Characteristics Affecting the Vulnerability of Soft Storey Mechanisms

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SUMMARY:
The seismic vulnerability of a six storey RC frame building typical of Italian construction practice from the 1970's is examined in this work, initially considering two different infill configurations; the first considers full masonry infill and the second considers an open ground storey with full masonry infill above the first floor. Results of incremental dynamic analyses indicate that structures with uniform infill are less likely to collapse. However, it is also recognized that the need to retrofit structures with partial infill should depend on the seismic hazard and acceptable risk. Therefore, to provide insight into the factors affecting the vulnerability of such RC frame structures, the impact of P-Delta effects, post-yield stiffness ratio and ground storey column characteristics are investigated. Results are discussed and because p-delta effects and column axial load ratios are found to affect the vulnerability significantly, an innovative retrofit strategy that considers these characteristics is proposed.

Keywords: Soft-storey, Vulnerability, P-delta, post yield stiffness, concrete frame retrofit

1. INTRODUCTION

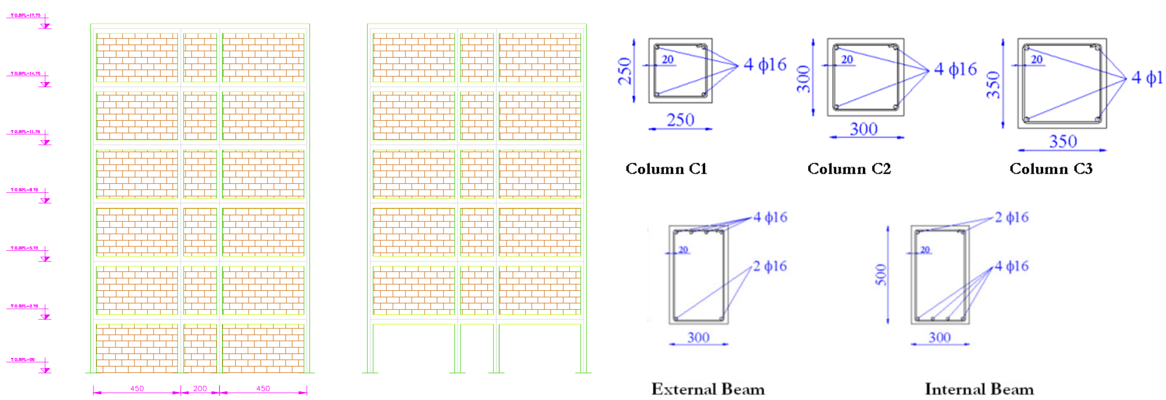
Soft storey behaviour has been recognized as one of the most undesirable mechanisms among the structural and earthquake engineering society. After in the large 1971 San Fernando (M_w=6.6) and 1978 Miyagi-ken Oki (M_s=7.7) earthquakes, which caused serious damages or collapse of first floor structural elements, a large amount of attention was directed to soft story structures, also called pilotis structures (Rutenberg et al., 1982, Chopra et al., 1973, Arnold, 1984). Such research led to the development of design procedures that do not allow column side-sway mechanisms, and a series of firm decisions was taken to prevent engineers from designing such kinds of structures (Esteva, 1992, Naeim, 1989, Vukazich et al., 2006). In New Zealand the benefits of designing for a weak-beam strong-column mechanism had already been recognised in the seventies (Park and Paulay, 1975). Most of these studies emphasized the fact that because deformation demands concentrate at the first storey, the column elements reach their ultimate capacity at considerably lower levels of intensity than mechanisms that involve distributed deformation demands.

Even though current design practice aims to avoid the use of soft-storeys, there are a large number of older existing buildings that could be expected to develop soft-storeys. Engineers will therefore be asked to decide when retrofit is required for such structures and what retrofit strategy is best to adopt. This paper therefore examines the effect of different structural characteristics on the seismic response of structures with soft storey mechanisms. Some initial case study structures are taken as benchmark buildings and incremental non-linear time history analyses are repeated varying some key parameters including the examination of P-Delta effects and the post-yield stiffness ratio of first storey columns. The last part of the paper will explore the factors affecting the capacity of ground storey columns and, in light of all the results obtained, will discuss a retrofit strategy for soft-storey structures.

