3-D Pushover Analysis for Eccentric Buildings

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ABSTRACT

Torsion is recognized as a principal cause of severe damage in eccentric multistorey buildings during earthquakes. To show the damage potential of torsion in these structures, this paper extends the use of nonlinear static pushover analysis to asymmetric buildings. Two 7-storey RC ductile moment resisting frame buildings, one symmetric and the other asymmetric are designed based on Canadian codes. Pushover analyses are performed to these structures and a comparison is made on displacements, interstorey drift ratio, ductility and hinge pattern of the edge frames to show the changes in their behaviour due to torsion.

INTRODUCTION

The use of two-dimensional static inelastic analysis with monotonic increasing lateral loading, commonly referred to as "Pushover analysis", has been recognized as a useful tool in the seismic design or seismic rehabilitation of buildings (Miranda 1991, Lawson et al. 1994). Typically, such an analysis is carried out on lateral load resisting elements (frames or walls) of a building, using a pre-determined load distribution along the height of the element. Such an analysis enables the engineers to follow the development of areas of potential damage as the loading increases so that special attention can be directed to those areas.

For buildings that are asymmetrical in plan, the loading on the edge frames is affected by the eccentricity of the structure. The eccentricity changes as the building is loaded into the inelastic state because the frames do not yield at the same instance. As a result, the actual load distribution on the edge frames can be significantly different from the assumed load distribution on the building.

The purpose of this study is to explore the application of pushover analysis for buildings that are asymmetrical in plan. A seven-storey reinforced concrete ductile moment resisting frame building is considered. It has three parallel frames with frame 2 centrally located, and frames 1 and 3 located at equal distance but on opposite side of frame 2, as shown in Fig. 1.

Based on such a plan, two seven storey buildings are created. In Building S (S for symmetrical), the centres of mass (CM) are assumed to be coincide with frame 2. In Building A (A for asymmetrical), CM is located between frames 2 and 3 at a distance equal to 0.15 b (b= width of building) from frame 2 at each floor.

All these frames in the building have the same member dimensions, but their strength is designed to satisfy the minimum loading specified by the National Building Code of Canada (NBCC 1990) and

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following the design requirements specified in the Canadian standard for concrete structures (CSA. A23.3-M84, chapter 21).
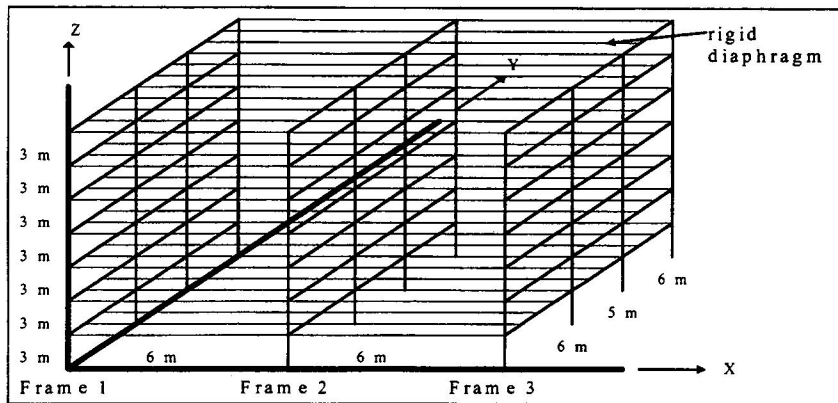

Figure 1. The seven-storey building

METHOD OF ANALYSIS

Gravity loading is first applied to the buildings S and A. Then, a set of monotonic increasing static lateral load are applied at the CM of the floors. The distribution of the lateral loading suggested by NBCC 1990 is used. The analysis is carried out using the computer code CANNY-C (Li 1993).

Due to the location of the CM in relation to the frames, frame 3 is the most susceptible to the effect of torsion in building A. For this reason, the responses of frame 3 will be the major focus in this paper. A comparison of frame 3 responses from buildings S and A will show the effect of eccentricity on the edge frames located on the flexible side of an asymmetrical building.

