3D seismic analysis of multi-storey wood frame construction

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ABSTRACT

In seismic design of multi-storey wood frame buildings, force reduction factors have been mainly evaluated through two-dimensional (2D) dynamic analysis on the basis of "rigid floors - symmetric behavior" assumption. In this paper, results from a 3D non-linear time-history dynamic analysis are presented for a sample wood-frame building, symmetric. The building has been designed in Vancouver (B.C., Canada) according to the provisions of the Canadian Building Code, and then analyzed using a 3D dynamic analysis program which included a pinching hysteresis model fitted to the test data obtained on full-size nailed shear wall specimens. The analyses are performed for several Vancouver area based accelerograms and several historical quakes and repeated for flexible and rigid floors to study the effect of diaphragm flexibility on seismic performance. For each building and each accelerogram, the Peak Ground Acceleration (PGAu) producing the "near collapse" ultimate state has been determined. The influence of flexibility of floor diaphragms has been then quantified in terms of reduction of PGA_u in respect to the reference "rigid/symmetric" case.

INTRODUCTION

Most seismic design codes contain action reduction factors (ARF) to be used to evaluate the forces to be accounted for when designing the structure using a simple elastic global analysis. ARF then reflects the capability of a structure to dissipate energy through inelastic behavior, and survive even exceptional earthquakes without complete collapse.

In fact any code's objective is the building to resist the foreseen quake for that area. It is evident that behind this idea there is an assumption of acceptable risk for the community. Because resistance against earthquakes results from a combination of hazard and vulnerability, to take into account the relevant uncertainties (according to semi-probabilistic approach philosophy), appropriate safety coefficients are considered in the codes both for the design action and the design resistance.

Therefore under these assumptions the easiest way to assess the appropriateness of an ARF value for a particular building type is just to refer to the definition of ARF: *"ARF is the factor to be used in calculating design inertia forces so that a structure designed linearly elastic using the code strength values can survive the design quake intensity, even if heavily damaged, but without its compete collapse."*, and apply the following procedure.

- Design the structure using the ARF according to the seismic code, and the resistant system according to the relevant codes (seismic and "static" codes). At the end of this step the resistant system will be completely anticipated.
- Model the building mechanical behaviour on the base of its mechanical characteristic (obtained by tests, and scaled to 5% percentile based on COV and test mean value, using additional safety coefficients eventually provided by the code for the earthquake load combination).
- Using a suitable non-linear analysis programme capable of following the displacement history of the building under a quake in the time domain, determine the PGA_u that the building will survive without exceeding a given "near collapse" failure limit (for example based on a maximum inter-storey drift, or a rupture in joints or in timber elements).

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- Compare this PGA_u against PGA_{code} prescribed by the code.
- Finally, if $PGA_u > PGA_{code}$ the previously chosen design ARF value is adequate.
- This procedure must be repeated for a series of earthquakes suitable for the design site, in order to have a global picture according to different possible inputs.

Note that this metodology does not need a definition of yielding limit, but uses only the definition of ultimate inter-storey drift limit. A lower yelding limit may be introduced to control damage in moderate earthquakes. If the design code values for materials strength are artificially low, the structure will be over-designed and consequently it will resist a greater PGA resulting in a greater margin between PGA_u and $PGA_{code.}$ Therefore this margin itself involves some kind of calibration of acceptable risk and becomes a matter for code writers (Ceccotti & Foschi, 1998). Using this methodology for multi-story wood frame shear walls, these factors have been previously evaluated through two-dimensional (2D) dynamic analysis on the basis of "rigid floors - symmetric behavior" assumption (Ceccotti &Karacabeyli, 2000).

The possible dynamic interaction between the floor deformability and the walls deformability has not be taken into account: under quake exitation a long building can deform along its length and some walls are more deformed than others, so that some walls can reach the near-collapse inter-storey drift while others not. Accordingly, the PGA producing the near-collapse state in a long building with deformable floors can be different from the PGA that produces the same effect in a building with rigid floors, where all walls are deforming in the same way. This is even more likely to happen in the case of non-simmetric buildings.

MATERIALS AND METHODS

<u>Modeling joints</u>

The pinching hysteresis model was previously developed at the University of Florence (Ceccotti& Vignoli, 1989) for the semirigid connections. This is a fitting model as shown in figure 1a & b.