2. DESCRIPTION OF CASE STUDY BUILDING AND ANALYSIS APPRAOCH

The case study structure selected for this work is a six storey three bay concrete frame, which is studied for two different distributions of masonry infills. In the first scenario, it is assumed that masonry infills are distributed over all stories uniformly, while in the next step and for considering soft storey effects, it is assumed that masonry infills are omitted in the first floor. The frame configurations are taken from Galli (2006) and is representative of typical Italian buildings designed during the 1950s to the 1970s. Accordingly, structural elements are designed just for gravity loads without following any rules of capacity design for seismic protection. As such, a strong column-weak beam (SCWB) mechanism does not develop for this case study.

The structure of Fig.1.1 is part of a building formed by a series of parallel frames at a distance of 4.5 m between centrelines of columns. A first floor height is 2.75 m, while other floors have the same height of 3m. The frame consists of two equal exterior bays of 4.5 m length and one interior span with a length of 2 m. The frame is therefore symmetric about the vertical axis. Figure 2.1 shows a schematic view of the frame for the two variants of full infill and partial infill, as well as section configurations.



(Variant 1) **(Variant 2)**

Figure 2.1. Six-storey concrete frame, variant 1) full infill uniform distribution, variant 2) Partial infill disconnected in the first floor

The time history analysis is carried out using ten recorded horizontal accelerograms selected as part of the DISTEEL project (Maley et al., 2012). The record set consists of 10 records that are scaled to be compatible with Eurocode 8 spectrum (CEN, 2004) for soil type C and a corner period Td = 8s. Acceleration and displacement response spectra the records are shown in Figure 2.2. It should be noted that nine different hazards levels are used for each set of records with the following peak ground accelerations for each intensity level: 0.05g, 0.10g, 0.15g, 0.20g, 0.25g, 0.30g, 0.35g, 0.40g, and 0.60g. Thus, a total of 90 ground motions are used for the nonlinear time history analyses.

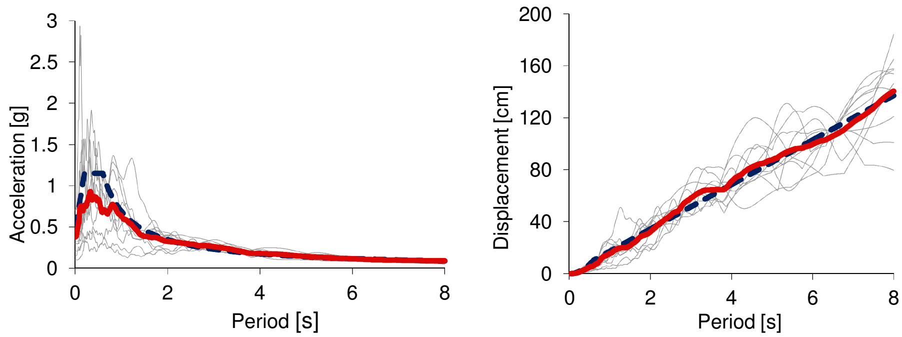


Figure 2.2. Acceleration and displacement Response Spectra for the selected records sets

The inelastic dynamic analysis program RUAUMOKO (Carr, 2004) is used for the numerical analyses. This program utilises a lumped plasticity approach, and contains several types of moment-curvature hysteretic rules for definition of plastic hinges in the elements and joints. A two-dimensional non-linear element is used for modelling the beams and columns. To this end, a plastic hinge is concentrated at the two ends of each element. RUAUMOKO contains several types of moment-curvature hysteretic rules for definition of plastic hinges in the elements and joints. Among them, the bilinear Takeda hysteresis rule (Otani, 1974) is used, where unloading stiffness is changed as a function of ductility and post yield stiffness ratio. The Emori and Schonbrich (1978)model is used in order to obtain the unloading stiffness. Due to the fact that the structure is designed for gravity loads only, the hysteresis shape should be defined so that relatively low levels of energy dissipation occur. For this reason the parameters α and β of the Takeda model (Carr, 2004) are chosen to be 0.5 and 0.0 respectively, instead of larger factors that are traditionally used for new RC frames. Future research could investigate the impact of using more refined hysteretic models.

