Seismic Deformation Demands on Gravity-Load Columns in Shear Wall Buildings

P. Adebar, P. Bazargani & H. Chin
Department of Civil Engineering, University of British Columbia, Canada

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LISBOA 2012

SUMMARY:
Procedures are presented for estimating seismic deformation demands on gravity-load columns in concrete shear wall buildings. The input is the top wall displacement, which is used to estimate wall drifts over the building height and maximum wall curvatures. A method is presented for estimating additional wall drifts due to shear deformation in the plastic hinge regions of flexural walls, which test have shown to be very significant. The deformations of the concrete shear walls are used as input to a nonlinear analysis of wall – column systems. Different procedures are used when the floors are flexible, e.g., flat plates slabs, or are fairly rigid, e.g., large beams framing into the columns. As expected, elongated gravity-load columns are predicted to have significantly reduced drift capacities compared to square columns. Preliminary results are presented from an on-going experimental study on elongated gravity-load columns subjected to seismic deformation demands.

Keywords: Displacement-based design, experiments, reinforced concrete columns, shear wall buildings.

1. INTRODUCTION

Traditional seismic design provisions for buildings are force based. The portion of structure that resists most of the lateral seismic force is treated differently even though the entire structure is subjected to the same earthquake-induced displacements. In Canada, gravity-load columns are often elongated like a wall, e.g., 30 × 240 cm, because the longer dimension (240 cm) reduces the slab span, while the smaller (30 cm) dimension allows the column to be hidden within walls. Until recently, it was common in Canada to have concrete shear walls with special seismic detailing immediately adjacent to elongated gravity-load columns that look like walls but are subjected to much higher axial loads as a ratio of $f_c'A_g$ and have no seismic detailing for ductility. Concern over the safety of such gravity-load columns (Adebar et al., 2010) resulted in a change to the Canadian Concrete Code (CSA A23.3 2004) in August 2009. This paper presents some of the recent research that is expected to lead to further refinements of the displacement-based design requirements for gravity-load columns in the 2014 edition of the Canadian Concrete Code.

The current project involved analytical research to estimate the deformation demands on gravity-load columns due to the deformations of concrete shear walls connected to gravity-load columns by many floor levels in multi-level buildings, as well as experimental research on the deformation capacity of elongated gravity-load columns subjected to high levels of axial compression.

2. SHEAR WALL DEFORMATIONS

Flexural deformations of shear walls control building drifts in the upper levels as well as a large portion of the drifts in the lower levels. Shear deformations of the plastic hinge region of shear walls, where the vertical reinforcement in the wall is yielding, contributes significantly to the drifts near the base of the building.

2.1 Flexural deformations

Dezhdar and Adebar (2012) conducted numerous nonlinear response history analysis on 13 different shear wall buildings in order to develop simplified methods for predicting seismic demands on cantilever shear wall buildings from linear dynamic (response spectrum) analysis. This included the effective flexural rigidity of cantilever shear walls that should be used to obtain an accurate estimate of maximum roof displacement using linear dynamic (response spectrum) analysis, a simplified envelope of interstory drifts that can be used to estimate seismic deformation demands on gravity-load frame members and the maximum curvature demand at the base of cantilever shear walls.

A safe estimate of the maximum curvature demand ϕ_d at the base of a shear wall is given by:

$$\phi_d = \phi_y + \Delta_i/(h_w - 0.5l_p)l_p \qquad (2.1)$$

Where ϕ_y is the yield curvature (maximum elastic curvature), which varies between $0.0025/l_w$ (for an "I"-shaped wall with a large flange to $0.004/l_w$ (for a rectangular wall); Δ_i is the inelastic portion of the displacement demand, which can be estimated as $\Delta(1 - 1/R)$ (Adebar et al., 2005), Δ is the total displacement demand at the top of the wall and R is the ratio of elastic bending moment demand at the base of the wall to nominal bending moment capacity of the wall; h_w is the height of the wall from the location of maximum base curvature to where the horizontal displacement Δ_i occurs; l_p is the length (height) of uniform maximum base curvature, which can be safely estimated as $0.5l_w$ or for taller walls, more appropriately by $0.2l_w + 0.05h_w$ (Bohl and Adebar, 2011).

Dezhdar and Adebar (2012) provide an envelope of maximum interstory drifts over the height of cantilever shear walls. The mean plus one standard deviation interstory drifts are equal to $2.2\,\Delta/h_w$ over the top 25% of the building height (from $0.75h_w$ to $1.0h_w$). The interstory drifts reduce linearly from $2.2\,\Delta/h_w$ at $0.75h_w$ to $1.0\,\Delta/h_w$ at the base of the building. Further details are given by Dezhdar and Adebar (2012).

