INELASTIC SEISMIC SHEAR IN MULTI-STOREY CANTILEVER COLUMNS

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Abstract. Seismic shear force magnification in the columns of multi-storey precast structures entering far into inelastic domain is addressed in the paper. Such structures consist of an assemblage of cantilever columns connected with ties. Considering analogy with cantilever walls, it has been expected that during inelastic response the actual shear forces in multi-storey cantilever columns could be considerably higher than the forces foreseen by traditional equivalent elastic analytical procedures (equivalent lateral force or modal spectrum).

A parametric study of a set of realistic three-storey structures/columns was performed. These structures were designed according to Eurocodes and shear forces were determined by the equivalent elastic (modal spectrum) analysis. Average values of the shear forces obtained by the inelastic response analysis were compared to those of the traditional (modal spectrum) procedure. They were also compared to magnified shear forces predicted using the shear magnification factor ε suggested in Eurocode 8 for RC ductile walls. In parallel, modelling issues related to the inelastic response analysis of multi-storey cantilever columns were also discussed.

Very large seismic shear force magnifications (up to 3 times and more) were observed. Therefore it is essential to account for this phenomenon in the seismic (capacity) design of cantilever columns in multi-storey precast buildings. It was demonstrated that for this purpose the expression given in Eurocode 8 to account for seismic shear force magnification in ductile cantilever walls could be used with some minor modifications.

1   INTRODUCTION

Precast buildings house a predominant share of industrial facilities in many European countries. The most common precast system in Europe has been using dowel joints providing hinged connection between the beams and columns. Therefore such structures behave essentially as an assemblage of cantilever columns with ties. Up to now many precast industrial buildings have been one-storey structures. In such structures the capacity design for shear in the columns and joints is straightforward. However, recently the multi-storey structures are emerging as competitive market potential in the reinforced concrete construction sector. Question arises how to perform seismic (shear) design of multi-storey cantilever columns entering far into inelastic domain. Indeed, considering analogy with cantilever walls [1 – 6] and the results of the PRECAST [7] and SAFECAST 7th EU Framework research projects, a considerable dynamic magnification of shear forces during inelastic response has been expected. The main reasons for the shear magnification are the following:

- Overstrength: a consideration of simple equilibrium shows that the design seismic shear forces increase proportionally to the flexural overstrength.
- Period shift: Due to the softening of the structure in the inelastic range, the first mode spectral acceleration value typically diminishes, whereas the spectrum values for the higher modes (having the natural period at least 6 times lower than that of the first mode) usually remain in the plateau of the spectrum. The relative influence of the higher modes therefore increases in the inelastic range.
- Amplified influence of higher modes: The first mode seismic forces contribute most of the overall seismic moment at the base of the cantilever, which is limited by its flexural resistance. Energy dissipation is therefore predominantly limited to the flexural response in the first mode. Consequently, the first mode shear forces are reduced due to the energy dissipating mechanism, whereas the shear forces due to the higher modes are not. This significantly increases the relative contribution of the higher modes to the shear force which occurs during the inelastic response.

The increased relative importance of higher modes lowers the position of the resultant of the seismic forces closer to the base of the wall (Figure 1). With the given bending moment at the base, which is equal to the flexural capacity of the column, it is obvious that the resultant seismic force (shear force) should increase.
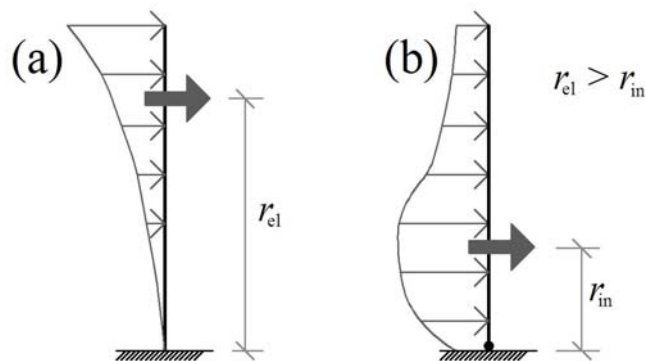

Figure 1 - Higher modes effects on the distribution of lateral forces and base shear

Since in the present design practice this increase in shear forces is frequently not considered properly, there is a danger of brittle shear failure, both in the column and in the connections at each storey.

