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P-DELTA EFFECTS ON TALL RC FRAME-WALL BUILDINGS

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ABSTRACT :

Current international design codes impose limits on the P-delta ratio, which appear to have been set to ensure a minimum reloading stiffness during cyclic response and with due consideration for the likely ductility demands imposed on structures. Whilst the current code limits may be reasonable for normal height structures, it is argued that the code limits should be reconsidered for tall buildings owing to limited displacements that real earthquake ground motions impose on such buildings. In this paper, the design of a 45-storey reinforced concrete frame-wall case study structure is used to highlight the significance of the p-delta limit within the modal response spectrum analysis procedure of the Eurocode 8. It is found that the strength of the structure is dictated by the P-delta limit for seismic actions, despite anticipated storey drifts and ductility demands being relatively low. A series of non-linear time-history analyses using a suite of spectrum-compatible real and artificial accelerograms, indicate that P-delta effects do not have a significant influence on displacements or storey drifts of the tall building. The likely causes of this behaviour are identified, making reference to earlier investigations into P-delta behaviour and with consideration of substitute structure concepts. To investigate the significance of the P-delta ratio further, a series of SDOF studies are undertaken for systems designed with P-delta ratios of up to 0.85. The results demonstrate that the p-delta ratio has little influence on the behaviour of long-period systems subject to real earthquake records and therefore it does not appear appropriate to impose strict limits on the P-delta ratio. Instead, it is recommended that the P-delta effects be evaluated for tall-building systems as part of an overall assessment of their response, using advanced non-linear time-history analyses with real records and within a large-displacement analysis regime.

KEYWORDS: P-delta, tall buildings, frame-wall, dual system, large displacement,

1. INTRODUCTION

As urbanisation increases worldwide, the construction of tall buildings in seismic regions is becoming increasingly common. While the scope of standard international codes such as the UBC97 (ICBO, 1997) and Eurocode 8 (CEN 1998-1) do include design procedures that can officially be used for the seismic design of tall buildings, there are concerns about the validity of such approaches as discussed by Sullivan (2007) amongst others. As part of a larger study conducted by Pham (2008), a 45-storey case-study RC frame-wall structure was designed using the modal response spectrum approach of the Eurocode 8. In this paper the critical design requirements of the Eurocode 8 identified during the design will be reported and the global response of the structure, assessed through non-linear time-history analyses, will be presented. The findings prompt a review of the basis of current P-delta limits in codes and consider whether P-delta limits are appropriate for tall buildings.

2. TALL FRAME-WALL BUILDING CASE STUDY

2.1. Design of the structure

As part of a general investigation into the effectiveness of the EC8 modal response spectrum procedure for tall RC frame-wall buildings, the 45-storey case-study structure shown in Figure 1 was designed by Pham (2008) (see reference for comprehensive details of the case study).