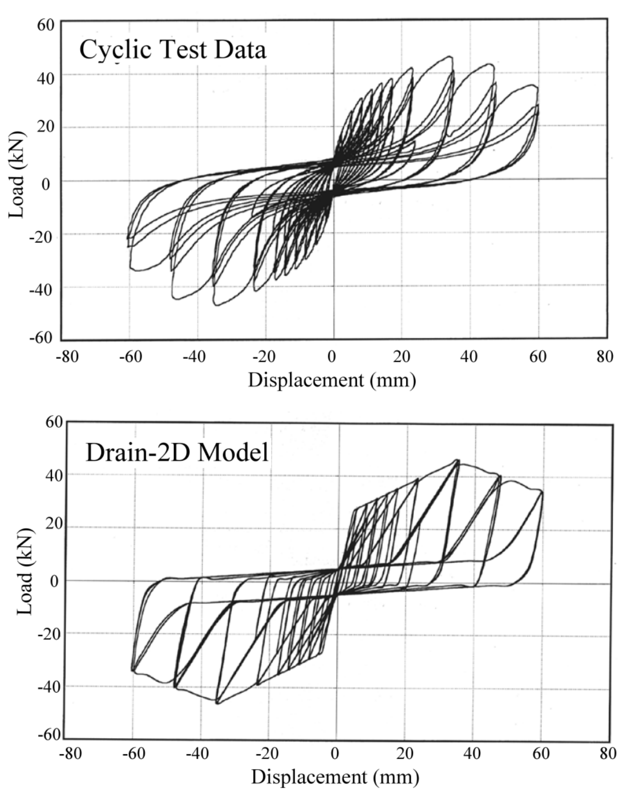

Figure 1a,b: “Florence” hysteresis model fitting plywood shear-walls test.

The model needs to be calibrated to test data. It is a piece-linear model and therefore is not extremely precise. From an energy point of view however, the representation obtained with the model is reasonable (fig 2, max error 15%).

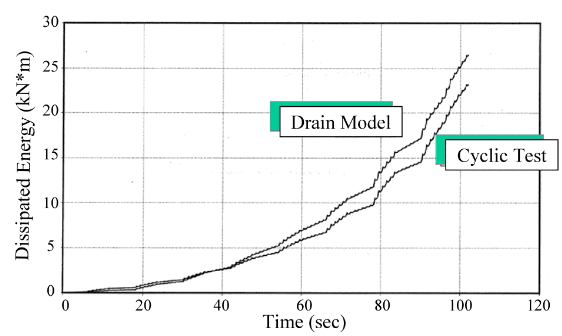

Figure 2: Differences in energy dissipation according to Figure 1.

Preliminary test results on shaking table, figure 3a,b,c,&d) validated its ability in detecting the PGA_u that is the most important parameter to be determined using this model.

