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Seismic Evaluation of R/C Buildings Using High Performance Materials

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SEISMIC EVALUATION OF R/C BUILDINGS USING HIGH PERFORMANCE MATERIALS

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ABSTRACT

The performance of reinforced concrete (R/C) buildings made of high performance materials (HPM) in seismic areas is evaluated through the parametric analysis of twenty-two buildings designed and detailed according to the current (prEN) version of Eurocode 8, for ductility classes (DC) "Medium" and "High", in combination with the current (prEN) version of Eurocode 2. Seismic assessment of the buildings is carried out by means of nonlinear static (pushover) analysis and nonlinear dynamic (time-history) analysis.

INTRODUCTION

Despite the ever-increasing need for the vertical expansion of metropolitan cities owing to the over-inflated prices of land, Eurocode 8 (1), which governs the earthquake resistant design of buildings, does not explicitly cover HPM, which would allow the construction of high rise buildings. The proposal to include HPM in the final (EN) version of Eurocode 8 (2) was put forward, but the drafting committee decided against it at the time due to the relative scarcity of information regarding the performance of structures using HSM. On the other hand, Eurocode 2 in its final draft of the EN version (3) includes provisions for the design of concrete sections with concrete compressive strength up to 100 MPa and steel grades of up to 600 MPa.

SEISMIC DESIGN OF THE R/C BUILDINGS

The twenty-two 15 - storey R/C buildings have the same structural configuration, shown in Figure 1, but differ from one another in terms of the properties of the materials used. Each individual building was designed for a different set of concrete strength and steel grade, and for either Medium or High ductility class, as tabulated in Table 1. Three different concrete strengths i.e. 50 MPa, 70 MPa and 90 MPa were considered. The influence of yield strength of longitudinal and transverse reinforcement on the design of the structure was also examined for each concrete strength by considering steel grades of 500 MPa, 800 MPa and 1200 MPa. Hence, the notation f_c50f_y500-M denotes the design of the structure using 50 MPa concrete strength, 500 MPa yield strength of longitudinal and transverse reinforcement and assuming Medium ductility class.