SYMMETRIC BUILDING

The symmetric building will be studied first and it serves as the reference system. Fig. 2 shows the normalized lateral load-displacement diagram for the symmetric building. In this diagram "Base Shear Ratio" is total applied lateral load divided by design base shear of the building. "H^*" is the height of an equivalent single degree of freedom system which can serve as a dynamic model for the building. For these 7-storey buildings, H^* equals approximately to 15 meter, which is the height of the fifth floor. Based on Fig. 2, the following load levels (L.L.) are defined:

      L.L. 1 = Lateral load equal to design base shear
      L.L. 2 = Lateral load level when the first beam hinging in frame 3 occurs
      L.L. 3 = Lateral load level when the first column hinging in frame 3 occurs
      L.L. 4 = Lateral load level when: (displacement at height H^* / H^*) = 0.5%
      L.L. 5 = Lateral load level when: (displacement at height H^* / H^*) = 1.0%

It should be noticed that even in designing symmetric buildings the effects of accidental torsion should be included. Therefore frame 2 has a lower lateral strength than frames 1 or 3. Due to this fact the first hinging in this building occurs at a beam in frame 2 (base shear ratio = 1.24). The first beam hinging in frame 1 and 3 occurs later at a base shear ratio equal to 1.47. Fig. 3 shows the hinge pattern in

the frames 2 and 3 of symmetric building at load level 4. The hinges are numbered in both frames in the order as they appear during the pushover loading process. Although all the frames have the same hinge pattern, the sequences of hinging are different. In fact there are more than ten hinges in frame 2 before frame 3 develops a hinge. Both frames have hinges mainly in the beams except at the column base at the first floor. This indicates that they follow the strong column-weak beam design philosophy.

The deformation shapes of building at different lateral load level are shown on Fig. 4. This figure indicates that general displacement shape does not change drastically as the frames become inelastic.

Fig. 5 demonstrates the interstorey drift ratio of the symmetric building at different load levels. The terms that are used in this paper are consistent with those suggested by Moehle (1992). Roof (top) displacement refers to lateral displacement of the roof relative to the base. Interstorey drift refers to the relative lateral displacement between two adjacent floors. Drift and displacement are used interchangeably. Drift ratio refers to the drift divided by the height above the base, except interstorey drift ratio refers to the difference in lateral displacements for two adjacent floors divided by the distance between the floors. The maximum interstorey drift ratio is in the second or the third storey. This diagram can be used to establish a measure of damage potential for the building. For example if it is assumed that the

Figure 2. Load-displacement diagram of the symmetric building
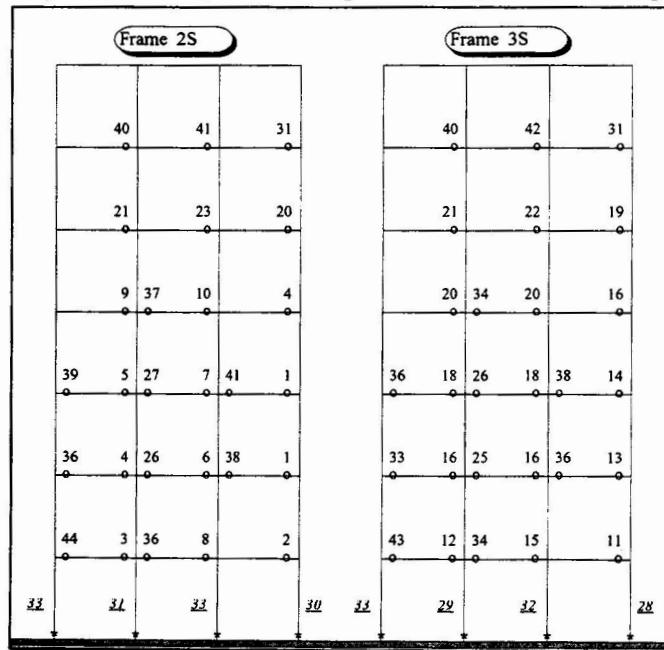
Figure 3. Hinge pattern of symmetric building (L.L. 4)