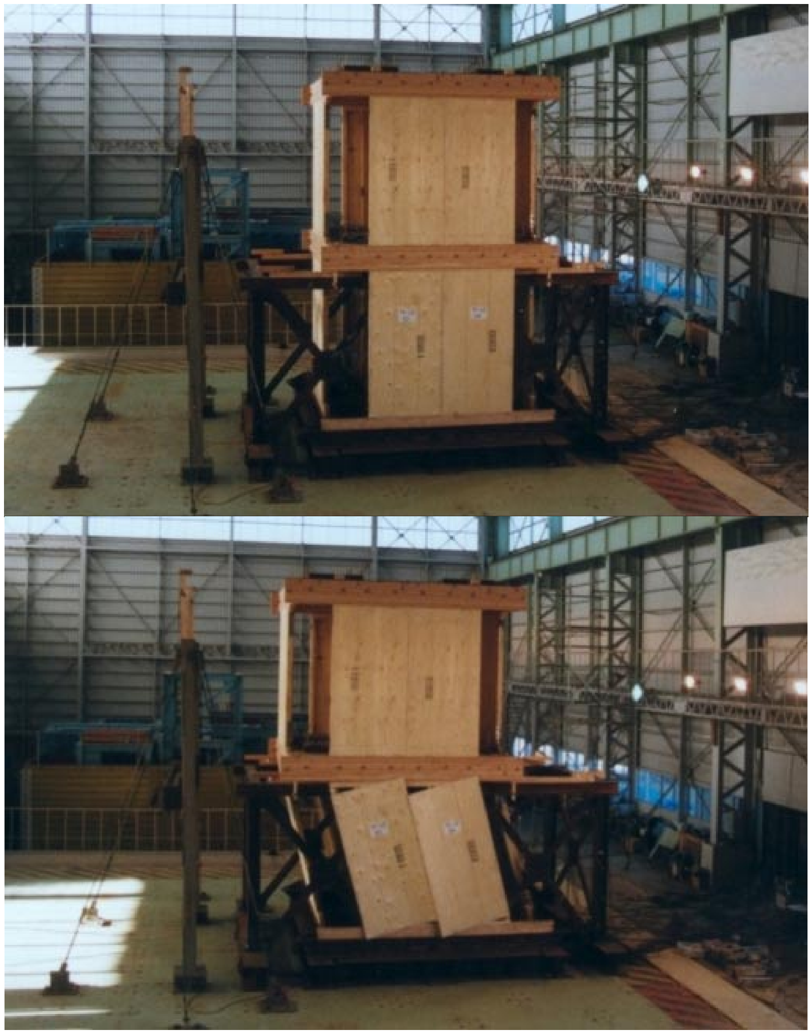

Figure 3a,b: Shaking table test performed at NIED in Tsukuba, Japan (El Centro 0.35 g, 0.45 g)

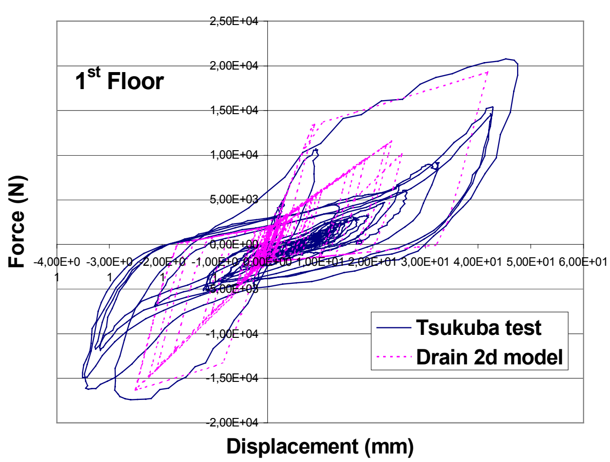

Figure 3c: Force-displacement history under El Centro 0.35 g.

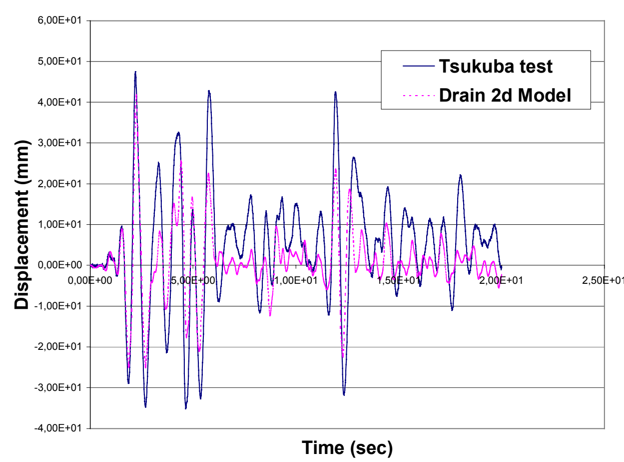

Figure 3d: Displacement-time history under El Centro 0.35 g.

Building design

A symmetric reference building has been designed in Vancouver (B.C., Canada) according to the provisions of the Canadian Building Code. It is a four storey building with 67.2 m length and 32.3 meter width . Shear walls are spaced at 9.6 m in short direction. Building was designed with ARF=3 where nailed plywood shear walls were the only lateral load resistant elements. The strength of gypsum walls was neglected. The fundamental period of vibration of the structure *(T_O)* calculated by using the Canadian Building Code was T_O= 0.2 seconds, that is much less than those found by dynamic analysis (see next). In determining the design shear force, however, T_O= 0.2 seconds was used.

Modeling the building

The building model is shown in figure 4, where timber equivalent frames with semi-rigid joints are used to simulate the behaviour of the actual walls.