The capacity design for shear in single storey cantilever columns is straightforward. Since there is no influence of the higher modes, only overstrength has to be considered. The maximum expected shear force is therefore simply obtained by dividing flexural moment capacity at the base by the height of the column. However, in the case of multi-storey cantilevers, the distribution of the moment over the height of the column is not known and it also changes during the response (see Figure 2(a) and 2(b)).
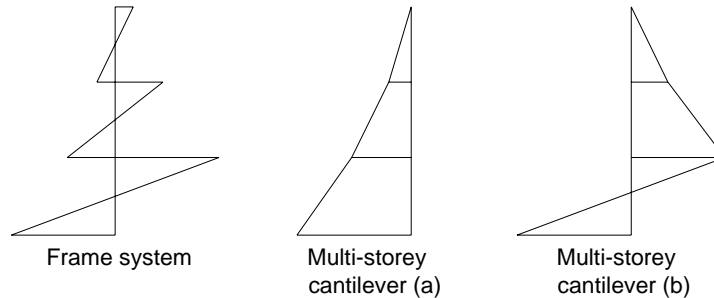

Figure 2 – Typical moment diagram shape in frame and cantilever multi-storey column

So designers are facing serious problem how to perform the capacity design of columns and connections required by Eurocode 8 [8] in order to preclude the brittle failure of these key components of the structural system. Contrary to the case of RC structural walls, the problem of the shear magnification in the multi-storey cantilever columns in RC precast structures has not been explicitly addressed in the current version of the Eurocode 8. It has been believed by the authors that appropriately modified shear magnification factors defined in Eurocode for structural walls could be also used for multi-storey cantilever columns in precast buildings.

To address the problems mentioned in the previous paragraph, a parametric study of the inelastic response of realistic three-storey cantilevered multi–storey precast buildings, designed according to Eurocode 8, was made [9]. The configuration of the building was defined by the three-storey building prototype which is being pseudo-dynamically tested in the European Laboratory for Structural Assessment in Ispra within the frame of the SAFECAST research project (coordinated by the Association of the precast producers in Italy ASSOBETON). Shear forces obtained during the inelastic response were compared to those predicted by traditional equivalent elastic design procedures to evaluate the expected shear magnification. Finally the shear magnification factor (ε) used in Eurocode for shear walls was tested in the case of multi-storey cantilever columns in precast structures.

2 MODELLING ISSUES

As shown before (see cases "a" and "b" in Figure 2), moment diagram shape can vary considerably during the response of the analysed structural systems. Since in lumped plasticity models it is assumed that the moment distribution along the element does not change during the time, the use of only one lumped plasticity element per storey is precluded or at least questionable. Several models were studied to overcome this problem. Two relatively simple models could yield appropriate solution. One is using several lumped plasticity elements per storey (Figure 3) and the other is based on the fiber approach.
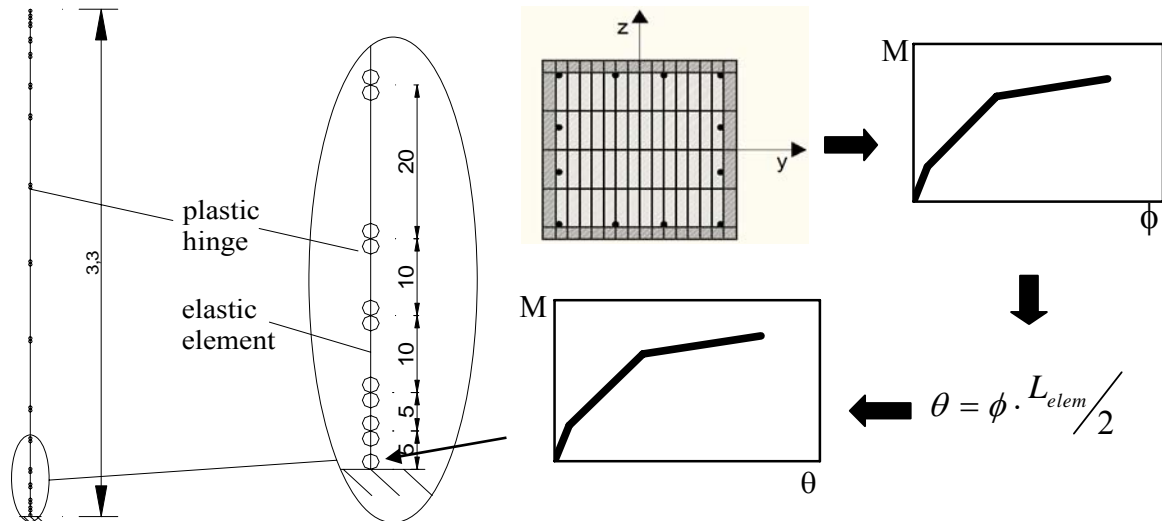
Figure 3 – Analytical model with several lumped plasticity elements per storey (bending in x – y plane is considered only)