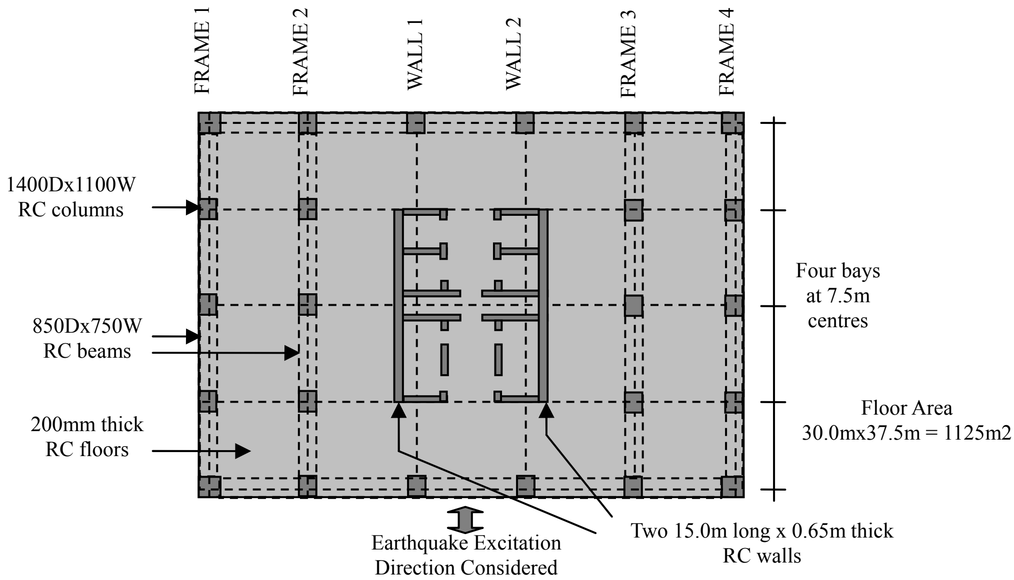

Figure 1 Plan view of 45-storey RC frame-wall case study structure (Pham Tuan et al 2008).

The case study structure possesses a practical 7.5m grid for architectural purposes and permits relatively lightweight floor slabs. Two 15m walls within a large central core work in parallel to four RC frames to resist the seismic loads in the North-South direction. A storey height of 4m was adopted, giving the building a total height of 180m. The concrete and reinforcement material properties adopted for the seismic design are values that could typically be found in tall building practice. Values for the concrete include: (i) f'c = 60.0 MPa and (ii) Ec = 33200 MPa. The expected strengths adopted for the reinforcing steel include: (i) fy = 500 MPa and (ii) Es=200000 MPa.

The design was conducted for a site possessing a PGA of 0.4g and soil type C of the EC8. In order to account for non-linear behaviour, the EC8 response spectrum method divides the elastic response spectrum by a behaviour factor, q, to obtain member design forces. The EC8 permits a maximum value of q of 5.4 for ductile reinforced concrete frame-wall structures. However, the code also requires that checks of P-delta be undertaken by evaluating a drift sensitivity coefficient or P-delta ratio, θ, given by Eq.(2.1), at every floor.

$$\theta = \frac{P_{tot}.d_r}{V_{tot}.h} \equiv \frac{P_{tot}\Delta}{M_n} \tag{2.1}$$

Where P_{tot} is the total gravity load at and above the storey considered; d_r is the design inter-storey displacements, evaluated as the difference of the average lateral displacements d_s at the top and bottom of the storey under consideration, V_{tot} is the design shear force; and h is the inter-storey height. The second part of the equation presents the equivalent expression for a SDOF system, where Δ is the system displacement and M_n is the system overturning resistance.
The code states that second-order effects (P-Δ effects) need not be taken into account only if the ratio given by Eq.(2.1) is less than 0.10, and that the maximum permissible p-delta ratio is 0.30. The possible basis of this limit is discussed in Section 3.1.

The initial design of the case study structure, undertaken using a q-value of 5.4, lead to p-delta ratios in excess of 0.30 and therefore to satisfy the code requirements, a lower value of q=3.0 was adopted, such that the design shear increased and the ratios obtained by Eq.(2.1) therefore decreased. As such, the controlling factor in the design of the structure was the p-delta limit.

At this stage, it is worth pointing out that behaviour factors in current codes (q in the EC8 and R in the UBC97) have been set considering the ductility capacity of a building system, with highly ductile systems being afforded greater reduction factors than systems of low ductility capacity. However, because long-period spectral displacements for real earthquakes reach a magnitude and distance-dependent plateau, the ductility demands that are typically imposed on tall frame-wall systems will be low, as pointed out by Sullivan (2007). As such, one could argue that the current use of high behaviour factors in current codes is non-conservative and hazardous. While there is clearly an argument for improved recommendations for the seismic design of tall buildings (which is being addressed by the current development of a number of tall-building codes) it is considered that the risk associated with the use of large behaviour factors has been somewhat off-set by two code limits; (i) Minimum base-shear and (ii) p-delta limits. The effect of the p-delta limit in the EC8 approach has just been highlighted in the design of the case-study structure. In a similar way, the minimum base-shear clause of the UBC97 essentially reduces the behaviour factor that one can apply in design. While it will be argued in this paper that p-delta limits may not be critical (and could be relaxed) for tall buildings, it is emphasised that this recommendation should not be used to justify lower strengths as part of a response-spectrum design approach. Instead, the argument is that p-delta limits could be relaxed when used within a rational design and analysis approach.

