildings. More significant, however, is the fact that the computed envelopes of shear remain nearly constant over almost the entire span of the diaphragm, rather than decreasing linearly towards the mid-span of the structure, as predicted by the static analysis. Such behavior is illustrated in Fig. 12 for a typical building. On this figure, it can also be seen that the computed shear forces near the bracings slightly exceed the capacity of the bracing bent. This apparent anomaly is due to the discretization of the roof mass in the modeling and because damping forces add up to member forces upon dynamic loading.

CONCLUSIONS AND RECOMMENDATIONS

This study revealed that considering the actual flexibility of the roof diaphragm in low-rise steel framed buildings has a significant influence on their seismic performance. At once, such in-plane flexibility contributes in increasing significantly the periods of the structures, which is not accounted for in the code equation for the fundamental period. This resulted in conservative design strength levels, especially for buildings located in regions with $Z_a > Z_v$ and $Z_a = Z_v$.

The response of all studied buildings remained stable under all ground motions considered in the analyses and the peak building drift at mid-span of the roof diaphragm was less than 2% of the building height in all cases. However, the ductility demand for these buildings exceeded significantly, up to 5 times, the value predicted by nonlinear dynamic analyses performed on equivalent SDOF systems having the same period and strength level. Moreover, the ratio of these two values varied significantly within the ensemble of buildings.

Due to the dynamic nature of the response, in-plane deformations of the roof diaphragm were amplified by a factor varying between 1.5 and 2.3 as compared to the values obtained from a static analysis. Bending moments in the roof diaphragm were also amplified by a similar amount. In-plane shear forces of magnitude equal to the strength of the vertical bracing were observed over almost the entire span of the roof diaphragms, which invalidate the concept of shear forces decreasing linearly from the supports to zero at mid-span.

In order to achieve a better performance of single-storey frames with flexible diaphragms under strong ground shaking, the following recommendations are tentatively proposed to be used in conjunction whith the equivalent lateral load seismic design procedure.

The actual flexibility of the diaphragm should be included in the calculations of the fundamental period for obtaining more realistic estimates of the base shear. The high ductility demand generally observed on the vertical bracing elements however suggests that seismic loads higher that those currently prescribed should be used in order to prevent premature failure of these elements. Unfortunately, data obtained in this study are not sufficient for suggesting more appropriate values.

Inelastic action should be confined to the vertical bracing elements. Therefore, the design shear force for the roof diaphragm should be based on the actual capacity of the vertical bracing elements. Such shear force should be used over the entire span of the diaphragm due to the observed distribution of the shears.

The computation of the horizontal deformations should be consistent with the expected behavior of the LLRS, that is, amplifying only the drift of the bracing bents to account for the inelastic action. The in-plane deformations of the roof diaphragm, as obtained from a static analysis, should however be multiplied by a factor of 2.3 to account for the dynamic nature of the response. Similarly, a dynamic amplification factor of the same magnitude should also be applied to the in-plane bending moment acting in the roof diaphragm computed with the static loading.

In order to obtain more definitive design guidelines, further research is needed to assess the influence of many parameters not included in this study. For instance, the effects of the design strength level, local soil conditions and of the load-deformation relationships of the vertical bracing elements should be investigated. Torsional effects in unsymmetrical buildings or as induced by non-uniform mass or stiffness distribution should also be studied.

APPENDIX - NOTATION

a	Acceleration ratio
b	Depth of the roof diaphragm (parallel to seismic loading)
D_s	Width of the vertical bracing (parallel to seismic loading)
E	Modulus of elasticity
F	Foundation factor
G'	Effective shear stiffness of the roof diaphragm
h	Building height
I	Bending stiffness of the roof diaphragm; Importance factor
L	Building length (perpendicular to seismic loading)
m	Roof mass per unit length
R	Force modification factor
S	Seismic response factor
T	Period of the structure
U	Calibration factor
$\ddot{u}_g$	Seismic induced ground acceleration
v	Velocity ratio
V	Shear in roof diaphragm
V_e	Equivalent elastic base shear
V_f	Factored base shear
W	Total roof seismic weight
x	Coordinate along the span of the roof diaphragm
Z_a	Zonal acceleration ratio
Z_v	Zonal velocity ratio
Δ	Horizontal drift