Both models were extensively tested to check their efficiency and first of all their numerical stability. Finally the model with several short lumped plasticity elements per storey was chosen. This model did not exhibit larger problems with numerical stability opposed to the fiber element which was strongly dependent on the number of the integration points and the influence of the second order theory effects. In addition to the pure flexural behaviour (described by both models) some other phenomena (like slip of the reinforcement and deformability due to the shear cracking) can be approximately (empirically) included into the hysteretic rules used in the lumped plasticity model.

3 PARAMETRIC STUDY

3.1 Description of the analyzed structures

The actual shear magnification factors were determined by inelastic response analyses on five three–storey cantilevered structures, typical for the construction practice in Europe. The height of the typical storey was $3.2m$.

Due to the hinged beam-column connections and assuming floors being rigid in their own plane, buildings were modelled as single multi-storey columns. To each of the five buildings/columns different value of the normalized axial force v_d $(0{,}05 \le v_d \le 0{,}20)$ was assigned to reflect actual spans and loads used in practice (Table 1). Only the highest value (0,20) tends to be unrealistic due to the drift limitations. However it was included in the research just to study the general trends of the results. Square section $80 \times 80cm$ was chosen for the column, while masses (assumed to be the same at each floor) and fundamental periods depended on the level of normalized axial forces.

The buildings were designed according to Eurocode 8, using standard design procedures based on the results of the equivalent elastic spectrum modal analysis ($a_{g,\max} = 0.25g$ and Soil Type B) considering one half of the inertia characteristics of the uncracked sections. The same reduction as for DCH cast-in-situ frames $(q = 4.5)$ was assumed [10]. Concrete C45/55 and steel B450C was used in the design.

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ν_d [-]	m [t]	A_{inf} [m^2]
0.05	32.6	45.7
0.075	48.9	68.6
0.10	65.2	91.4
0.125	81.6	114.3
0.20	130.5	182.9

Table 1 – Normalized axial force, floor mass and influence area (assuming distributed storey weight of w=7kN/m^2) of the five analyzed buildings

The response history analyses were performed using OpenSees [11] with a set of 9 accelerograms, matching the EC8 spectrum for $a_g = 0.25g$ and Soil Type B (Figure 5). These accelerograms were obtained by the modification of the actual accelerograms recorded in Europe. Again the mean spectrum of these recorded accelerograms matched the EC8 spectrum (Figure 4). Five percent mass and current stiffness proportional Rayleigh damping was considered in the first and second modes.
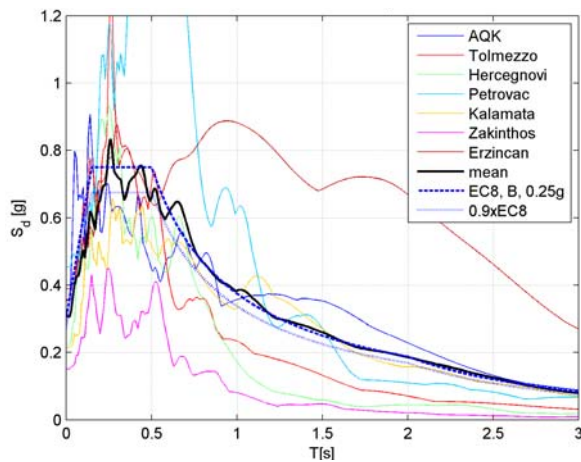