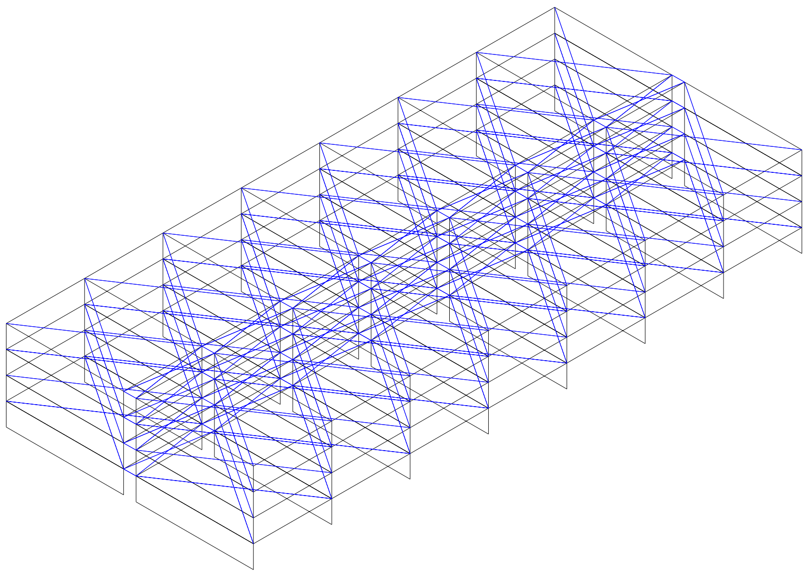

Figure 4: DRAIN 3D scheme for studied building

Floors were considered both rigid and deformable. The second feature was simulated by considering cross bracing having the equivalent deformability of a plywood floor. Though the bracing is able to plasticize, an all-elastic behaviour was forced (so deliberately neglecting the further contribution to resist the quake action given by the dissipation of energy provided by the floors themselves). Bracing is working only in tension. The effective elastic periods of the whole building with deformable floors was found to be 0.53, 0.48 and 0.43 seconds for first, second and third mode of vibration respectively (fig 5 a,b & c), while for the case with rigid floors the correspondent periods were found to be 0.51, 0.23 and 0.19 seconds for the first, second and third mode of vibration rispectively. Raleigh damping of 5% was assumed, in order to consider the contribution to dissipation of energy provided by the gypsum walls.

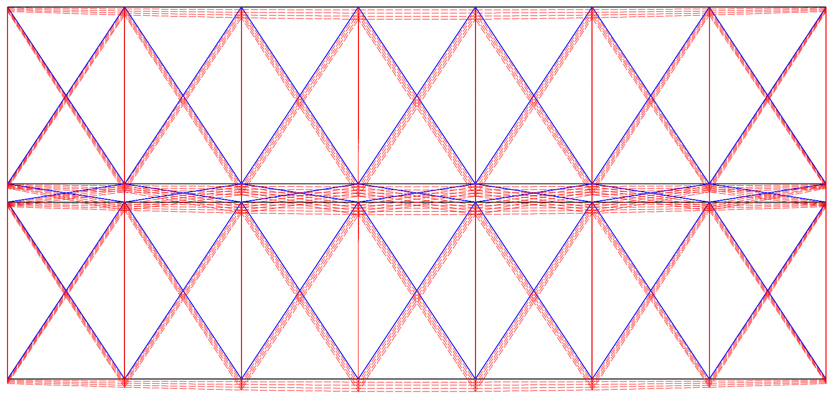

Figure 5a: 1st mode of vibration with deformable floors (T=0.531 s)

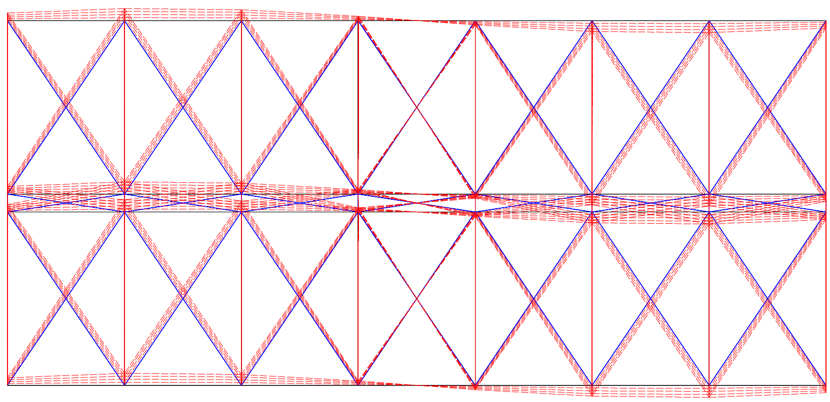

Figure 5b: 2nd mode of vibration with deformable floors (T=0.486 s)