A widely used approach is adopted for the modeling of masonry infills, based on the use of axial springs acting as equivalent compression diagonal struts. The stiffness of the equivalent diagonal strut is evaluated according to the Stafford (1969) model modified as proposed by Bertoldi (1993).

The beam-column joints are modelled with a couple of rotational and axial springs based on a simple model proposed by Trowland (2003). In the model, the spring is split into two elements that are interposed between the beam end nodes and the upper and lower column nodes respectively. The upper and lower column ends are slaved to move together in lateral translation and rotation. The advantage of this model, that extends on the model of Pampanin et al. (2002) is considered to be that that effect of axial load on joint resistance is also included.

3. EFFECT OF P-Δ

This section presents the results of numerical analyses of the six-storey frame with and without the inclusion of P-Δ effects. To this end, incremental time history analyses of the structure with full and partial infill are repeated for the case that large displacements are considered in nonlinear deformation capacity of the structure. These results are compared with the results obtained using small displacement analyses. Figure 3.1 shows the structural responses with and without considering P-Δ. It should be noted that the stability index versus intensity is also plotted for the four aforementioned cases. By definition, the stability index is obtained from Equation:

$$\theta_i = \frac{P_i.\delta_i}{V_i.H_i} \tag{3.1}$$

Where, P_i and, V_i are respectively, the portion of total gravity load and the total calculated shear storey at level i. δ_i and, H_i are lateral storey drift (m) and height of the floor at level i, respectively.

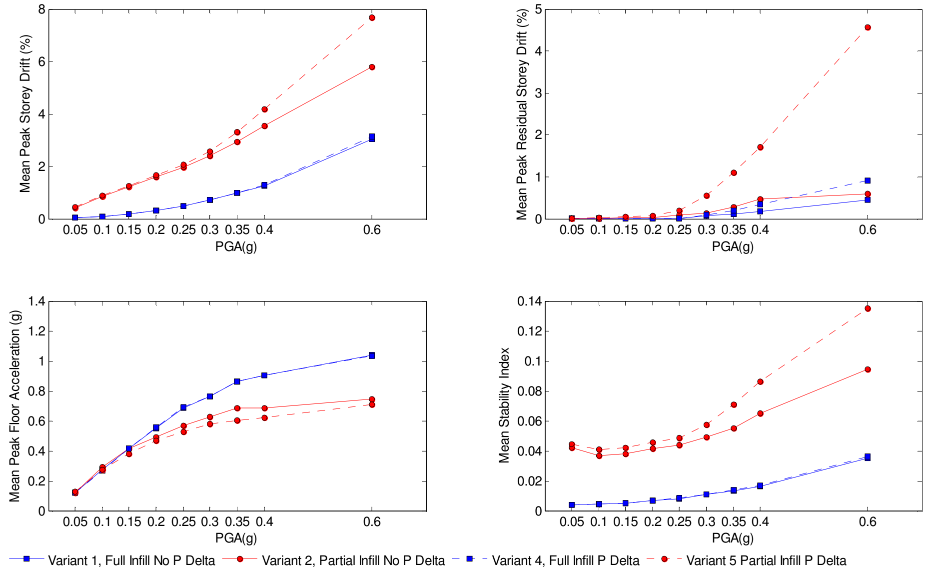


Figure 3.1. Comparison of responses obtained from incremental NTHA with and without P-Δ effects

For the case of the full versus the partial infill frames, it is apparent from Figure 3.1 that the partial infill frame registers significantly larger peak and residual storey drift demands than the full infill case but with smaller floor accelerations.

Considering the effect of P-Δ, the responses are almost the same departing for the full infill case with or without P-Δ effects and this is arguably a reflection of the low stability indices for this structure. The impact of P-Δ is more significant for the soft storey case where difference between peak- inter-storey drifts increase with intensity level. The drift ratios at the hazard level of 0.6g are respectively, 7.5% and 5.9% with and without p-delta effects. On the other hand, the effect of P-Δ on residual drift is much more sensitive to the increasing level of intensity. It can be seen that the residual drift at a PGA of 0.6g is less than 1% without P-Δ, while P-Δ amplifies this value by a factor of around 5.