APPENDIX - REFERENCES

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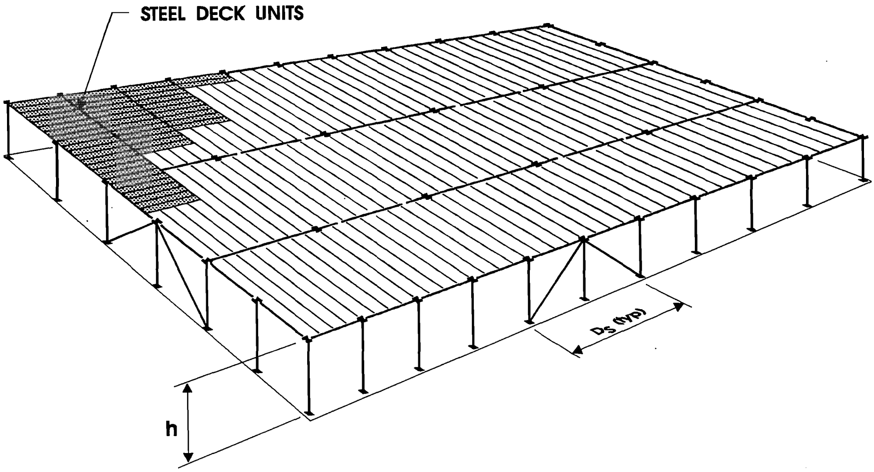

Fig. 1 Typical single-storey steel frame building with a roof diaphragm.

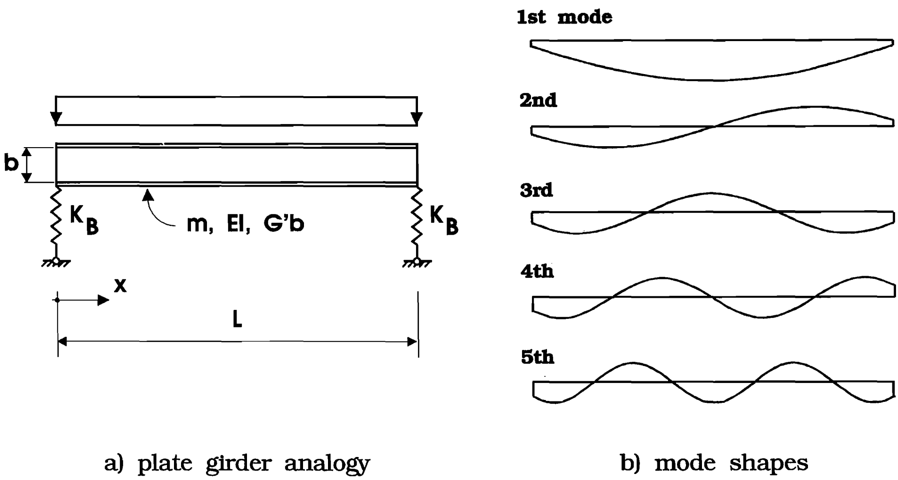

Fig. 2 Plate girder analogy and dynamic mode shapes.

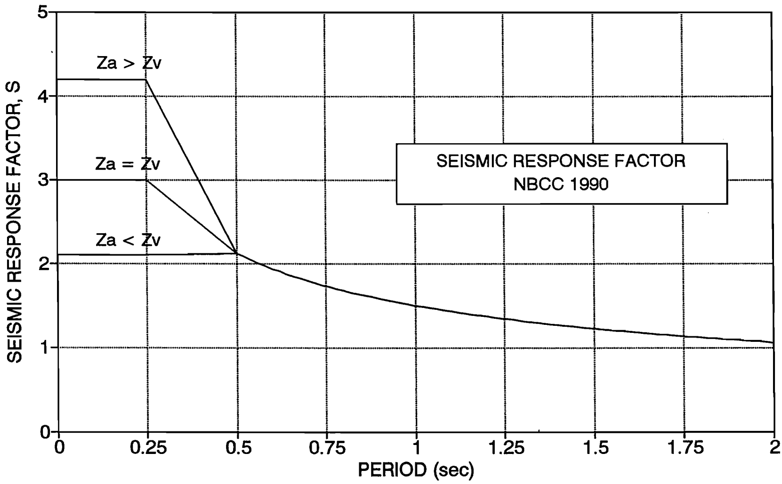

Fig. 3 1990 NBCC Seismic response factor.