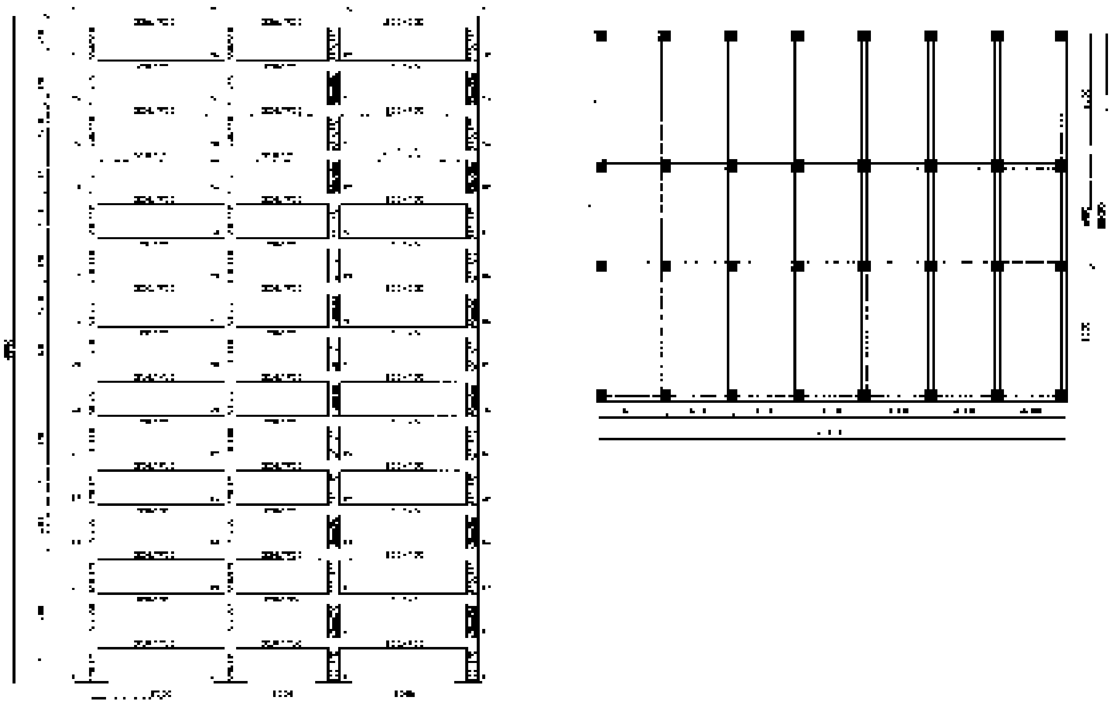


Figure 1. Geometry of the buildings

Table 1. Details of materials and DC considered in each building

No	Notation	f_c (MPa)	f_{yw}/f_{yl} (MPa)	DC
1	f_c50f_y500-M	50	500/500	Medium
2	f_c50f_y800-M	50	800/800	Medium
3	f_c50f_y1200-M	50	1200/1200	Medium
4	f_c70f_y500-M	70	500/500	Medium
5	f_c70f_y800-M	70	800/800	Medium
6	f_c70f_y1200-M	70	1200/1200	Medium
7	f_c90f_y500-M	90	500/500	Medium
8	f_c90f_y800-M	90	800/800	Medium
9	f_c90f_y1200-M	90	1200/1200	Medium
10	f_c50f_c90NY-M	50 for beams 90 for columns	500/500	Medium
11	f_c50f_c90HY-M	50 for beams 90 for columns	1200/500	Medium
12	f_c50f_y500-H	50	500/500	High
13	f_c50f_y800-H	50	800/800	High
14	f_c50f_y1200-H	50	1200/1200	High
15	f_c70f_y500-H	70	500/500	High
16	f_c70f_y800-H	70	800/800	High
17	f_c70f_y1200-H	70	1200/1200	High
18	f_c90f_y500-H	90	500/500	High
19	f_c90f_y800-H	90	800/800	High
20	f_c90f_y1200-H	90	1200/1200	High
21	f_c50f_c90NY-H	50 for beams 90 for columns	500/500	High
22	f_c50f_c90HY-H	50 for Beams 90 for Columns	1200/500	High

Additionally, another two structures were designed utilising different concrete strengths for beams and columns. In structures number 10 and 21 in Table 1, a concrete strength of 50 MPa was used for the beams throughout the building, while 90 MPa concrete strength was adopted for the columns. Normal yield steel of strength 500 MPa was used for both beams and columns and longitudinal and transverse reinforcement, hence the two capital letters NY in the notation. Similarly, in structures 11 and 22 in Table 1 while concrete strength varied between beams and columns, high yield steel of yield strength 1200 MPa was used for transverse reinforcement while 500 MPa steel was utilised for the longitudinal reinforcement. The design of each structure was carried out by considering the planar structure shown in Figure 1, which is permitted by Eurocode 8 as an alternative to 3D structural analysis. In order to simplify somewhat the designs, the reinforcement placed in beams was kept constant for every three storeys and the reinforcement in columns for every five storeys.

The buildings were designed for a design ground acceleration of 0.25g assuming subsoil class C and importance category II (γ_I=1.2). The behaviour factor used to derive the design seismic actions was 2.66 for the DC M and 4.0 for DC H, which is in both cases more conservative than the maximum allowable by the prEN version of Eurocode 8. The vertical component of the seismic action has been ignored, as allowed by the code for a regular R/C frame. In addition to self-weight, the buildings were designed to carry all superimposed dead loads, live loads and seismic load. The value of the load, due to floor finishing and partitions was taken equal to 3.5 kN/m at all spans and storeys. A uniformly distributed "live load" of 2 kN/m^2 was assumed to act at each storey, leading to a distributed beam load equal to 8 kN/m. Since wind load did not govern the design, it was therefore not considered.