The stability index is an interesting parameter for the case of soft storey structures. It can be seen that the index has a fluctuating or even reducing trend for some low intensity levels, but that it increases linearly after medium levels of intensity. The fluctuation can be explained by the fact that the rate of increase in inter storey displacement is not as high as shear forces at low intensity levels. This rate can be gauged from the floor acceleration figure, where it is seen that accelerations increase rapidly up to medium level, which leads to a reduction in the ratio δ_i / V_i. While, for high intensity levels the rate of stability index follows inter storey drift graph, as floor accelerations will remain approximately constant once a storey mechanism has formed.

3.1. Effect of Increased P-Delta Load

In the next step, the effect of an increased P-delta load on the frame response will be explored. The greater P-delta scenario could occur for buildings with lateral load resisting frames in external bays and gravity framing internally, when diaphragms are rigid and can transform lateral loads from middle spans to external ones. In this case, vertical loads due to P-Δ effects will be increased, while gravity loads on the columns will not be changed. Thus, to consider the effect of increasing P-delta axial load without changing the gravity load on the case study column elements, dummy columns are added to the structure and extra vertical loads are assigned to them. It should be noted that care is required in order to ensure that dummy columns do not add any stiffness to the structure. For this reason, dummy columns are transversally slaved to a structural element in each level, while other degrees of freedom (including end rotations) are released.

Results obtained from the case of two times the total gravity load (i.e. two times the P-Δ coefficients) are shown in Figure 3.2. In this case, the structure was unstable for intensities greater than PGA=0.3g, with 90% of records causing dynamic instability at a PGA=0.6g, and thus, drift response is shown up to an intensity level of 0.4g.

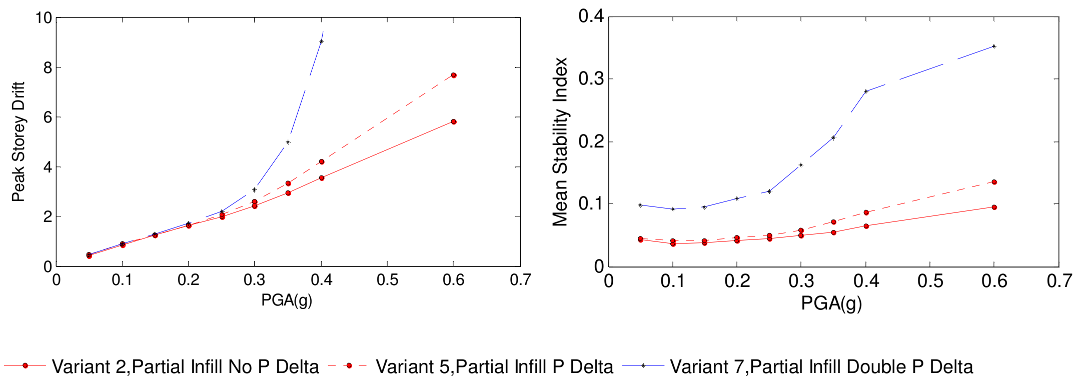


Figure 3.2. Comparison of responses obtained from incremental NTHA when the total gravity load is doubled

The results of this section indicate that vertical loads can have significant effects on the response. From Figure 3.2 it can be seen that for the case of double the gravity load, peak storey drifts are more than two times the case without P-delta effects, which indicates that the effect of axial load is important and should be considered carefully in vulnerability assessments.

4. EFFECT OF POST YIELD STIFFNESS

In this section, the effect of the post-yield stiffness ratio on the response trend lines is explored. It should be mentioned that the post yield moment-curvature stiffness, R, was taken as 2.5% (R=0.025) of the initial stiffness for the analyses reported in the previous section. To this extent, the partial infill building considering P-Δ effects (variant 4), is re-run with different R-values of R= 0.05, 0.07,0.10 and 0.15 and are called cases 6, 7, 8 and 9 respectively. Figure 5 presents the results obtained.

The incremental time history results indicate that the post yield ratio can affect responses at higher intensity ranges. Residual displacements have been significantly reduced by increasing the R value for medium intensity level, while they remained almost the same for low level. Generally, increasing R ratio decreases residual displacement, and as it can be seen for a PGA=0.4g, residual drifts have decreased 50% from 1.0% to 0.5%. This result agrees with previous research on the subject of residual drifts such as that reported by Pettinga et al. (2007). However, and surprisingly, a large post yield ratio produced high residual drifts for high levels of deformation demand. It can be seen that residual deformations were decreased from R=0.025 to R=0.75, and suddenly increased for values of R= 0.10 and 0.15. This could be due to a change in the dynamic response characteristics attracting larger demands for the higher R values but should be investigated further as part of future research. Overall, one could argue that the post-yield stiffness does affect the vulnerability but not greatly, with reduced drifts but increased accelerations, in the order of 10-20% depending on the seismic intensity.