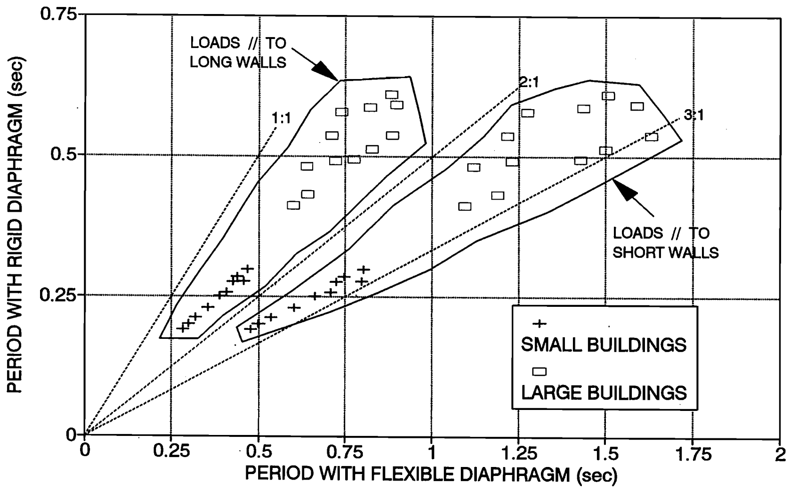

Fig. 4 Computed fundamental period of the studied buildings.

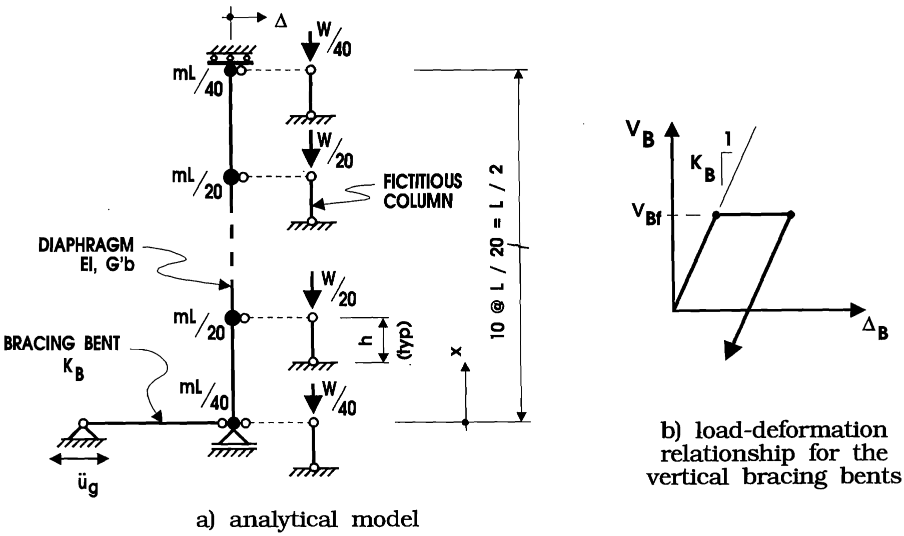

Fig. 5 Analytical models.

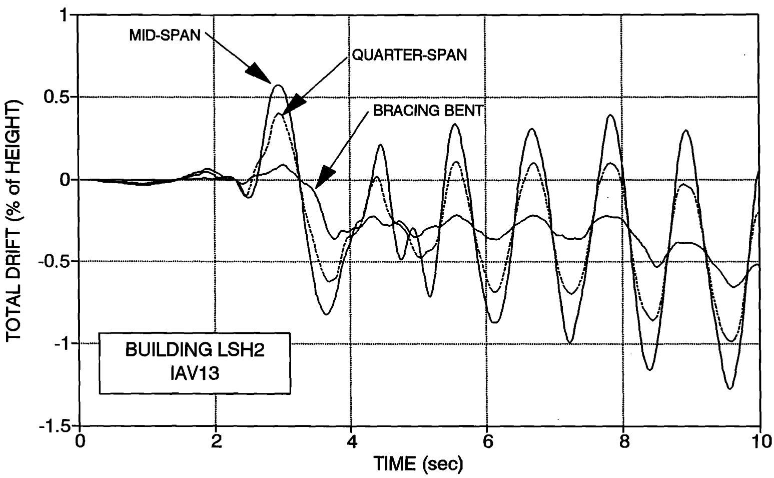

Fig. 6 Typical nonlinear seismic response (building LSH2, record IAV13).