2.2. Non-linear time-history analyses of the structure and discussion of results

To investigate the dynamic behaviour of the case study structure, inelastic time-history analyses were carried out using Ruaumoko [Carr, 2004]. Small and large-displacement analyses were conducted to consider the response of the structure and the influence of p-delta effects when subject to real and artificial records. The displacement spectra for the various accelerograms used are shown in Figure 2. Note that the spectral displacements for the real records plateau at around 7.5s whereas the spectral displacements for the artificial records continue to increase to periods of around 15s.

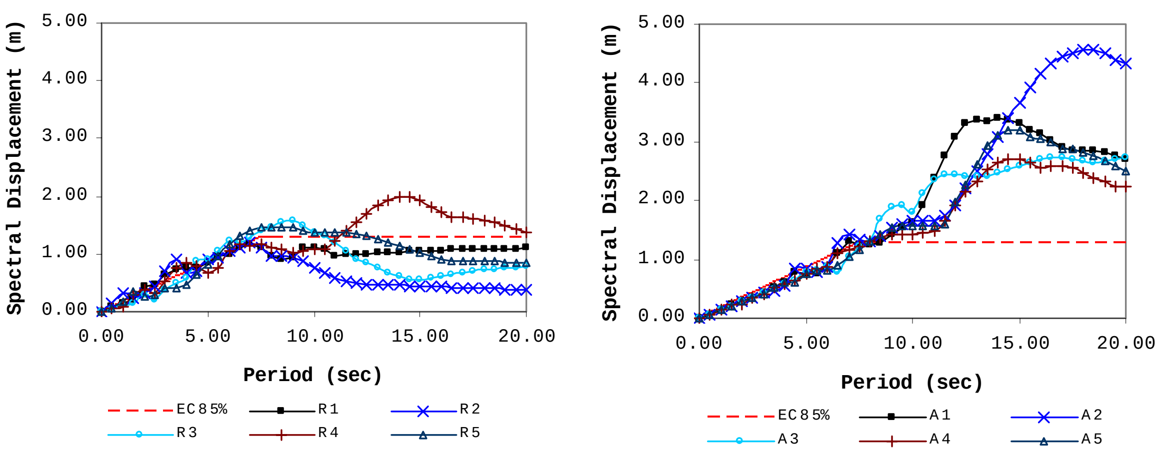

Figure 2 Displacement Spectra for: (left) five real records obtained from different earthquake events of magnitude ranging between 7.1 and 7.7 and (right) five artificial accelerograms.

A 2D lumped-plasticity plane-frame model of the structure was made assuming that the floor would behave as a rigid diaphragm. Giberson beam elements (refer Carr 2004) were used for the structure with modified Takeda hysteresis and plastic hinge lengths set in line with the recommendations of Paulay & Priestley (1992). The Rayleigh tangent-stiffness damping model was adopted with 5% damping set on first and second modes. An integration time-step of 0.005s was found to give stable results. Analyses were run on models with varying values of beam axial stiffness to account for beam lengthening effects (see Peng et al. 2007). The average of the maximum storey drifts recorded for both the real and artificial records, under both small and large-displacement analyses for the frame-wall system with reduced beam axial stiffness, are presented in Figure 3. For the system behaviour under other values of beam axial stiffness, and a general discussion of the effects this phenomena could have on tall buildings, refer to Pham (2008).