2.2 Shear deformations

Although shear deformations generally constitute a minor portion of the total wall displacement at the top of tall flexural walls, experiments on flexural walls have shown that the contribution to wall drifts within the plastic hinge region is very significant compared to flexural deformations. These additional deformations are particularly important for gravity-load columns which are subjected to the maximum axial loads over the height of the plastic hinge region in cantilever walls.

Shear strains in the plastic hinge region of flexural walls are the result of large axial tensile strains combined with inclined cracks. A simple model was developed to estimate the average shear strains of flexural walls assuming the average biaxial strain conditions of the plastic hinge region shown in Figs. 1(b) and 1(c). As the horizontal reinforcement in flexural dominated walls maintain the horizontal normal strain at very low values compared to the vertical strains, the horizontal normal strain was assumed to be zero for simplicity. From the geometry of the Mohr circle of strain and the assumption of zero horizontal normal strain, the average shear strain can be calculated very simply as:

$$\gamma_{vh} = \varepsilon_v \,.\, |\tan(2\theta)| \qquad (2.2)$$

where ε_v is the average vertical strain and θ is the angle between the principle compression strain direction and the vertical axis of the wall. Extensive nonlinear finite element analysis (Bazargani 2012) indicate that the average principle strain angle θ remains fairly constant on the tension side of the plastic hinge zone and that assuming the angle to be 75° gives very good estimates of wall shear strain.

Assuming a linear variation of vertical strain in the wall (Fig. 1a), the average vertical strain can be

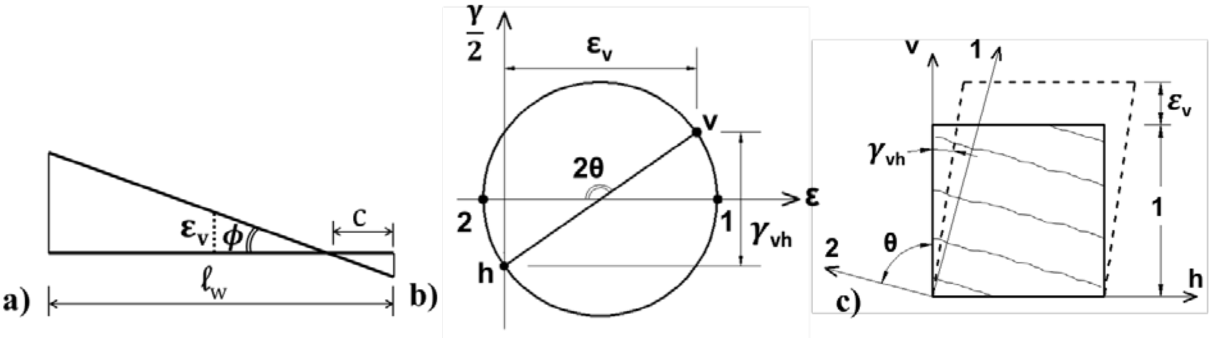

Figure 1. (a) Linear variation of vertical strains over flexural wall length, (b) Mohr circle for typical average biaxial strains in plastic hinge zone of flexural walls, and (c) biaxial deformations of wall hinge zone with zero horizontal strain and principal compression inclined at θ = 75° from vertical

calculated from:

$$\varepsilon_v = \phi\ (l_w/2 - c) \tag{2.3}$$

where c is the horizontal distance from the compression face of the wall to the neutral axis (fairly constant in the plastic hinge region), l_w is the length of the wall section and ϕ is wall curvature demand discussed in Section 2.1 (see Fig. 1a).

Combining the equations above results in a simple expression for estimating shear strains in the plastic hinge zone of flexural shear walls:

$$\gamma = 0.58\,\phi\ (l_w/2 - c) \tag{2.4}$$

The direct link between curvature and shear strain of flexural walls given by Eq. 2.4 has been observed in many tests (Bazargani, 2012). Figure 2.2 compares predictions from Eq. 2.4 with results from these tests. It is important to note that the experimentally observed shear strain in the plastic hinge region of flexurally dominated concrete walls has been as large as 0.8%.