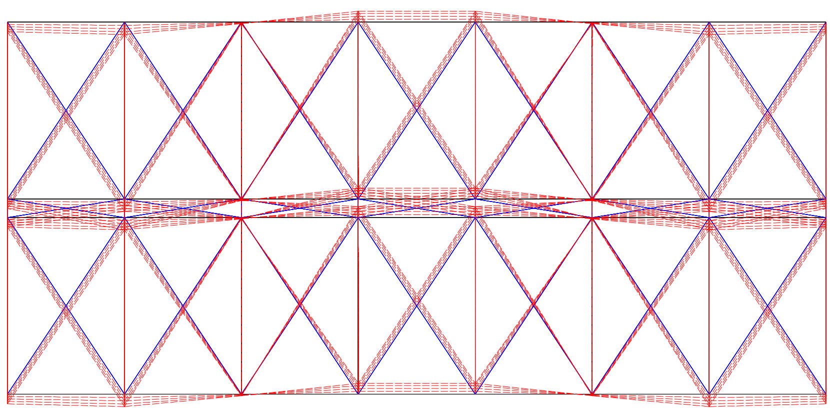

Figure 5c: 3rd mode of vibration with deformable floors (T=0.431 s)

Dynamic analysis

A non-linear time-history dynamic analysis has been performed in one direction (parallel to the short dimension of the building) for the selected wood-frame building, using 3D version of DRAIN® (Follesa, 1998) which included the "Florence" hysteresis model fitted to the test data obtained from cyclic tests on full-size nailed plywood shear wall specimens at Forintek Canada Corp. Western Laboratory in Vancouver (Karacabeyli & Ceccotti, 1996).

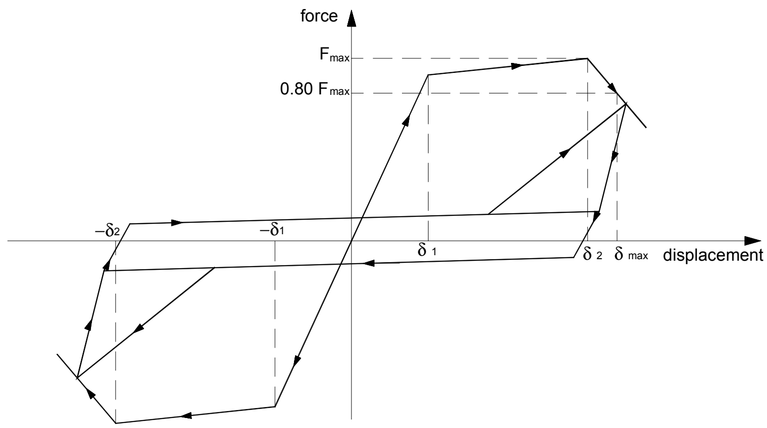

Figure 6: Near-collapse criterion (δ_{max})

The analysis was performed for several historical quakes and some Vancouver area based accelerograms, and repeated for flexible and rigid floors to study the effect of diaphragm flexibility on seismic response of the building (12 accelerograms in total).

The ultimate displacement, δ_{max}, used as the "near-collapse" criterion, is defined as the displacement at 80 percent of the maximum load on the descending portion of the skeleton curve (Figure 6).

In the analysis, the skeleton curve for the hysteresis model is selected based on the 5th percentile (determined by assuming a 10 percent coefficient of variation and a normal distribution) of the first envelope curves obtained in cyclic tests. No further adjustments were used.

Each accelerogram was scaled until the Peak Ground Acceleration (PGA_u) value producing the "near collapse" inter-storey drift was reached in the most stressed wall (Figure 7 a&b) for both cases, rigid and deformable floors.