The elastic analysis of the buildings involved four separate analytical models with respect to concrete strength. Each of these models was analysed and the action effects for each individual member of the building were obtained using the *Statik 3* (4) structural analysis program. In order to account for the influence of axial loading on the degree of cracking, the effective stiffness of the structural elements has been taken as 50% of the uncracked sections, which is the upper limit recommended by Eurocode 8. The effect of the non-structural elements, such as partitions, parapets etc., on the deformational behaviour of the buildings was not taken into account in order to avoid any confusing results caused by such an interaction. Finally, second order (P-Δ) effects were ignored as the Eurocode 8 condition for the interstorey drift sensitivity coefficient (θ) was satisfied in all storeys. However, it has to be mentioned that the resulting values of θ were close to the limiting value of 0.10, a fact that determined the choice of cross-sections. Prior to any section design being carried out, the elastic displacements d_{ei} resulting from the design seismic action were checked to satisfy the limits defined in the code and no violation of these limits was found. In general, the high-rise buildings studied herein meet the criteria for regularity in plan and in elevation and satisfy the geometrical constraints of Eurocode 8, regarding the size of the beams and columns. In the case of columns, the normalised design axial force v_d is well below the limiting values of 0.65 for the medium DC buildings and 0.55 for the high DC buildings. The detailed design of the twenty-two buildings is given elsewhere, Konstantinidis (5).

PERFORMANCE ASSESSMENT

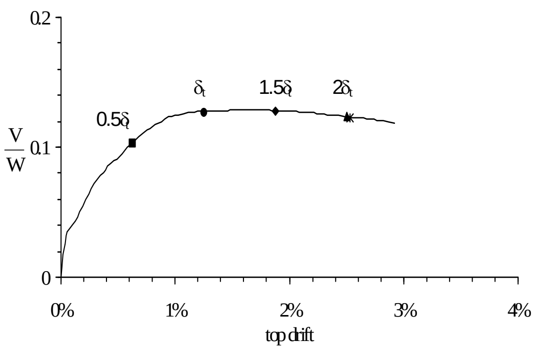
The 15-storey R/C moment resisting frame system was modelled as a 484 node and 525 element system. The total number of elements results from the subdivision of each column and beam into five elastoplastic cubic elements, [Izzuddin (6)]. This is considered to be satisfactory, because except for the increased accuracy, this further subdivision captures the effect of extra confinement within the critical region of the columns and the beams. Both pushover and nonlinear time history analyses have been carried out using the finite element program ADAPTIC, [Izzuddin (6)]. On the concrete modelling side, the model proposed in Konstantinidis (5) was used to represent the cyclic stress-strain behaviour of unconfined and confined concrete. The bilinear steel model with kinematic strain hardening was adopted for modelling the reinforcing steel. The material properties used in assessing the performance of the buildings were based on mean values (e.g. 58 MPa for concrete with characteristic compressive strength 50 MPa), since a deterministic assessment is sought. The mean yield value for all steel reinforcement was assumed 10% greater than the characteristic yield strength.

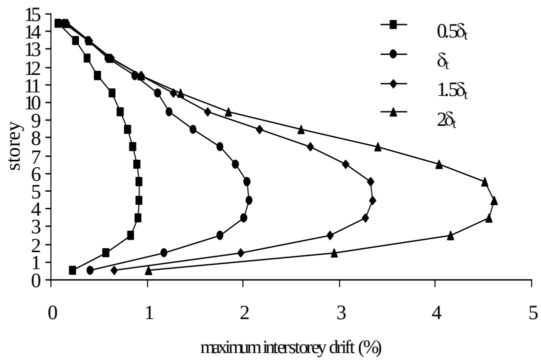
Assessment of the Performance Using Pushover Analysis