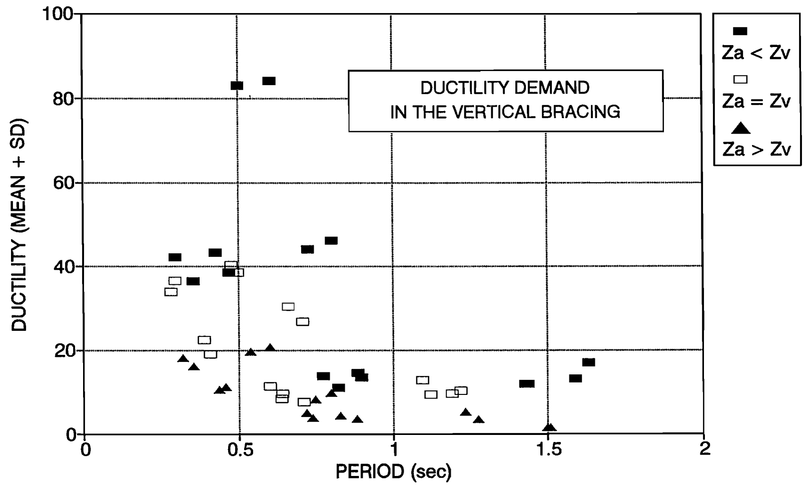

Fig. 7 Computed peak ductility in the vertical bracings.

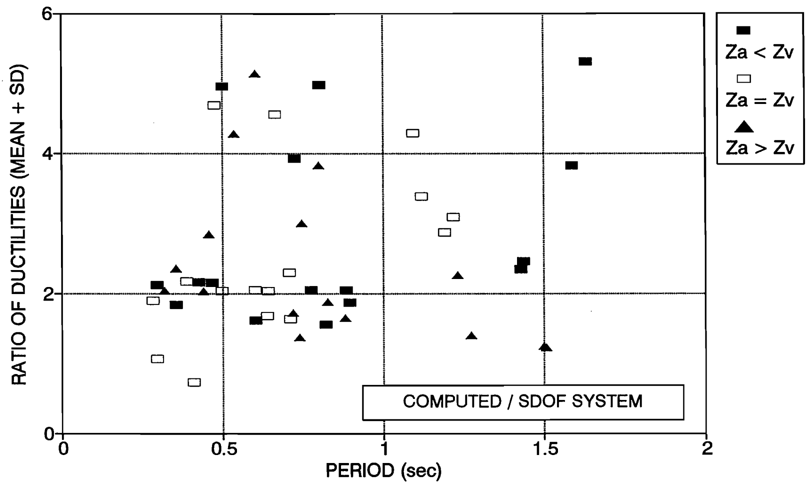

Fig. 8 Ratio of the computed to the expected peak ductility in the vertical bracings.

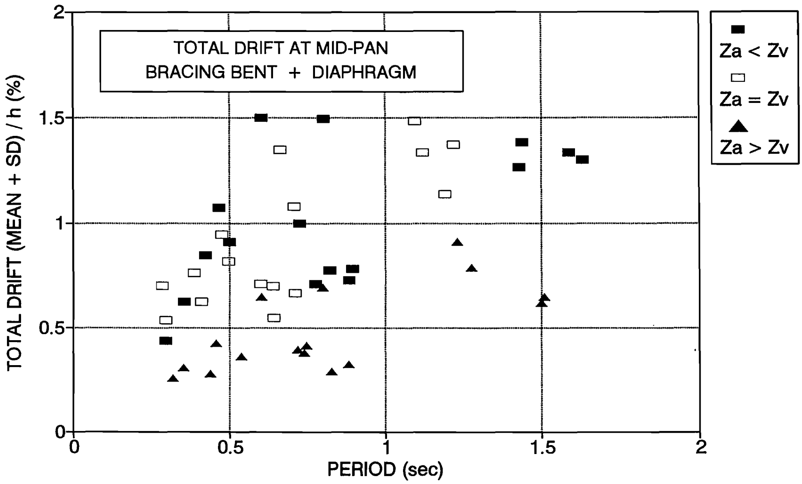

Fig. 9 Peak total drift at mid-span of roof diaphragm.