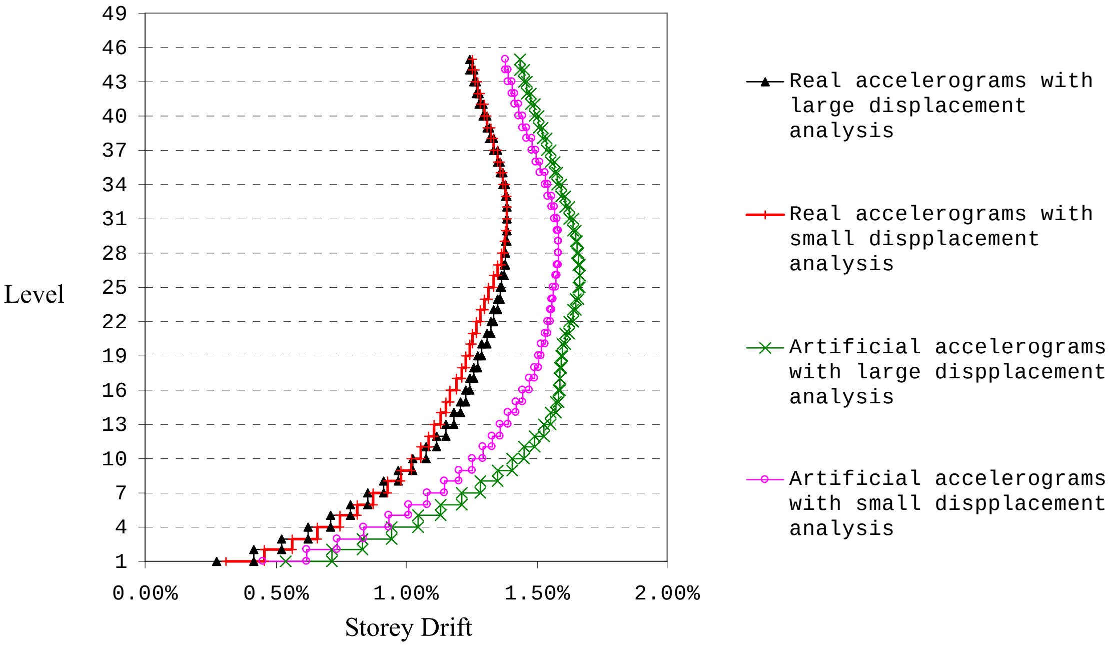

Figure 3 Storey drifts recorded up the height of the 45-storey case study structure, when subject to real and artificial accelerograms under both small and large-displacement analyses. In contrast to the artificial accelerograms, note that P-delta does not significantly affect the drifts for the real records.

The maximum storey drifts shown in Figure 3 indicate that when artificial records were utilised, p-delta effects (active only under the large-displacement analysis regime) increased drifts by around 10% up the height of the building. In contrast, p-delta effects had little impact on the drifts when real records are utilised, increasing values only marginally from levels 10 to 25. This behaviour will be accounted for with reference to Figure 4.

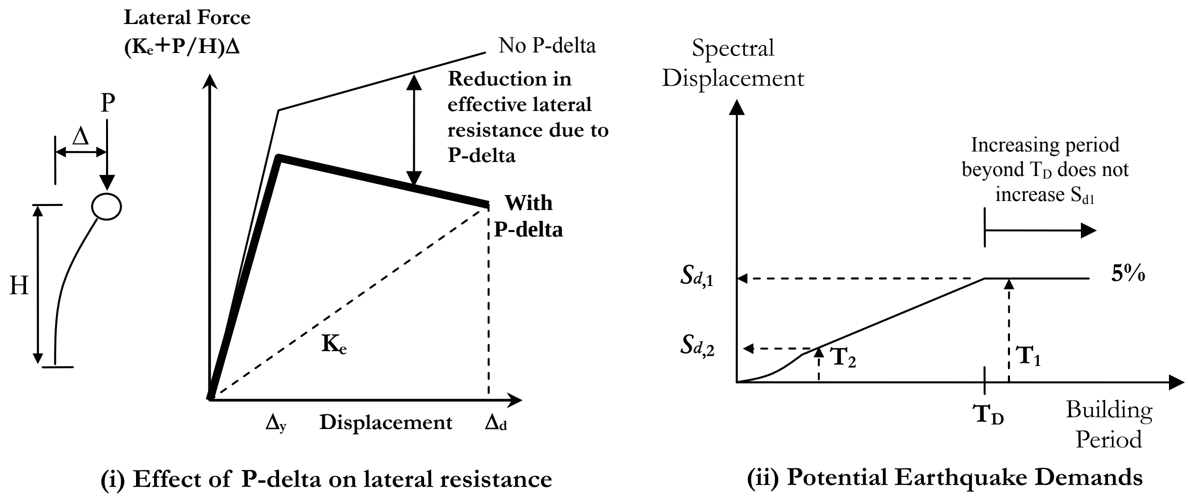

Figure 4 (i) Effect of P-delta on the effective lateral stiffness, K_e, of a structural system (adapted from Macrae et al. 1991), (ii) the effect that increasing the effective period (caused by a reduction in effective stiffness) would be expected to have on the likely displacement demands.