For each building the curve of the seismic coefficient, defined as the ratio of the base shear to the total weight of the building, versus the top drift, defined as the ratio of the displacement of the control node (top storey slab) to the total height of the building, is plotted. The change in slope of this curve indicates yielding of the structural elements. It has to be clarified that since the material models in ADAPTIC do not take account of material failure under any ultimate limit state, but continue to describe the material responses "indefinitely", the curves are very smooth even at large drifts. For this reason, the point where any structural member reaches its ultimate concrete strain, defined as the strain value at 50% drop of maximum confined concrete strength, was acquired separately and was plotted using the star sign on Figure 2a. Target displacements for seismic actions corresponding to the four performance levels described by FEMA-356 (7) are also shown on the same figure (they are multiples of the displacement δ_t corresponding to the design earthquake of 0.25g). The maximum values of the interstorey drift ratios along the height of the buildings are shown in Figure 2b and the distribution of plastic hinges formed in the structure at the four performance levels, which gives an indication of the energy absorbed by the system, is illustrated in Figure 2c. Due to space limitation, detailed presentation of results is restricted to building f_c50f_y500-H only [see Konstantinidis (5) for other cases].

The overstrength factor determined at global yield, for the structures designed for DC M, ranged between 1.14 for f_c50f_y500-M to 1.44 for f_c90f_y1200-M. It has to be recalled that the effective stiffness of the structural elements for design was calculated assuming 50% of the uncracked section, while for the assessment it was taken initially equal to the uncracked section (EI_g) and the reduction in stiffness was accounted for via the concrete model. The value of the ratio of maximum shear to the code-defined base shear ranged from 1.19 for f_c70f_y500-M to 1.56 for f_c90f_y1200-M. Significantly higher overstrength values were found for buildings consisting of the same materials, but designed for DC H. Specifically, the overstrength factors determined at the yield limit state ranged between 1.26 for

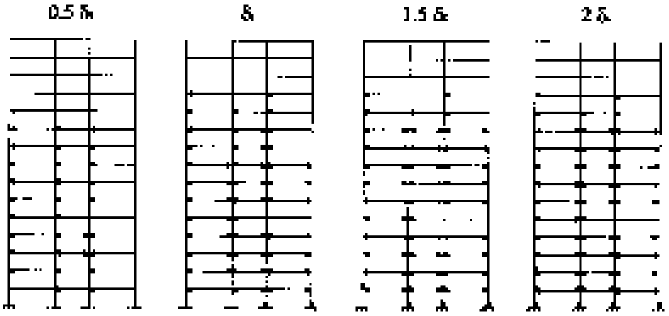
f_c70f_y500 and 1.88 for f_c90f_y1200, whereas overstrength factors calculated based on maximum shear varied between 1.39 and 2.04 for f_c50f_y500-H and f_c90f_y1200-H, respectively. For the buildings using the same concrete strength, the tendency of developing higher overstrength factors, as the yield strength of steel reinforcement increases is more pronounced for DC H buildings. The same overstrengths were found for the buildings designed for the same ductility class, but using different concrete strengths in the beams and the columns and different yield strengths of transverse reinforcement.





a) Base shear vs. top drift

b) Interstorey drifts



c) Distribution of plastic hinges

Figure 2. Pushover analysis results for f_c50f_y500-H building

The values of target displacements and top drifts corresponding to each discrete performance level are illustrated in Table 2. The selected soil class C corresponding to deep deposits of loose-to medium cohesionless soil, which amplified the response spectrum (amplification factor S equal to 1.35), played a significant role in obtaining high values of target displacements. For the immediate occupancy performance level ($0.5\delta_t$) the FEMA-356 interstorey drift limitation of 1% is satisfied for all buildings, while no structural element failed. The defined limit of 2% interstorey drift at the life safety level (δ_t) is also respected in most buildings except for f_c50f_y500-H (2.06%) and f_c90f_y500-H (2.26%). An unclear situation regarding the maximum values of interstorey drifts results at the collapse prevention level ($2\delta_t$), which was chosen as an earthquake with intensity twice that of the design earthquake (0.50g). The buildings designed for concrete strength of 50 MPa not only exceeded the 4% limit, but in most cases and for both ductility classes element failure was observed with the single exception of f_c50f_y500-M, in which the maximum value of interstorey drift was well below the limit. The buildings designed for 70 MPa concrete strength generally exceeded the limit except for buildings