capacity of non-structural elements for interstorey drift ratio is 0.5% (Mayes 1994) then Fig. 5 shows that at L.L. 4 some damage to non-structural elements can be expected in the 2nd, 3rd and 4th-floor.
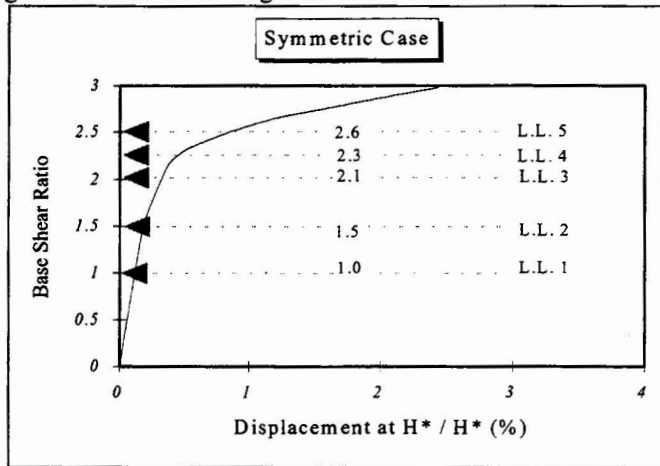
In order to relate the results obtained from a pushover analysis to the dynamic results using a single degree of freedom system as a dynamic model, it is necessary to relate the interstorey drift ratios to the drift ratio at level H^*. This relation is shown in Fig. 6 in which interstorey drift ratios are normalized by the drift ratio at level H^*. The maximum normalized interstorey drift ratio is about 1.2 for all the load levels considered. This is similar to the value reported by Moehle (Moehle 1992) for ductile moment resisting RC frames.
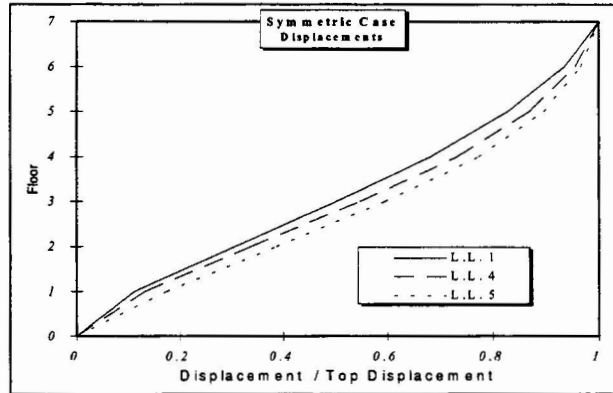
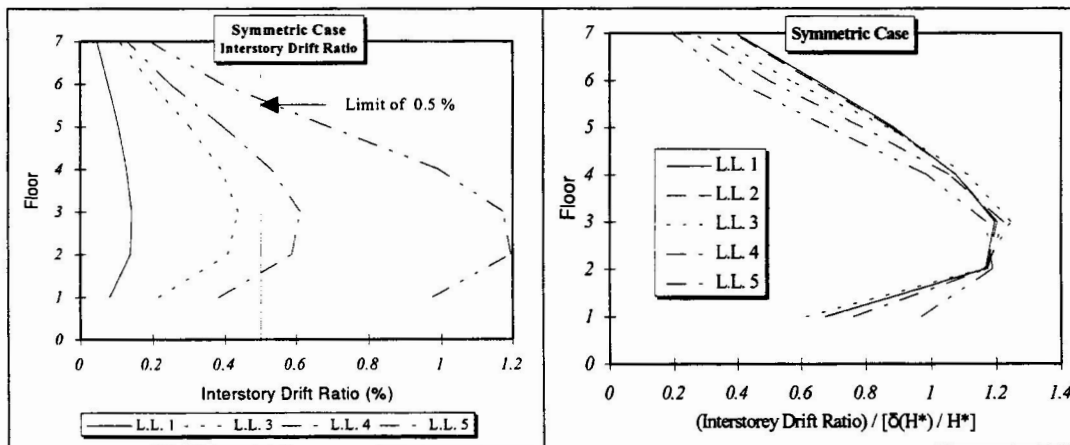Figure 4. Displacement shapes for symmetric building