Figure 4 - Spectra of the recorded accelerograms (note that the mean spectrum matches the EC8 spectrum very well)
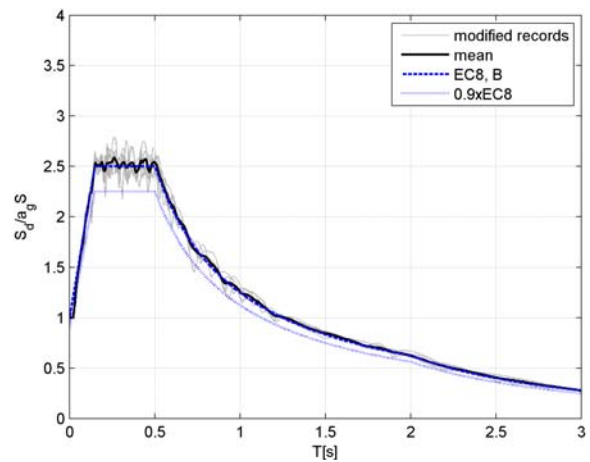
Figure 5 – Normalized spectra of the modified accelerograms matching EC8 – Soil B spectrum

3.2 Results for the shear magnification factors

Figure 6 shows the shear magnification factor (the ratio between the shear forces obtained by the inelastic analyses and those obtained by the equivalent elastic spectrum modal analysis) for the five investigated structures, which are denoted using the corresponding normalized axial force value ν_d.

For each structure, three different assumptions regarding stiffness of the columns and overstrength were considered in the inelastic response analyses. In Figure 6 the circles denote results of the model based on the actual stiffness during response (model 1). Squares indicate the results obtained with the inelastic analysis using the bilinear model having the same initial stiffness as it had been used in the design – model 2 (one half of the inertia characteristics based on the uncracked section were used). Model 3 (triangles) is basically the same as Model 2, except for the overstrength, which is not considered.
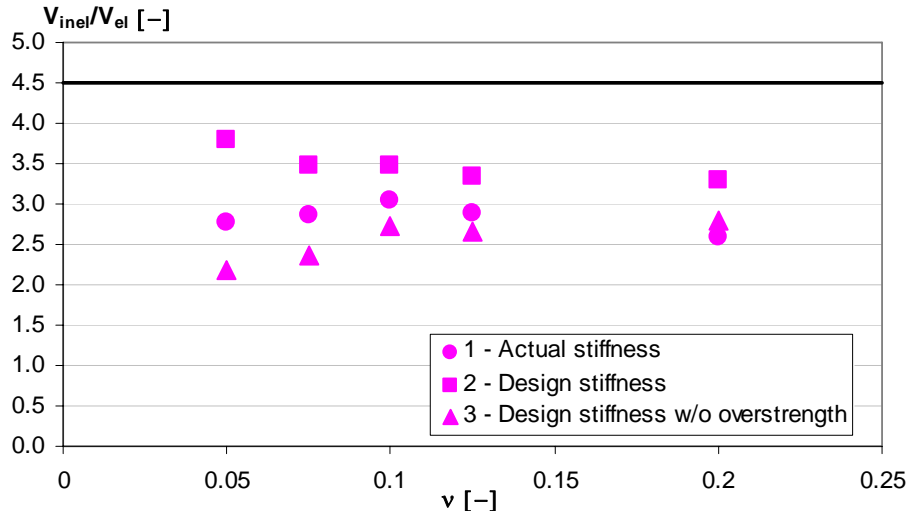
Figure 6 - Shear magnification ratios evaluation for the five analyzed structures using different stiffness/overstrength models

As clearly shown in Figure 6, the actual shear forces induced during the inelastic response of multi-storey columns in typical precast industrial buildings are much higher (2 to 4 times) of those predicted with equivalent elastic procedures commonly used by the design engineers! So the shear forces provided by standard computer design programs, using the reduced seismic forces, grossly underestimate actual shear demand if they are not appropriately corrected. Fortunately in these slender columns the absolute value of the shear forces predicted by traditional methods is typically quite low. However, when multiplied with a factor up to 4 to obtain the actual level of shear forces, it becomes clear that this problem calls for serious attention. Therefore, to avoid brittle failure of the columns and/or in the beam-to-column connections it is essential to account for this phenomenon in the capacity design of cantilever columns in multi–storey precast buildings.