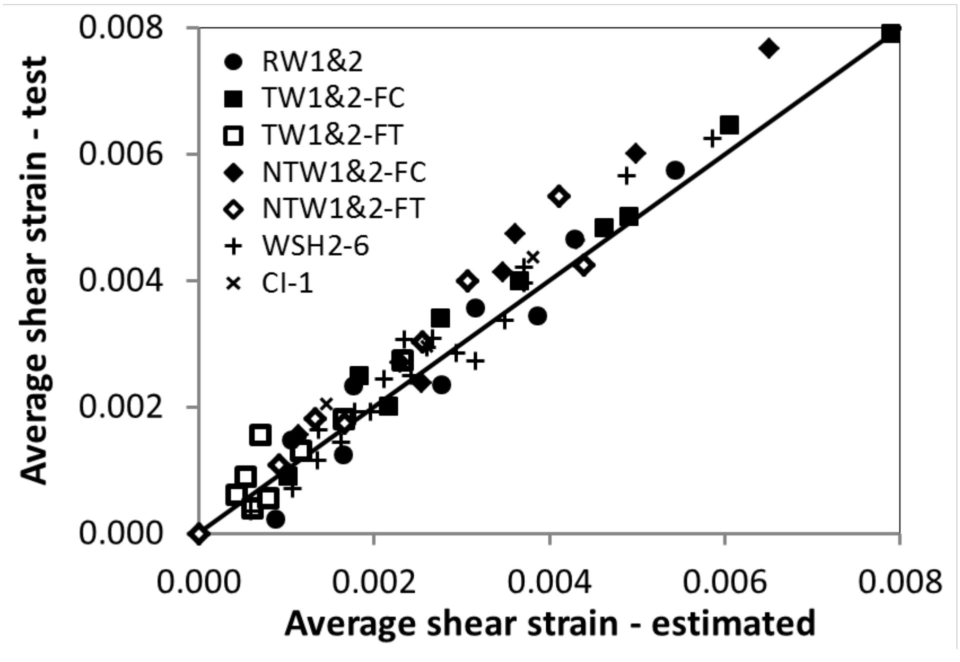

Figure 2. Comparison of predicted average shear strains using Eq. 2.4 with measured shear strains in flexurally dominated shear wall experiments.

3. COLUMN DEFORMATION DEMANDS

Deformation demands on gravity-load columns depend on the overall deformations of the building as controlled by the shear walls and the relative stiffness of the floor system and gravity-load columns. If the out-of-plane bending stiffness of the floor system is low compared to the bending stiffness of the gravity-load column, for example when the floors consist of a thin flat-plate floor slabs, all the localized bending will occur in the floor slabs. The slabs, which have high in-plane stiffness, will push and pull the columns so that they have the same displacement profile as the walls; but will not induce significant localized bending moments into the columns. On the other hand, when the floor system has significant bending stiffness, for example when large beams frame into gravity-load columns, significant column drift will occur over a region of uniform building drift. These two cases are discussed separately below.

3.1 Flat-Plate Floor Systems

For the purpose of determining the deformation demands on gravity-load columns, flat-plate floor systems have low out-of-plane bending stiffness; but high in-plane stiffness. They can be modelled as rigid pin-ended members that interconnect the shear walls and gravity-load columns. When there are numerous floor slabs, such as in a multi-story building, the gravity-load columns will have the same displacement profile as the shear walls at floor slab levels.

The flexural deformation demands on a gravity-load column can be determined using the nonlinear moment-curvature response of the column (e.g., shown later in Fig. 5) in an analysis where the column is pushed to the deformation profile of the wall at the floor slab levels (Bazargani and Adebar, 2010). It was found that using five levels above the level with maximum curvature demands in the wall (i.e., the critical section at the base of the wall where yielding in the wall starts) and three (basement) levels below the wall base results in the same demands as when more levels are included in the analysis.

The nonlinear moment-curvature response was used up to the point of peak moment strength and beyond that point, the response was assumed to be perfectly “plastic” over a certain height of the column analogous to a plastic hinge. The Canadian concrete code limits the maximum compression strain of concrete to 0.0035. The column curvature capacity was assumed to be the curvature at which the maximum compression strain of concrete is first reached. A curvature capacity and “plastic hinge” height of the column, define the deformation capacity of the column. A comprehensive study was carried out to examine the effect that different shear wall and gravity-load column parameters and configurations have on column demands (Bazargani, 2012). Some results from this study are briefly summarized below.