As illustrated in Figure 4(i), for a SDOF system with a given level of lateral strength, P-delta effects effectively cause as a reduction in the lateral resistance that the system can offer against seismic actions. The reduced effective stiffness implies that the system will possess a longer effective period. Figure 4(ii) plots the shape of typical displacement spectra at long periods (see Faccioli et al. 2004 and EC8) and considers how changes in period can affect displacement demands. With reference to Figure 4(ii), it is clear that for short and medium rise structures, with effective periods less than the spectral displacement cut-off period, T_D, an increase in the effective period implies that the system displacements are increased proportionately. However, for tall building structures in which the fundamental period lies beyond T_D, even large increases in the effective period should not cause an increase in the 1st mode system displacements. That is, for a very long period SDOF system, the peak displacements are limited by the peak spectral displacement demands of the ground motion, irrespective of whether P-delta effects are active or not.

Also shown in Figure 4(ii) is a tall building 2^{nd} mode period, T_2. P-delta effects could be expected to increase the displacement demands associated with such a higher mode, since lengthening the effective 2^{nd} mode period causes an increase in spectral displacement demands. However, the results presented in Figure 3 suggest that the overall influence higher mode p-delta effects might have on tall RC frame-wall buildings may not be very significant, as the peak drifts (at around level 32) were unaffected.

In summary, the case study results suggest that p-delta effects may not be very significant for tall RC frame-wall structures. This behaviour has been attributed to the limited displacement demands real earthquakes impose at long periods. The finding suggests that a review of the p-delta limits, currently imposed by international codes for tall buildings, may be warranted.

3. REVIEW OF P-DELTA LIMIT

3.1. Background

The source of current limits included in the EC8 and other international codes is not certain. However, the limit of 0.3 does correspond with the recommendations made by Macrae et al. (1991) as part of a large study into P-delta effects in seismic regions. Their research utilised the results of a number of experiments on RC columns to set P-delta limits. The argument made was that for a system in which p-delta effects are accounted for using the effective resistance concept (see Figure 4(i)), the ratio of the reloading to the unloading stiffness of the system should never be less than 0.05. Consequently, Macrae et al. (1991) found that for the Takeda (Otani, 1981) hysteretic shape and a displacement ductility of 6.0, a p-delta limit of 0.3 is required.

One could follow a similar argument to that of Macrae et al. (1991) for tall RC frame-wall structures, which are not likely to be subject to ductility demands greater than 3.0, and find that the p-delta limit could be increased to 0.6 for tall buildings. However, for alternative hysteretic models such as the bi-linear rule with low strain hardening, the need to ensure a reloading to unloading stiffness ratio of 0.05 might appear to require more restrictive limits than the 0.3 currently included within codes. Furthermore, the arguments made in the previous section suggest that the characteristics of the ground motions may be more critical in assessing the significance of p-delta effects than the p-delta ratio. For these reasons, a series of SDOF studies were undertaken to study the significance of the p-delta ratio for tall building systems, as explained next.

3.2. SDOF studies

To investigate the significance of the p-delta limit for long-period structures, a number of SDOF analyses were undertaken in Ruaumoko (Carr, 2004). The SDOF systems were given strengths and gravity axial loads such that P-delta ratios of approximately 0.3, 0.5 and 0.7 could be investigated. Note that the vertical load and effective mass were set constant, and the height was varied to obtain a suite of

SDOF systems. Both small and large-displacement non-linear time-history analyses using 5 real records were then run for the SDOF systems. Further details of the modelling and analysis approach can be found in Pham (2008).

The peak displacements recorded for a range of the SDOF systems with P-delta ratio of 0.5 are presented in Figure 5. Displacements for the other P-delta ratios are reported by Pham (2008). The peak displacements shown in Figure 5 are relatively uniform with and without P-delta effects. A clear exception to this trend occurs for record R4, which causes the SDOF systems of period greater than 9s to collapse. The reasons for this will be discussed later. Also note that the peak displacement curves recorded for SDOF systems of period 7 to 12s are very similar in shape to the displacement curves recorded using small-displacement analyses for SDOF systems with period 10 to 15s. This general trend supports the concept that p-delta effects cause an increase in the effective period of the system.