It can be further observed that:
- The shear force magnification is much larger if the same initial stiffness as in the case of design (one half of that obtained for the uncracked gross section – squares in Figure 6) is used also in the inelastic analysis. Considering the actual/realistic stiffness degradation due to cracking (up to 4 times in the case of elements with small to moderate compressive axial force) the shear increase is smaller (circles in Figure 6).
- If the effect of overstrength is eliminated (Model 3; triangles in the Figure 6) the magnification is still high (over 2). This demonstrates that, at least in the case of the analyzed buildings, the effect of higher modes on the shear magnification is predominant.

4   PROPOSAL OF THE DESIGN SHEAR MAGNIFICATION FACTOR

Eurocodes [8] already provide a shear magnification factor ε for RC ductile walls, while for multi-storey precast structures this problem is not properly addressed in the current codes as well as in the literature.

The ε factor for walls is used in Eurocode to multiply the values obtained by the linear-elastic lateral force or modal response spectrum analysis. For walls which enter far into the inelastic range (ductility class "high"), large shear magnifications are expected, and the factor ε should be calculated using the expression proposed by Keintzel [2], which explicitly takes into account the effects of higher modes in the inelastic range as well as flexural overstrength:

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0.1 \cdot \left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \begin{cases} \leq q \\ \geq 1.5 \end{cases}$$

(1)

where:

q is the behaviour (seismic force reduction) factor used in the design;

M_{Ed} is the design bending moment at the base of the wall;

M_{Rd} is the design flexural resistance at the base of the wall;

γ_{Rd} is the model uncertainty factor on design value of resistances accounting for various sources of overstrength;

T_1 is the fundamental period of vibration of the building in the direction of shear forces;

T_C is the upper limit period of the constant spectral acceleration region of the spectrum;

$S_e(T)$ is the ordinate of the elastic response spectrum.

Although Keintzel's expression for the shear magnification factor in RC structural walls has been based on a series of quite crude assumptions and approximations, like superposition principles in inelastic range, it was demonstrated by numerical tests that it has been working fine [6, 12]. It has been believed by the authors that a similar modified expression for the ε factor could be used also in the case of RC multi-storey columns in precast structures.

In the study, the values of the seismic design shear forces (denoted as V_{Ed}), evaluated by applying ε to the base shear resulting from the SRSS analysis, are compared with those obtained by inelastic response history analysis (denoted as V_{an}). The ratios V_{Ed}/V_{an} are illustrated in Figure 7.

If ε is applied to the base shear determined by the contribution of the first mode only (white dots in Figure 7), as originally intended by Keintzel, the magnification is very well estimated (note that $v_d = 0,20$ is not realistic in practical design). The proposed factor is slightly conservative, which makes it suitable for design purposes.

Most computer programs would, however, output the total base shear based on the contribution of all the considered modes. It is therefore most likely that designers will tend to apply ε to the total shear forces. This would be somewhat (up to 1,5 times) conservative (see black dots in Figure 7). An alternative solution is to propose a modified magnification factor, which is still under investigation.

It should be also noted that:

- The equivalent elastic shear forces were calculated based on the inertia of the cracked concrete sections equal to one half of the uncracked gross section. In this way the actual stiffness of the typical precast columns is overestimated. Therefore if the designer opts to use realistic (calculated) yielding stiffness in design, the proposed ε factor may be unconservative.
- It is suggested in Eurocode 8 that the overstrength factor $\gamma_{Rd} = 1.2$ should be used in Keintzel's formula for structural walls. In the presented study the actual overstrength factor for the analysed columns was closer to 1,3. If considered, the proposed factor will be slightly more conservative.
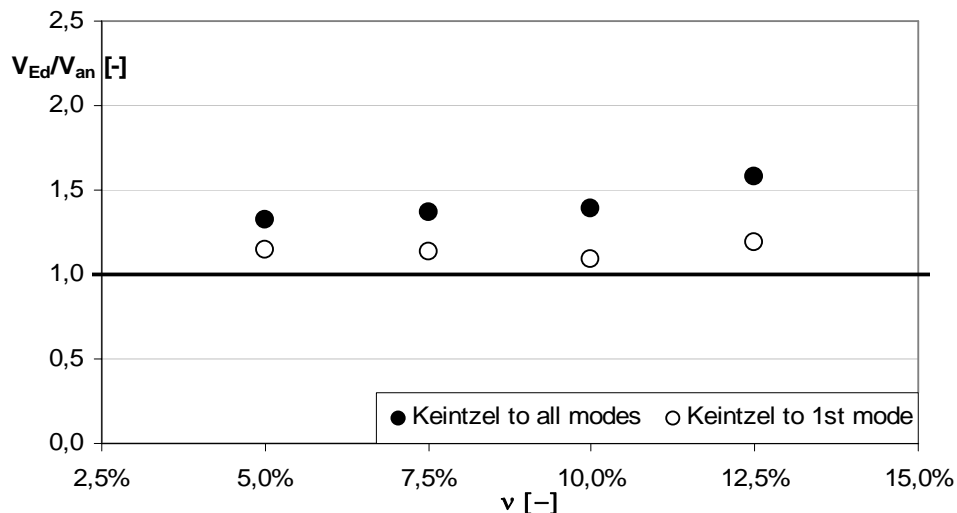
Figure 7 – Design base shear evaluated according to Keintzel's factor compared to actual base shear resulting from dynamic analysis

5 SELECTED IMPORTANT RESULTS OF THE RELEVANT AND/OR ONGOING RESEARCH

Some other relevant conclusions of the research, which are not discussed in detail in this paper include (see [9] for additional information):

- A short study on a hypothetical 10-storey building suggested that the same base shear magnification factor could be used also for buildings higher than 3-storeys.
- ε factor was originally proposed for the base shear and all the above conclusions and discussions apply for the base shear. However, it was demonstrated (though by the study, which was limited in scope) that the same factor could be used over the entire height of the column (at least in the case of 3-storey buildings).
- Local shear magnification can be very critical for the design of the dowel beam-to-column connections. In particular in the first storey the design force on the connection could be underestimated up to 10-times if proper magnification was not considered.

6 CONCLUSIONS

The actual shear forces in multi storey cantilevered structures due to seismic loads are considerably higher than the forces foreseen by the equivalent linear-elastic lateral force analysis, or by the modal response spectrum analysis specified in the codes. Simply said, this magnification occurs due to flexural overstrength and the amplified effect of the higher modes in the inelastic range.

In this study five realistic three–storey cantilevered structures, typical for the construction practice in Europe and designed according to Eurocode provisions were tested. The actual shear magnifications were determined by the inelastic response analyses as well as compared with the values obtained by the EC8 (Keintzel's) shear magnification factor, valid for RC structural walls.

Two main conclusions have been made:

1) It has been demonstrated that the actual shear forces induced during the inelastic response of multi-storey cantilever columns in typical precast industrial buildings are much higher (2 to 4 times) of those predicted with equivalent elastic procedures commonly used by the design engineers. So the shear forces provided by standard computer design programs using the reduced seismic forces grossly underestimate actual

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shear demand if they are not appropriately corrected. Therefore to avoid brittle failure of the columns and/or in the connections, it is essential to account for this phenomenon in the capacity design of the cantilever columns in multi–storey precast buildings. However, neither Eurocodes, nor national seismic codes explicitly require appropriate seismic shear magnification for such columns. Related capacity design procedures are also not defined.

2) It has been demonstrated that the similar shear magnification factor as proposed in Eurocode 8 for ductile (DCH) RC structural walls can be used also in the case of multi-storey cantilever columns in precast buildings:

$$\varepsilon = q \cdot \sqrt{\left(\frac{\gamma_{Rd}}{q} \cdot \frac{M_{Rd}}{M_{Ed}}\right)^2 + 0.1 \cdot \left(\frac{S_e(T_C)}{S_e(T_1)}\right)^2} \begin{cases} \leq q \\ \geq 1.5 \end{cases} \tag{2}$$

This factor primarily depends on (a) the flexural overstrength and (b) the magnified contribution of the higher modes during inelastic response.

The above results strictly apply for the base of the analyzed 3-storey structures. However, in parallel research the conclusions were generalized also to higher buildings as well as for the shear forces along the entire height of the column.

ACKNOWLEDGEMENT

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