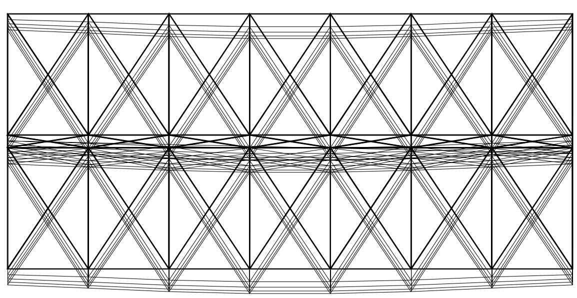

Figure 7a: Deformed shape at the near-collapse interstorey drift

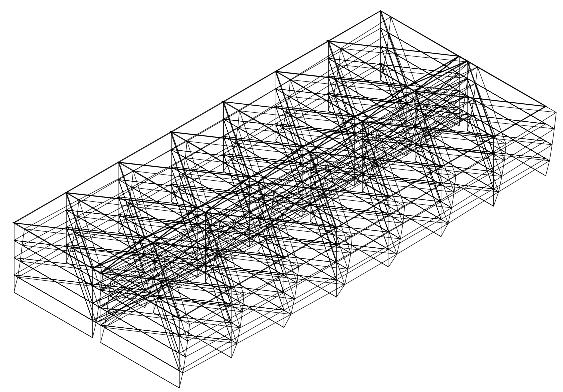

Figure 7b: Deformed shape at the near-collapse interstorey drift

Finally the influence of floor flexibility has been quantified in terms of a reduction in PGA_u with respect to the reference "rigid/symmetric" case.

RESULTS

The results of non-linear dynamic analysis are shown in Figure 8 where the reduction of the peak ground accelerations causing the inter-storey drift to reach the shear wall's ultimate displacement δ_{max} are shown for flexible floors versus rigid floor case according to the 12 accelerograms. These results lead to the following conclusions: deformability of floors reduced the achievable PGA*u* up to 15% maximum in most cases. Only in one case a 30% reduction was obtained.

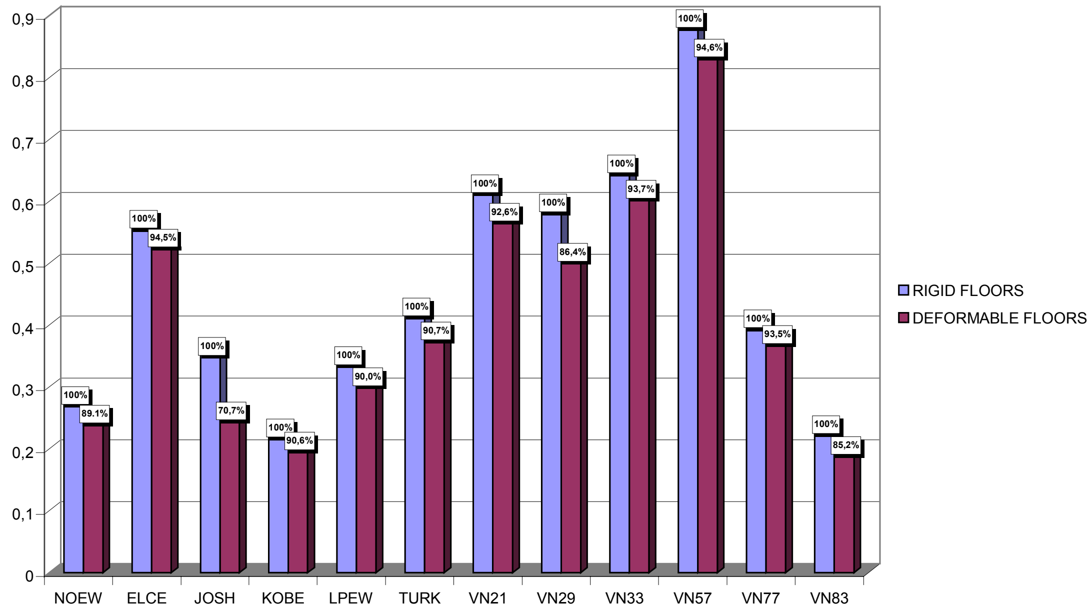

Figure 8: Loss in PGAu for deformable floors versus rigid floors.

DISCUSSION

ARF values previously found with a 2D analysis, given the unavoidable uncertainties in this problem, may be considered appropriate for symmetric buildings. Moreover the contribution of dissipation of energy has been neglected in the analysis, therefore one can expect even better behaviour in real life applications.

Authors have not found any evidence of dramatic reduction of performance in buildings with deformable floors such the one found by Stiemer and Tremblay (1996) for steel buildings. This is probably due to the differences in building configurations. In platform frame timber buildings, in general, shear walls are regularly and closely-spaced while in the steel buildings studied by Stiemer and Tremblay, shear walls were very distant from each other (30, 60 and 120m) in respect to the width of buildings (15, 30 and 60m respectively).

This leads to the conclusion that further research must be undertaken for industrial timber buildings, as the present results may be not applicable to them. In addition it is important to remember that regularities in plan and height are very important in order to assure an adequate behaviour under earthquakes. Buildings prone to torsion may be affected in a worst manner than symmetric buildings with deformable floor diaphragms.

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