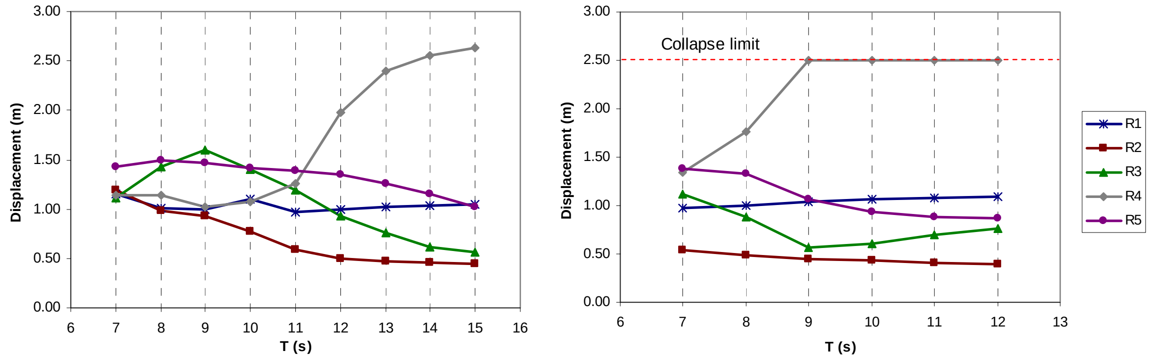

Figure 5 Peak displacements recorded for the SDOF systems designed to for θ=0.5 and analysed using: (left) small-displacement analyses, and (right) large-displacement analyses (i.e. with p-delta effects).

To better gauge the significance of the p-delta ratio (θ from Eq.2.1), the p-delta ratio was recalculated using the peak displacements measured for each earthquake record. The ratio of the peak displacement recorded with p-delta effects active to the displacement recorded without p-delta effects active, is plotted versus the p-delta ratio (theta) in Figure 6.

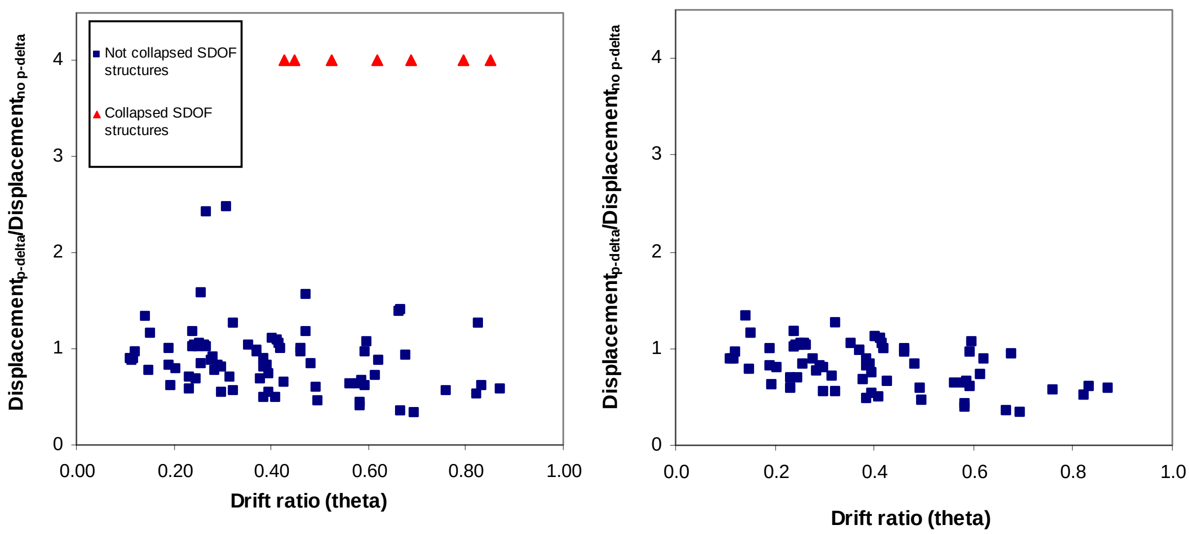

Figure 6 Ratio of peak displacements obtained for long-period SDOF systems with and without p-delta effects versus the p-delta ratio. The left side shows the results obtained for all records. The right side excludes the results of record R4.