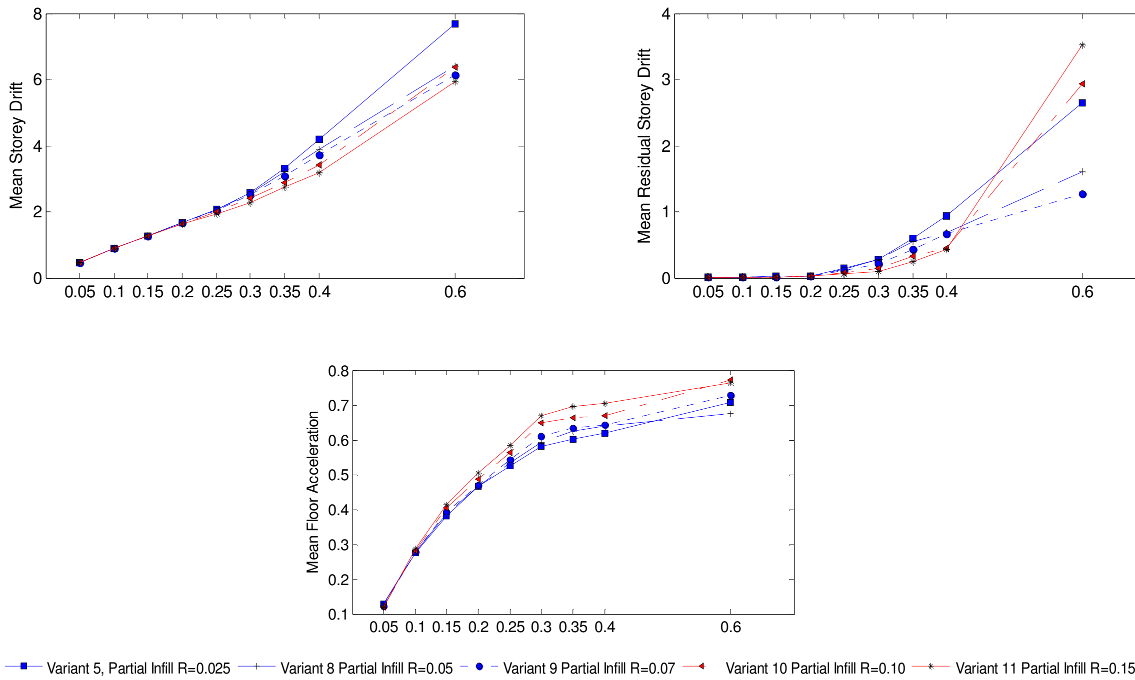


Figure 4.1. Effect of post yield ratio of responses

5. INFLUENCE OF COLUMN CHARACERTISTICS ON SOFT-STOREY CAPACITY

Another aspect that is considered in this study of "key characteristics" is the impact of different column characteristics on the frame capacity. To do this, cyclic analyses are conducted for the columns shown in Table 5.1, considering different section depths, different axial load ratio and different reinforcement contents in order to identify the likely strength and deformation capacity of the columns. The parameters ρ, Rp and Kc indicate longitudinal reinforcement ratio, ρ, axial force ratio, γ, and confinement factor, CF , respectively, defined as:

$$\gamma = \frac{P}{A_g f_{cc}'} \quad , \quad \rho = \frac{A_{st}}{A_g} \quad , \quad \mathrm{CF} = \frac{f_c'}{f_{cc}'} \qquad (5.1)$$

where, P is the axial load, A_{st} is the total area of longitudinal reinforcement, A_g is gross section area, f_c' and f_{cc}' are respectively the un-confined and confined concrete compressive strength. For this study, whereever one parameter is changed, other parameters are kept constant. The reference values of the variables include a column dimension, D, of 40x40 cm, a longitudinal reinforcement ratio of 0.015, an axial load ratio of 0.30 and a confinement factor of 1.2.