f_c70f_y800-M and f_c70f_y500-H. Failure occurred in the buildings with HYS of 1200 MPa for both ductility classes. From the buildings designed for concrete strength of 90 MPa only f_c90f_y500-H did not exceed the limit at this performance level. Interestingly, no element failed at such a high earthquake level. The best behaviour at this level was obtained in the building with different concrete strengths in beams and columns, as none exceeded the limit of 4% drift. Finally, $1.5\delta_t$ was not corresponded to a discrete performance level, but the results were included in order to provide additional information about the behaviour of the structures at an intermediate stage between the collapse prevention and life safety performance levels).

Table 2. Target displacements and top drifts at the four performance levels

Building	$0.5\delta_t$ max. (%)	δ_t max. (%)	$1.5\delta_t$ max. (%)	$2\delta_t$ max. (%)
f_c50 f_y500-M	0.65	1.33	2.20	3.34
f_c50 f_y800-M	0.78	1.59	2.61	4.10 Failure
f_c50 f_y1200-M	0.93	1.85	3.02 Failure	4.11 Failure
f_c70 f_y500-M	0.77	1.57	2.79	4.13
f_c70 f_y800-M	0.78	1.57	2.57	3.81
f_c70 f_y1200-M	0.86	1.77	2.67	4.19 Failure
f_c90 f_y500-M	0.78	1.69	2.92	4.19
f_c90 f_y800-M	0.78	1.65	2.78	4.20
f_c90 f_y1200-M	0.86	1.77	2.76	4.17
f_c50 f_c90NY-M	0.76	1.59	2.75	3.95
f_c50 f_c90HY-M	0.76	1.59	2.75	3.95
f_c50 f_y500-H	0.92	2.06	3.35	4.61 Failure
f_c50 f_y800-H	0.88	1.86	3.34	4.85 Failure
f_c50 f_y1200-H	0.88	1.79	2.89 Failure	4.66 Failure
f_c70 f_y500-H	0.82	1.68	2.82	3.87
f_c70 f_y800-H	0.81	1.62	2.82	4.07
f_c70 f_y1200-H	0.89	1.84	2.94	4.42 Failure
f_c90 f_y500-H	0.81	2.26	2.62	3.71
f_c90 f_y800-H	0.83	1.76	3.09	4.30
f_c90 f_y1200-H	0.91	1.79	2.66	4.04
f_c50 f_c90NY-H	0.74	1.43	2.44	3.44
f_c50 f_c90HY-H	0.73	1.42	2.41	3.44

Assessment of the Performance Using Nonlinear Dynamic Analysis

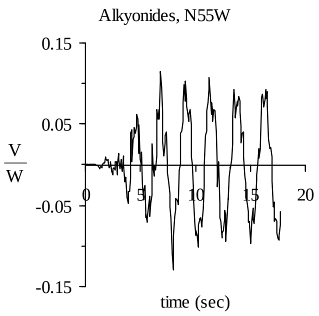
The most sophisticated approach for examining the inelastic response of a structure involves the application of nonlinear dynamic analysis, which determines the time-history of the model's response under the action of an earthquake record. However, the most significant uncertainty of the method is the selection of the appropriate earthquake input for which the assessment will be carried out. From the strong motion data available in Greece since 1972, three of the most damaging earthquakes recorded were selected (Volvi 1978 earthquake - Thessaloniki N60W record, Alkyonides 1981 - Corinth N55W, and Kalamata 1986 N10W) and which are compatible with the assumed design soil class C. The method proposed by Kappos and Kyriakakis (8) was adopted for the purpose of scaling the selected input motions, which is based on spectrum intensities calculated in the period band around the fundamental period of the structure analysed. For the purpose of scaling the natural records to the seismic level corresponding to collapse prevention, the resulting scaling factors for the design earthquake were doubled [Paulay and Priestley (9), Kappos (10)].