The results presented in Figure 6 clearly show that the p-delta ratio, theta, has little influence on the behaviour of long-period systems subject to real earthquake records. For some SDOF systems that possessed large P-delta ratios, greater than 0.8, the effect of including p-delta effects was not to increase displacements but instead to reduce them, lengthening the effective SDOF system period to longer values where the displacement demand was less (see displacement spectra of Figure 2).

The question that remains is then why record R4 caused the collapse of some of the SDOF systems, even when the p-delta ratio was quite low. Furthermore, one should question how record R4 managed to impose displacement demands greater than 2.5m (see Figure 5) when the peak spectral displacement demands were only just greater than 2.0m (see left side of Figure 2). Some insight to these questions can be obtained by considering the hysteretic response of a SDOF system subject to record R4, shown both in terms of moment-curvature and force-displacement for a 14s SDOF system in Figure 7.

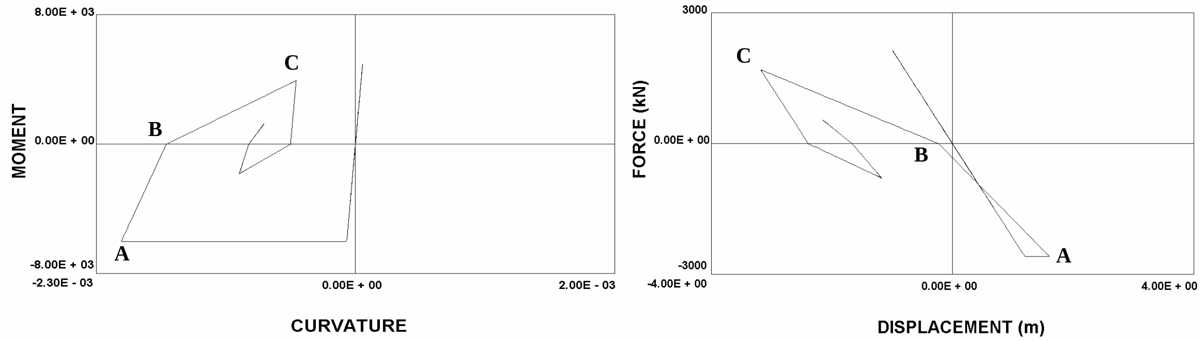

Figure 7 Window of time-history response in terms of Moment-Curvature (left) and Force-Displacement (right) for a 14s SDOF system subject to record R4.

Consideration of the moment-curvature and force-displacement response together reveals why the SDOF system recorded peak displacements well in excess of the peak spectral displacements, even under a small-displacement analysis regime. Firstly, recall that when a structure displaces inelastically in a lumped-plasticity analysis, the total displacement is the sum of the elastic displacement and the displacement due to inelastic deformation of the plastic hinge, as shown in Eq.(3.1).

$$\Delta_t = \Delta_e + \Delta_p = \frac{\phi_e H^2}{3} + (\phi_t - \phi_e) L_p H \qquad (3.1)$$

where Δ_t and Δ_e are the total and elastic displacements respectively, Δ_p is the displacement due to deformation of the plastic hinge, ϕ_t and ϕ_e are total and elastic curvatures, L_p is the plastic hinge length and H is the height of the structure.

The SDOF system referred to in Figure 7 is subject to a displacement ductility demand of only 1.3 at point A, but the corresponding curvature-ductility demand is around 20. The large curvature ductility reflects the fact that the long period system was tall with a relatively short plastic hinge length. The moment curvature loops follow the Takeda rule, with an unloading and reloading stiffness that reduces as ductility demands increase. As such, the elastic curvature that is recovered when the force unloads to zero from points A to B, is much greater than the elastic curvature developed in displacing the structure out to point A. As a result, the analysis suggests that the structure has a negative displacement after unloading from points A to B. As reloading begins in the opposite direction, the Takeda hysteretic rule instructs the moment-curvature loop to head towards the positive yield curvature. Therefore, when reloading, the total elastic curvature that can develop before yield is many times greater than the yield curvature. As a result, in deforming from points B to C, the structure is responding elastically and the displacement is computed using the $\phi_e H^2/3$ term of Eq.(3.1). As the height of the SDOF system is large and the elastic curvature being recovered is large, the predicted displacement response becomes very high, significantly greater than the peaks predicted by elastic response spectra.