Numerical models were developed and analyzed in Seismo-Struct (SeismoSoft, 2004) for all geometrical and loading characteristics. The compressive strength of the concrete is 20 MPa and yield strength of longitudinal reinforcement is 380 MPa. The tensile strength of concrete is neglected. The model of Mander et al. (1988) is adopted in order to define constitutive relations of concrete in compression. The unconfined concrete strain (ε_{c0}) corresponding to the maximum compression strength was taken as 0.002, while the value for the confinement factor for the confined concrete in variants I, II and III is set as 1.2 for the confined concrete and 1.0 for the concrete cover. Confinement factor for variant IV is chosen as Table 5.1. All 3.00m high piers were modelled by a single force-based element along the columns height. Five integration sections per element were used, each one containing 200 integration points. The model of Filippou et al. (1983) is applied for the longitudinal reinforcement. The steel Young's modulus was taken equal to 200 GPa. The iterative procedure developed by Taucer et al. (1991) and Spacone and Filippou (1996) was adopted for the force-based element. Additionally, a co-rotational formulation was used to account for geometrical nonlinear effects. Cyclic loading was implemented through lateral displacement control of the top end of the each column. Loading was continued until columns reach a certain limit state. This limit is defined based on either steel fracture at strain 0.06 or ultimate strain in concrete obtained from Mander model.

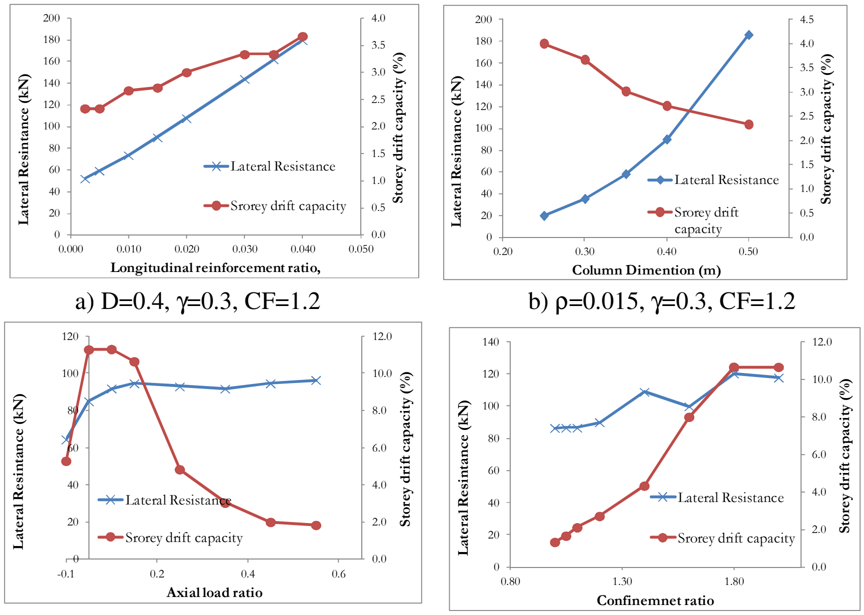
Table 5.1. Characteristics of different column studied, with a cantilever length of 3m

Var.	Case	Depth m	Width m	ebar rati	Rp	Rc
Variant I	1	0.4	0.4	0.0025	0.3	1.2
	2	0.4	0.4	0.005	0.3	1.2
	3	0.4	0.4	0.01	0.3	1.2
	4	0.4	0.4	0.015	0.3	1.2
	5	0.4	0.4	0.02	0.3	1.2
	6	0.4	0.4	0.03	0.3	1.2
	7	0.4	0.4	0.035	0.3	1.2
	8	0.4	0.4	0.04	0.3	1.2
Variant II	1	0.25	0.25	0.015	0.3	1.2
	2	0.3	0.3	0.015	0.3	1.2
	3	0.35	0.35	0.015	0.3	1.2
	4	0.4	0.4	0.015	0.3	1.2
	5	0.5	0.5	0.015	0.3	1.2
Variant III	1	0.4	0.4	0.015	-0.05	1.2
	2	0.4	0.4	0.015	0	1.2
	3	0.4	0.4	0.015	0.05	1.2
	4	0.4	0.4	0.015	0.1	1.2
	5	0.4	0.4	0.015	0.2	1.2
	6	0.4	0.4	0.015	0.3	1.2
	7	0.4	0.4	0.015	0.4	1.2
	8	0.4	0.4	0.015	0.5	1.2
Variant IV	1	0.4	0.4	0.015	0.3	1
	2	0.4	0.4	0.015	0.3	1.05
	3	0.4	0.4	0.015	0.3	1.1
	4	0.4	0.4	0.015	0.3	1.2
	5	0.4	0.4	0.015	0.3	1.4
	6	0.4	0.4	0.015	0.3	1.6
	7	0.4	0.4	0.015	0.3	1.8
	8	0.4	0.4	0.015	0.3	2