Figure 3 shows the results of the time history analyses for f_c50f_y500-H building. The full report of 60 inelastic time-history analyses for both ductility classes are presented elsewhere, Konstantinidis (5). It has to be mentioned that the dynamic analyses were first carried out for the buildings with reinforcement steel of 500 MPa yield strength using all input motions scaled to the design earthquake (0.25g). The resulting most critical earthquake (i.e. that causing the highest demands), which in all cases was found to be the Corinth N55W record was then used for the buildings with 800 MPa and 1200MPa steel and for the earthquake corresponding to collapse prevention (0.50g).

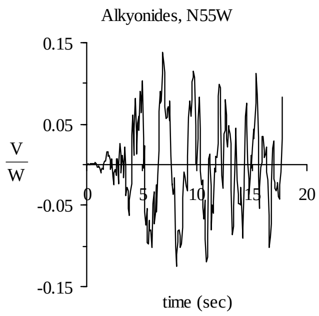
Table 3. Overstrength of the designed buildings resulted from dynamic analysis

Building	Volvi, N60W **0.25g**		Alkyonides, N55W **0.25g**		Kalamata, N10W **0.25g**		Alkyonides, N55W **0.50g**	
	DC M	DC H	DC M	DC H	DC M	DC H	DC M	DC H
$f_c50\ f_y500$	1.21	1.35	1.23	1.40	1.00	1.05	1.45	1.50
$f_c50\ f_y800$	-	-	1.30	1.41	-	-	1.51	1.50
$f_c50\ f_y1200$	-	-	1.20	1.21	-	-	1.63	1.61
$f_c70\ f_y500$	1.24	1.24	1.26	1.35	0.92	1.09	1.35	1.56
$f_c70\ f_y800$	-	-	1.41	1.47	-	-	1.58	1.73
$f_c70\ f_y1200$	-	-	1.35	1.29	-	-	1.67	1.64
$f_c90\ f_y500$	1.33	1.10	1.33	1.34	1.17	1.11	1.43	1.64
$f_c90\ f_y800$	-	-	1.59	1.65	-	-	1.91	1.81
$f_c90\ f_y1200$	-	-	1.43	1.41	-	-	1.74	2.15
$f_c50\ f_c90$ NY	1.23	1.33	1.23	1.44	1.13	1.09	1.41	1.52
$f_c50\ f_c90$ HY	-	-	1.24	1.55	-	-	1.44	1.50

The ratio of the maximum base shear resulting from the dynamic analysis to the base shear defined by the code, varied also with the input record. Under the design earthquake (0.25g) the building f_c90f_y800 for DC M and DC H was able to resist 59% and 65% more seismic forces than it was designed for (see Table 3), which is considered to be very good performance for a bare frame R/C building. From the results obtained from the Corinth N55W record, it can be concluded that the buildings designed for the same yield strength of steel but with increasing concrete strength resulted in greater overstrengths, with the exception of those buildings designed for DC H and using 500 MPa steel. Under the earthquake intensity for the collapse prevention limit state (0.50g), this is only observed in the f_c50f_y500-M and f_c70f_y500-M buildings, while in general high overstrength values were obtained. In buildings designed for DC M the highest overstrength is found for f_c90f_y800 (1.91), while the building f_c90f_y1200-H was able to resist more than double the design base shear. It is also notable that the building incorporating different concrete strengths in the beams and the columns and HYS as transverse reinforcement resulted in 7.6% higher overstrength factor for DC H under the design earthquake. For the collapse earthquake the difference was insignificant for both ductility classes. Based on maximum interstorey drift values, the behaviour of the buildings under the life safety and the collapse prevention events was generally satisfactory as they were well below the 2% and 4% limits of FEMA 356.