The above discussion assists in understanding why such large displacements were recorded for R4. The behaviour observed raises questions over the sensitivity of lumped plasticity analyses to the input values of plastic hinge length in proportion to member length. Furthermore, the manner in which the results of non-linear analyses varied from elastic spectra highlights the fact that to obtain an accurate estimation of system response, non-linear time-history analyses should be undertaken.

3. CONCLUSIONS

This paper has shown that the design strength of a tall RC frame-wall building may be governed by the P-delta ratio limit of 0.3 when using the Eurocode 8. Non-linear time-history analyses suggested that the behaviour of a case study building subject to real records may not, in fact, be very sensitive to P-delta effects. An hypothesis was then made that for very long period systems, the peak displacements are limited by the peak spectral displacement demands of the ground motion, irrespective of whether P-delta effects are active or not. To investigate the significance of the P-delta ratio further, a series of SDOF studies were undertaken for systems designed with P-delta ratios of up to 0.85. The results demonstrated that the p-delta ratio has little influence on the behaviour of long-period systems subject to real earthquake records and therefore it does not appear appropriate to impose strict limits on the P-delta ratio. Instead, it is recommended that the P-delta effects be evaluated for tall-building systems as part of an overall assessment of their response, using advanced non-linear time-history analyses with real records and within a large-displacement analysis regime. Finally, this study has raised questions about the sensitivity of lumped plasticity analyses to the input values of plastic hinge length in proportion to member length. This uncertainty should be investigated further as part of future research.

REFERENCES

Carr, A. J. 2004. *Ruaumoko3D – A program for Inelastic Time-History Analysis*. Department of Civil Engineering, University of Canterbury, New Zealand.
CEN 1998. *Eurocode EC8 - Design of structures for earthquake resistance -Part 1: General rules, seismic actions and rules for buildings*, prEN-1998-1, Comite Europeen de Normalization, Brussels, Belgium.
Faccioli, E., Paolucci, R., Rey, J., 2004. "Displacement Spectra for Long Periods." Earthquake Spectra, 20(2).
Peng, B.H.H., Dhakal, R.P., Fenwick, R.C., Car, A.J., Bull, D.K. [2007] "Flexural, Axial Load and Elongation Response of Plastic Hinges in Reinforced Concrete Member," Proceedings of 8th Pacific Conference of Earthquake Engineering, Singapore. Paper No. 30.
ICBO, 1997. *Uniform Building Code*, International Conference of Building Officials, Whittier, California, USA.
Macrae, G.A., Priestley, M.J.N, Tao, J., 1991 "P-Δ in seismic regions," Structural Systems Research Project, Department of Applied Mechanics and Engineering Sciences, University of California, San Diego, Report No.SSRP-93/05.
Otani, S. 1981. "Hysteresis models of reinforced concrete for earthquake response analysis" Journal of the Faculty of Engineering, University of Tokyo, Vol. XXXVI, No.2, pp125-159.
Paulay, T., Priestley, M.J.N., 1992. *Seismic Design of Reinforced Concrete and Masonry Buildings*, John Wiley & Sons, Inc., New York.
Pham, T.H., 2008 "Seismic Design Considerations for Tall Buildings" MEEES Masters Dissertation supervised by Sullivan, T.J. and Calvi, G.M., held at ROSE School, University of Pavia, Italy. 85pages.
Priestley, M.J.N., Calvi, G.M., Kowalsky, M.J. 2007. *Displacement Based Seismic Design of Structures*, IUSS Press, Pavia, Italy. 720pages (www.iusspress.it).
Sullivan, T.J. 2007. "Displacement Considerations for the Seismic Design of Tall RC Frame-Wall Buildings", Proceedings of 8th Pacific Conference of Earthquake Engineering, Singapore. Paper 125.