Figure 3 summarizes the results from eight different push-over analyses of shear wall – gravity-load column systems. The two parameters were whether or not the shear wall had significant shear deformations and the length of the gravity-load column. In all cases, the columns were subjected to an axial compressive force of $0.10f_c'A_g$. The shear wall deformations are not influenced by the gravity-load columns and Fig. 3 presents only the flexural component of the shear wall response. Thus there is only one (red) line in Fig. 3 that represents the response of the shear walls in all eight analyses. The response of the gravity-load column does strongly depend on the length of the column and whether or not there are wall shear strains. Thus there are eight separate column response curves shown in Fig. 3.

When shear wall deformation was due to flexural deformations only (no wall shear strain), and both the shear wall and gravity-load column have a fixed base at the same level, the curvature distribution in the column closely followed that of the wall, resulting in the maximum column curvature demand being approximately equal to the maximum wall curvature demand. When the shear wall experiences significant shear strains, the drift demands on the columns are increased; but as the columns do not undergo shear deformation (large axial compression in the columns prevents diagonal cracks and vertical tension strains), the additional drifts cause increased bending (curvatures) in the columns, which increases the compression strain demands on the column.

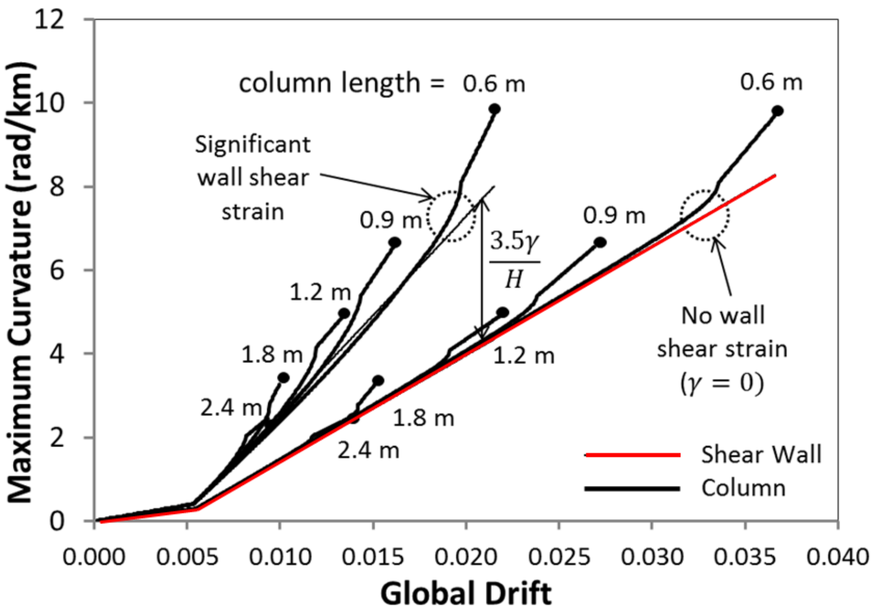

Figure 3. Effect of column length and wall shear strain on curvature demand and drift capacity of gravity-load columns subjected an axial compressive force of $0.10f_c'A_g$.

The additional curvature demand on a column $\Delta\phi_{col}$ due to wall shear strain can be estimated as

$$\Delta\phi_{col} = 3.5\gamma/h_s \tag{3.1}$$

where h_s is the height of the first story and γ is the average wall shear strain in the plastic hinge zone. As described above, there is a linear relationship between wall maximum curvature and average shear strain thus the additional curvature demand on the column will also be proportional to the maximum curvature demand on the wall. The additional curvatures predicted by Eq. 3.1 are shown in Fig. 3.

As expected, the longer-length columns reached their curvature capacities at lower drift levels because longer concrete compression depths are required to sustain the axial load on the more elongated columns, which results in these columns having a lower curvature capacity.

The conclusions from some of the other analyses follow. Damage of the column due to loss of concrete cover and buckling of vertical reinforcement caused a concentration of curvature at the location of damage that further increased the maximum curvature demand and reduced the column drift capacity. The specific heights of the column “plastic hinge” zone or damaged zone were found to have a relatively small effect on the maximum curvature demands on gravity-load columns. This was due to the short curvature reserve from the point of peak moment strength on the moment curvature response of the column to the point of concrete crushing or column curvature capacity.

Flexibility of the column support condition at the base was found to have an important impact on column curvature demand. When the column with the same dimensions continues below the base of the shear wall and the floor system at that level does not provide significant restraint to rotation of the column at the wall base (i.e., the column is pinned at the base), the column curvature demands are significantly reduced. This highlights the importance of accurately modelling the support conditions at the base of the gravity-load columns.