The effect of each parameter on column performance is shown in Figure 5.1. The influence of the aforementioned characteristics on shear strength and drift capacity of the columns is plotted within the same graphs. Generally, the effect of longitudinal reinforcement and column dimension on lateral resistance is more visible in comparison to other effects. A non-uniform trendline can be seen for drift capacity in part (c), where the column is affected differently with axial load changes which is attributed to classic axial-moment interaction, whereby the most resistance is achieved when deformation limits for concrete and reinforcement are obtained at the same time. The results shown in part (d) of Figure 5.1 indicate that confinement has a significant influence on drift capacity up to a certain level, but doesn't affect the strength significantly. This also agrees with traditional structural mechanics considerations (see, for example, Paulay and Priestley (1992)). Some further discussion of the trendlines and their implications for retrofit is provided in Section 6.

6. DISCUSSION OF RESULTS AND POTENTIAL IMPLICATIONS FOR RETROFIT

This work began by comparing the behaviour of two RC frame buildings (variant 1 and variant 2 of Figure 1.1) that differed only by the fact that one had masonry infill from the first floor upwards whereas the other had masonry infill over its full height. Results of incremental dynamic analyses tend to indicate that the full masonry infill case could sustain much larger intensity ground motion intensities for the collapse limit state. This might encourage structural engineers faced with the task of retrofitting the partial infill building to consider infilling the ground storey to render the structure similar to variant 1. While this option may help in reducing the probability of building collapse, it would not necessarily reduce the damage and losses expected of the building in low to moderate earthquake intensities. This is partly because the floor accelerations for the infill case are likely to be higher than in the partial infill case but in addition, it is well known that damage to infill masonry



a) D=0.4, γ=0.3, CF=1.2

b) ρ=0.015, γ=0.3, CF=1.2

c) ρ=0.015, D=0.4, CF=1.2

d) ρ=0.015, D=0.4, γ=0.3

Figure 5.1. Comparison of key characteristics on column hysteresis response

occurs at much lower levels of drift than in traditional RC frame structures, with masonry infills requiring repair at drifts of 0.3% (Hak et al., 2012). In addition, the open ground storey scenario would have a lower probability of reaching a partial-collapse limit state associated with masonry failure out-of-plane.

In addition to the points made above, it is recognised that the decision to retrofit a structure or not should be made within a risk assessment framework in which the probability of different levels of seismic intensity is compared with the probable losses for each intensity level. With this in mind, it was decided that the effect of different structural characteristics on the seismic vulnerability of RC frame structures with soft storey mechanisms should be examined in more detail. As such, sections 3 and 4 have examined the influence of p-delta effects and the post-yield stiffness ratio on the drift and acceleration demands of the open-ground-storey structure, whereas Section 5 examined how column dimensions, reinforcement contents and axial loads could affect deformation capacity.

The results of Section 3 have shown that P-delta effects will tend to increase the probability of collapse significantly, increasing peak and residual drifts significantly, particularly at high intensities. This intuitive observation is not yet well recognised by code assessment methods and therefore improvements to code assessment procedures should be an area for future research. Furthermore, it suggests that soft-storey structures could benefit from de-coupling of the gravity system from the lateral load resisting system. This point will be discussed further in later paragraphs.

The results of Section 4 have instead shown that while an increased value of post-yield stiffness ratio does help residual both peak and residual drifts, the overall impact does not appear to be large. As such, while the provision of some post-yield stiffness is important, it should not necessarily be a critical factor in retrofit efforts for soft-storey structures.