Figure 5. Interstorey drift of symmetric building

Figure 6. Normalized Interstorey drift ratio

COMPARISON OF FRAME 3 IN SYMMETRIC AND ASYMMETRIC BUILDINGS

In this section, the behaviour of the frame on the flexible side (frame 3) in Building S and Building A are compared. For simplicity, these frames will be referred to as Frame 3S and Frame 3A respectively. The comparison is made based on the displacements, interstorey drift ratios, ductility demands and hinge patterns of the two frames.

Displacements

A comparison of the "load"-displacement curve for the frames is shown in Fig. 7. It should be noted that the "load" in the figure refers to the total applied lateral load on the buildings and not the loads on frame 3. Under the same loading on the building, the Frame 3A has a large displacement because of torsion. As a result, the Frame 3A appears to be less stiff than Frame 3S. The behaviour of the two frames can best be discussed using Table 1. Listed in the first two columns of Table 1 are the total
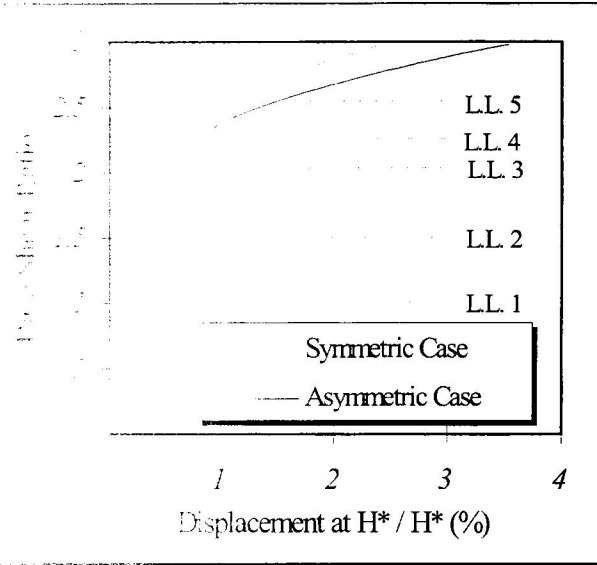
Figure: Lateral load-displacement diagram

(axis labels) Displacement at H* / H* (%)

Legend: L.L. 5, L.L. 4, L.L. 3, L.L. 2, L.L. 1

Symmetric Case
—— Asymmetric Case

... computed and listed in Table 1. The effective
... when the first column hinge forms in the
... in effective global ductility for Frame 3A

... in analyses

	Comments
	At design base shear of building
	First beam hinge (Frame 3S)
	First beam hinge (Frame 3A)
	First column hinge (Frame 3A)
	$[\delta(H^*) / H^*] = 0.5\%$ (Frame 3A)
	First column hinge (Frame 3S)
	$[\delta(H^*) / H^*] = 0.5\%$ (Frame 3S)
	$[\delta(H^*) / H^*] = 1.0\%$ (Frame 3A)
	$[\delta(H^*) / H^*] = 1.0\%$ (Frame 3S)

Interstorey drift ratio

The interstorey drift ratios of the two frames at L.L. 4 are shown in Fig. 8. Both frames have their maximum intrestorey drift ratio located at the third storey. However, Frame 3A has a significantly larger interstorey drift ratio. L.L. 4 is defined as a load level that will give an overall drift ratio of 0.5% to Building S. At this load level, all bottom five storey in Frame 3A have an interstorey drift ratio that exceeds 0.5%, with the 3rd and 4th-storey having an interstorey drift ratio approaching unity. Using published data on non-structural element damages (Mayes 1994), one can expect the large interstorey drift of Frame 3A will cause severe non-structural element damages in the 3rd and 4th-floors.