An increase in the first storey height increased the maximum column curvature demand because the slabs that help to gradually bend the column were not present resulting in a concentration of curvatures at the base of the column.

3.2 Floor Systems with Significant Flexural Stiffness

When the floor system consists of thin floor slabs, uniform building drift will not cause significant column drift. The columns will be inclined at the angle of the building drift; but will be straight (not bent). In this case, all the required rotations in the column-slab frame will occur in the floor slabs. On the other hand, when the floor system has significant flexural stiffness, e.g., large beams that frame into the columns, uniform interstory drift will cause column drifts.

A simple expression can be developed for the column drift that results from uniform interstory drift by assuming the following: the floors and columns are relatively uniform over a number of stories, and the stiffnesses of the floors and columns can be reasonably represented by a constant flexural rigidity EI_e. The ratio of flexural stiffnesses of the floors and columns are expressed using the following parameter:

$$\alpha = \frac{EI_{e\,col}/h_{col}}{EI_{e\,flr}/L_{flr}} \tag{3.2}$$

where $EI_{e\ col}$ and $EI_{e\ flr}$ are the effective flexural rigidity of the column and floor, h_{col} is the column story height and L_{flr} is the span of the floor from the shear wall to the column.

The ratio of column drift to interstory drift is given by 1.5/(3α + 1). This relationship is plotted in Fig. 4 using a logarithmic scale. Note that if the flexural stiffness EI_e/L of the columns is 100 times the flexural stiffness of the floor (logα = +1.0), the column drift will be only 5% of the uniform interstory drift, while when the floor stiffness is 100 times the column stiffness (logα = -1.0), the column drift is 115% of the interstory drift.

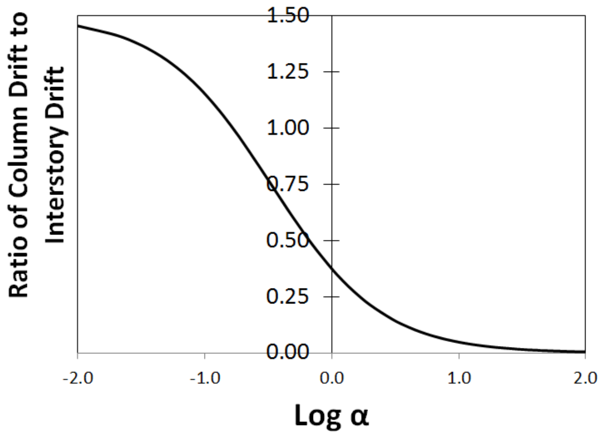

Figure 4. Variation of column drift demands (as a ratio of interstory drift) with the logarithm of column-to-floor stiffness ratio given by Eq. (3.2).

If a linear bending moment – curvature relationship is assumed for simplicity, the column curvature demand ϕ_c due to the building drift θ_w can be estimated as follows.

$$\phi_c = \frac{9}{3\alpha+1} \times \frac{\theta_w}{h_{col}} \tag{3.3}$$

Accounting for the actual non-linear bending moment – curvature relationship of the column will further increase the curvature demands on the column. As described in Section 3.2, the interstory drifts are largest near the top of the building. Thus if the floor systems are equally stiff over many floors, the largest curvature demands on the gravity-load columns will occur at the upper floors. On the other hand, the largest axial loads in the columns occur in the lower floors and curvature capacity reduces with increasing axial load.

4. DRIFT CAPACITY OF GRAVITY-LOAD COLUMNS

Figure 5 shows an example bending moment – curvature relationship for a gravity-load column. The example column was tested as part of the experimental study described later in the paper. The axial compression applied to the column is slightly less than the load that results in the balanced strain condition at failure. Thus the vertical reinforcement yields at a curvature only slightly less than the curvature at which the maximum compression strain reaches 0.0035 (see Fig. 5). The shape of the bending moment – curvature relationship is defined primarily by the concrete compression stress-strain relationship.