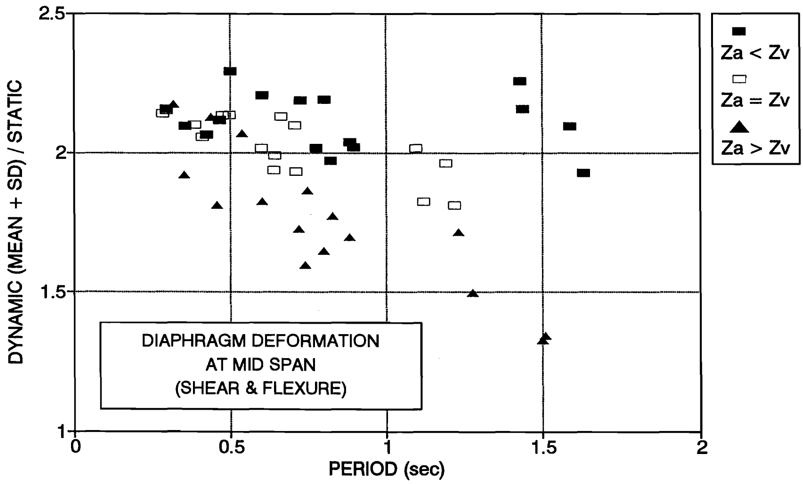

Fig. 10 Peak in-plane diaphragm deformation at mid-span of the roof diaphragm.

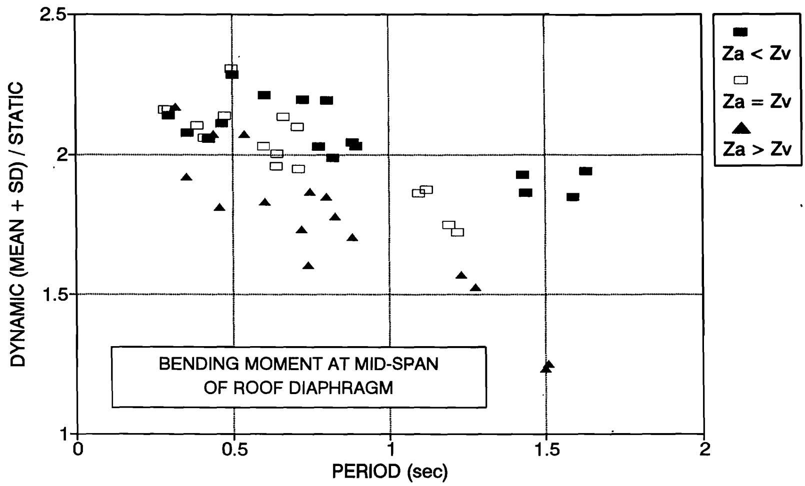

Fig. 11 Peak bending moments at mid-span of the roof diaphragm.

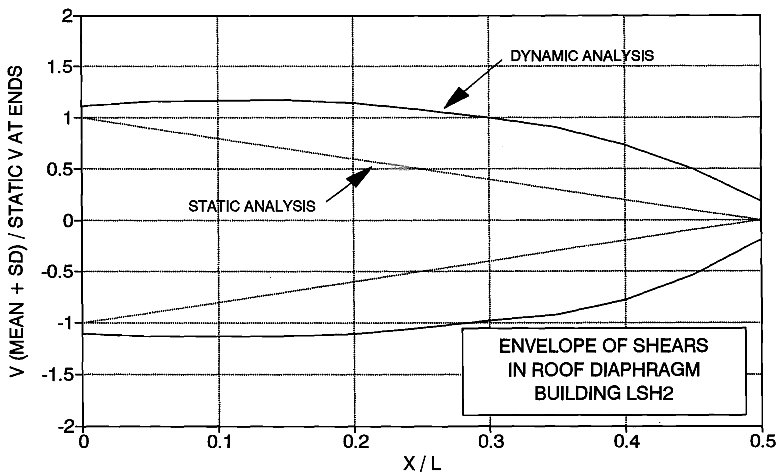

Fig. 12 Typical envelope of in-plane shear force in the roof diaphragm.

SUMMARY

Nonlinear dynamic analyses of typical single-storey steel framed buildings with metal roof deck diaphragm subjected to earthquake ground motions have been performed. The results indicate that the diaphragm experience amplified in-plane deformations and bending moments whereas the shear forces appeared to be nearly constant throughout the length of the structures.