The results of Section 5 permit a number of points to be made regarding RC columns that are relevant for retrofit design. Firstly, note that the drift capacity of RC columns will tend to increase in proportion to the confinement provided. This supports the increasing use of jacketing and FRP wrapping solutions in the retrofit of structures. Secondly, and perhaps most interestingly, note that columns with high axial load ratios are likely to possess considerably less deformation capacity than those low to moderate axial load ratio. By reducing the axial load on a column from 0.4 (typical of existing RC buildings in Europe) to 0.1 the deformation capacity of the column could increase by a factor of four, from 2.0% to 8.0%. This again suggests that retrofit solutions that manage to reduce the axial loads on columns could greatly reduce the vulnerability of the soft-storey structures.

The above discussion has argued that if the gravity load system could be de-coupled from the lateral load resisting system this could help reduce the likely deformation demands which tend to be amplified by p-delta effects. In addition, it was just explained that if the axial load ratios on column sections could be reduced their deformation capacities could be significantly increased. One potentially effective and innovative means of retrofitting a structure with an open-ground storey could therefore be to introduce a series of gravity columns at the ground level, as shown in Figure 6.1, that slide with the first storey. By doing this, p-delta effects should be minimised. In addition, by jacking the gravity column system into position, the axial loads on existing columns may be reduced, thus greatly increasing their deformation capacity, without significantly affecting their lateral strength and potential for energy dissipation. Given the potential benefits of this retrofit scheme, this possibility should be explored as part of future research.

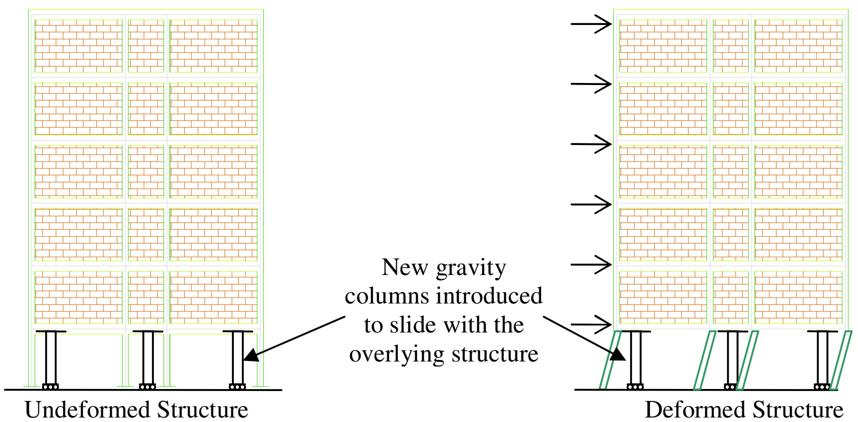


Figure 6.1. Possible means of de-coupling gravity loads from lateral loads in a soft-storey building.

7. CONCLUSIONS

The effect of some key characteristics on the behaviour of soft first storey buildings has been explored. A six-storey reinforcement concrete frame was analyzed for two scenarios of partial and full masonry infill, with soft-storey response developing at the ground storey for the partial infill case. Incremental nonlinear time history analyses were used to investigate the influence of P- delta effects and post yield stiffness ratio on the peak seismic demands for the case study frame.

The analysis results have indicated that P- delta effects can considerably affect the vulnerability of RC frames, although the effect depends on the intensity level. The post-yield stiffness ratio also affected the vulnerability, with increased post-yield stiffness ratios reducing peak and residual drifts but increasing peak floor accelerations. Overall, however, it could be argued that the post-yield stiffness did not influence the engineering demand parameters very significantly.

The results of static cyclic analyses of RC columns with different geometrical and mechanical properties were used to highlight the influence of some characteristics such as bar ratio, section dimensions, axial load ratio and confinement factor on the lateral resistance and drift capacity of RC columns. The trends observed are considered to be helpful in assessing the potential vulnerability of RC frame structures in which soft-storeys are expected to develop at the ground floor.

Finally, the implications of the analysis findings were discussed for potential retrofit implications. A novel retrofit scheme was proposed in which sliding gravity columns are introduced to reduce the impact of p-delta effects on displacement demands and to increase the deformation capacity of existing columns. Future research will look to develop this retrofit solution in more detail.

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