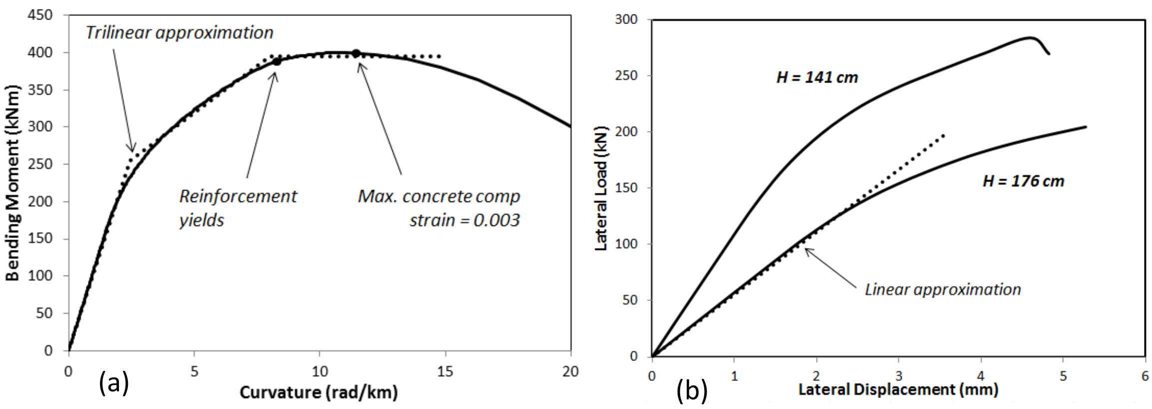

Figure 5. Example response of a gravity-load column: (a) bending moment – curvature response, (b) load – deformation response of the column assuming two different heights *H* of the cantilever column.

If the nonlinear bending moment – curvature relationship is used to estimate the lateral load – deformation relationship of the cantilever column and the bending moment to shear ratio $M/V = H$ at the base of the column is 141 cm, the relationship given in Fig. 5(b) results. The load – deformation relationship is very peaked because the nonlinear curvatures are very concentrated at the base of the column. A trilinear approximation of the bending moment – curvature relationship is also shown in Fig. 5(a). In order to use this relationship to estimate the load – deformation response, an assumption needs to be made for the "plastic hinge" length, i.e., the height over which the curvature capacity of the column can be assumed to be uniform. Determining this height was one of the reasons for undertaking an experimental study on gravity-load columns subjected to deformation demands.

5. EXPERIMENTAL STUDY

Experiments are currently being conducted on a series of four half-scale gravity-load columns (Chin, 2012). The columns all have approximately the same cross sectional area of concrete, they all have 8 – 15M vertical reinforcing bars and are all subjected to an axial compression of 1500 kN. Figure 6 shows the cross section of the four columns. The purpose of the study is determine how the cross sectional geometry influences the failure mode and drift capacities of the columns.

To date, two of the columns have been tested – the 275 x 550 mm and 200 x 800 mm columns; however the data from the second test is still be analysed and therefore only the detailed results from the first test are reported here.

All specimens were approximately 2 m tall including the bases which were 35 cm high. The distance from the top of the base to the elevation of the horizontal load applied on the columns to impose bending deformations was 141 cm. The height from the lateral load to the underside of the base was 176 cm (see Fig. 6b). All specimens were cast in the up-right position. The bases were cast first using concrete with a specified cylinder compression strength of 50 MPa in order to reduce the deformation