Despite the difference in interstorey drift ratios, the normalized interstorey drift ratios for the two frames are similar, as shown in Fig. 9. This implies that if one can estimate the overall drift ratio $[\delta(H^*) / H^*]$ for Frame 3A at any load level then the interstorey drift ratio curve for the frame can be constructed, and the non-structural damage potential at the flexible edge of the eccentric building can be estimated. The overall drift ratios at different load levels are shown in Table 1 for both frames. In the inelastic range beyond L.L. 3, the increase of overall drift ratio due to torsion is in the 60% range.

Ductility

The beam and column ductility demands are taken as the structural damage indices for Frames 3S and 3A. Shown in Fig. 10 are the maximum beam and column ductility demand across a floor in each of the two frames at L.L. 4.

Figure 8. Interstorey drift ratio

Figure 9. Normalized interstorey drift ratio

There is larger ductility demand, both for the beams and columns, in Frame 3A. While the beam ductility demand remains essentially the same at the lower floors of Frame 3S, larger demand exists at the 2nd, 3rd and 4th-floors in Frame 3A. The increase in ductility demand in these floors is about 50%. Similarly, there is an increase of ductility demand on the columns of Frame 3A over Frame 3S. In Frame 3S, the column ductility demand is below unity except at the base. In Frame 3A, column ductility demand exceeds unity at upper floors, and indicates column damage at these floors.
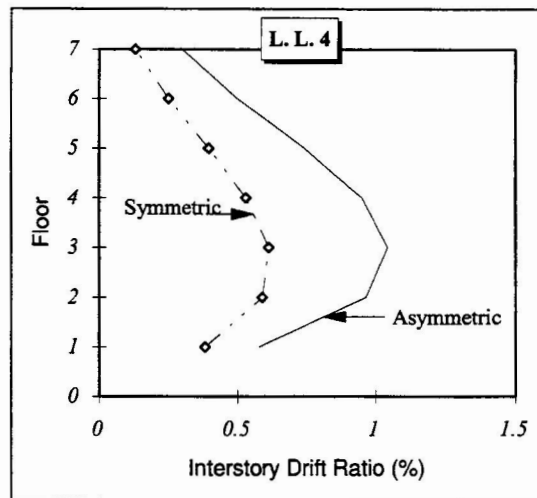
CONCLUSIONS

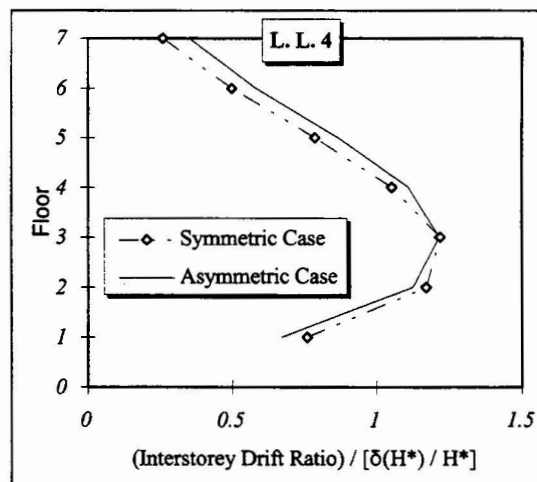This paper demonstrates the application of a 3-D pushover analysis for an asymmetrical RC ductile moment resisting frame building designed following the Canadian codes. Special focus is on the responses of the frame at the flexible edge of the building which is particularly vulnerable to torsional damage. To highlight the effect of torsion, the results are compared to the responses of an edge frame in a similar but structurally symmetrical building. The comparison is carried out based on the displacements, interstorey drift ratios, ductility demands, and damage pattern of the two frames. It is shown that when subjected to the same level of lateral loads on the building, the edge frame of the asymmetrical building will experience significantly larger interstorey drift ratio, larger ductility demand on both the beams and columns, and will have a different damage pattern from the edge frame of the symmetrical building. These changes may be the results of load redistribution among different frames when the building is pushed into the inelastic range. In view of the noted differences, a standard 2-D pushover analysis may not be adequate to evaluate the damage potential of an edge frame in an asymmetrical building. A 3-D pushover analysis is recommended for this type of building.

ACKNOWLEDGMENTS

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