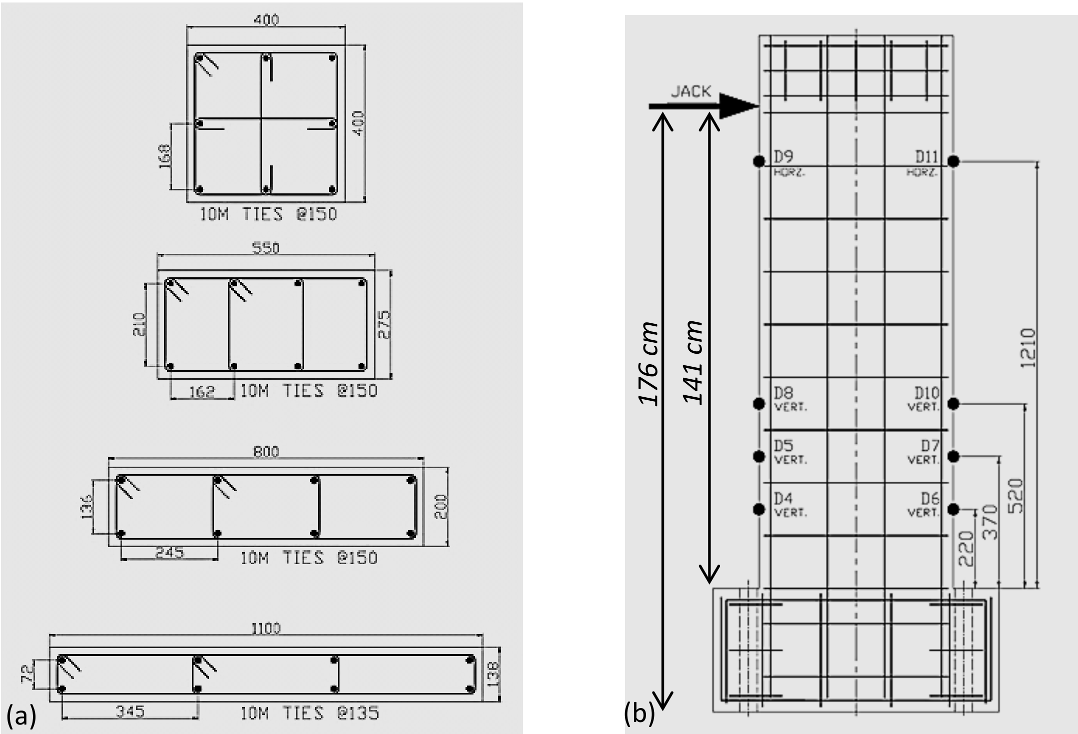

Figure 6. Experimental study on gravity-load columns: (a) cross sections of four specimens, (b) elevation of one specimen showing location of displacement transducers.

of the bases. The columns themselves were cast using concrete that has a 28-day cylinder compression strength of 30 MPa. The specimens were tested about one year after casting and at that time the concrete had gained only about 10% additional compression strength.

Figure 5(b) shows the predicted load-deformation relationship of the 275 x 550 mm specimen assuming the base is perfectly rigid (line labelled H = 141 cm in Fig. 5b). Figure 7 presents the measured load-deformation relationships. Figure 7(a) shows the full hysteresis loops, which are pinched because of the high axial compression, while Fig. 7(b) shows the envelope of the measured response. The prediction from Fig. 5(b) is also shown in Fig. 7(b).

The test results indicate the initial stiffness of the specimen is about half the predicted value. Measurements indicate that significant slip occurred in the column base. For example, when the specimen lateral displacement at 1210 mm from the top of the base (see Fig. 6b) was 28 mm, the vertical uplift at the top of the base (tension side of specimen) was 2.8 mm. Based on the measured strain profile at the base, the location of zero vertical strain is estimated to be at 269 mm from the compression face. Thus the horizontal rotation of the column due to the uplift is estimated to be 13.4 mm. The remaining 28 – 13.4 = 14.6 mm horizontal displacement is due to deformations of the column above the base.

The prediction in Fig. 5(b) was recalculated assuming the height of the specimen is 176 cm, which is the distance from the underside of the base to the point of horizontal load application (see Fig. 6b). The initial linear slope of this loading curve shown in Fig. 5(b) as a dotted line is also shown in Fig. 7(b) as a dotted line. This curve gives a much better prediction of the measured initial load – deformation response.

Using the nonlinear bending moment – curvature relationship shown in Fig. 5(a), the specimen is predicted to have a displacement capacity of only 5 mm. As described previously, this is because the nonlinear bending moment curvature relationship results in a small zone of concentrated maximum curvature at the base of the column. The actual displacement capacity of the test specimen above the

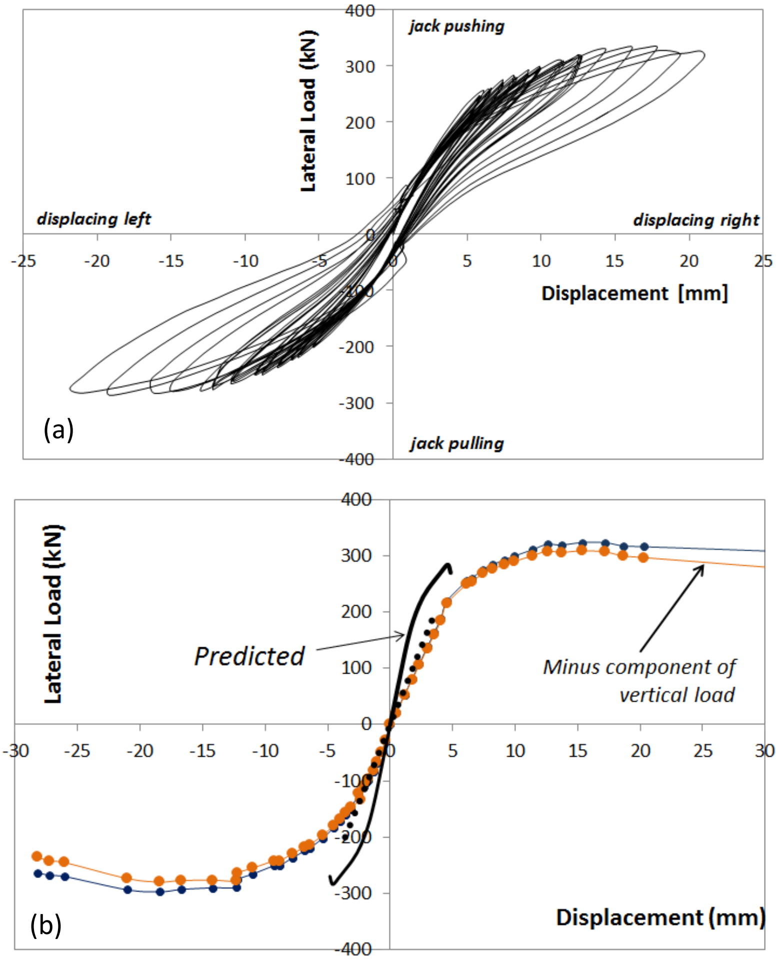

Figure 7. Results from 275 x 550 column test: (a) load-deformation hysteresis loops, (b) comparison of predicted and measured load-deformation envelopes.

base was larger than 15 mm – the test was stopped because of limitations in the test setup. There are two reasons for the very large displacement capacity. The first is the large distance over which the cover spalled and the damage spread. As shown in Fig. 8, the height of damage – cover spalling, diagonal cracking, yielding of vertical reinforcement, i.e., the "plastic hinge" height – was three full ties spacing, which is 450 mm.

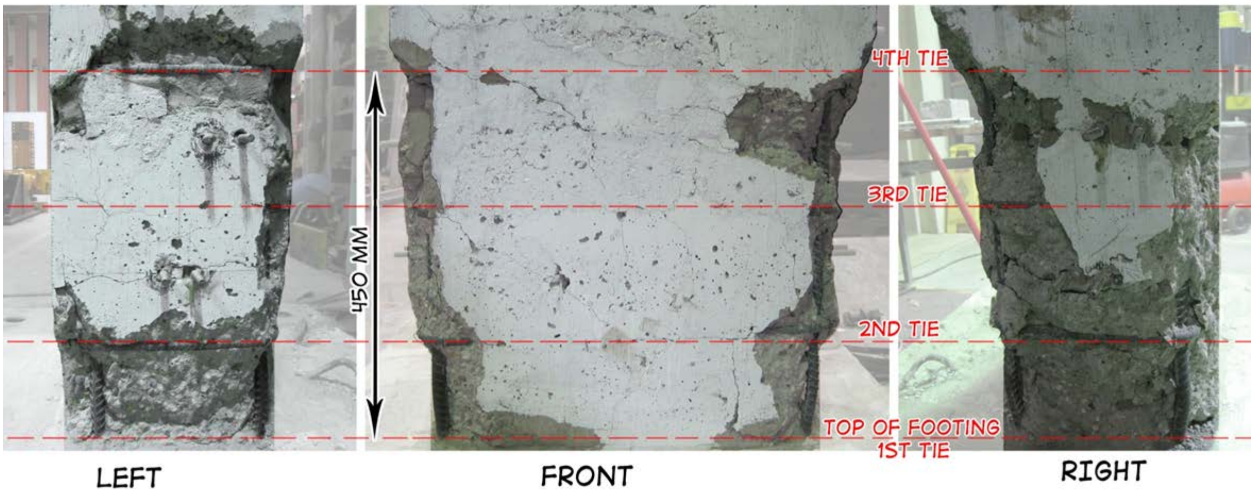

Figure 8. Photographs of 275 x 550 mm column after the test showing height of damaged zone.

The second reason the specimen performed so well was because of the tie arrangement. The specimen cross section had essentially three separate "cores" of concrete within column ties. Each of these is 162 x 275 mm in dimension (see Fig. 6a). During the test, the outer two "cores" of concrete crushed; but the central "core" remained intact and this concrete was sufficient to support the 1500 kN axial load. As a result, the column did not collapse even though the concrete in the two outer "cores" was pushed well beyond the code limit of 0.0035.

A very different result occurred during the recent testing of the 200 x 800 mm gravity-load column. The column failed in a brittle manner because the concrete within the central "core" of the column failed. The reason is that after the cover spalled on the two sides of the column, the remaining thickness of the "core" was not sufficient for the concrete to be stabilized by the rectangular column ties. The failure mode was very similar to recent failures observed in thin concrete wall elements (Adebar and Lorzadeh, 2012).

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