a) Base shear time history



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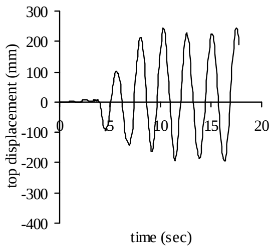
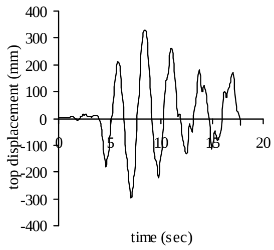
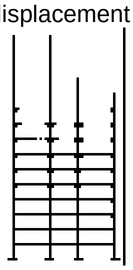


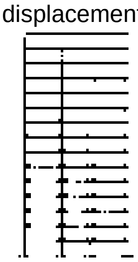
b) Top displacement time history



b) Top displacement time history



c) Distribution of plastic hinges



c) Distribution of plastic hinges

Life safety earthquake (**0.25g**) | Collapse prevention earthquake (**0.50g**)

Figure 3. Dynamic analysis results for f_c50f_y500-H building

The most notable difference in the response of the buildings under the three natural earthquake records can be seen in the distribution of plastic hinges among the structural members at the end of the "effective duration" of the record. Significantly more plastic hinges were formed in the buildings using the Corinth N55W record. Despite the yielding that had occurred in the columns for DC M buildings these were concentrated in the bottom of the columns, except for the f_c90f_y500-M building for which there is a high probability of the formation of a column sideway mechanism at the top three storeys, as plastic hinges also form at the top of the columns. Under the collapse prevention event, significant yielding occurred in the structural members showing that the designed buildings have the ability to dissipate the imparted seismic energy. It is notable that even for a PGA as high as 0.50g the buildings designed for DC H presented a very satisfactory performance with yielding spreading in the beams, and occasionally at the exterior base storey column. It can therefore be concluded that the code concept of designing strong columns – weak beams is satisfied in all designs. On the other hand, extensive hinging was observed in the columns of the buildings designed for DC M incorporating steel yield strength of 500 MPa, with very high probability of developing an undesirable column sideway mechanism at the top storeys. The use of higher yield steel resulted in the reduction of plastic hinges formed in the columns, achieving a better performance and larger safety margins against the collapse limit state.

CONCLUSIONS

Based on the results of both the pushover and the nonlinear dynamic analyses the performance of the buildings designed for DC M and DC H was generally deemed satisfactory under the design earthquake level (0.25g). Since at this earthquake level no failure occurred in any structural elements, it can be concluded that the resulting interstorey drifts will possibly affect only the nonstructural elements of the buildings (partitions etc.). Plastic hinges mainly formed in the beams of the buildings, which were subjected to the three natural earthquake records scaled to the design earthquake intensity.

At the collapse prevention earthquake level, inelasticity in the buildings designed for DC H was concentrated in the beam elements, while columns remained essentially elastic. On the other hand, plastic hinges formed in the columns of the buildings designed for DC M, wherein no capacity design was applied (capacity design for DC M was added as a requirement in the final 2003 version of Eurocode 8). Based on the results of the pushover analysis, exceedance of the 4% limit of interstorey drift accompanied by failure in the elements occurred in the buildings designed with 50MPa concrete strength and for DC H. Failure in the elements also occurred in the buildings designed for DC M incorporating 50 MPa concrete and HYS of 800 MPa and 1200 MPa. The performance of the buildings with different concrete strengths in the beams and the columns was very satisfactory, since even at such a high PGA the interstorey drift limitation was nowhere exceeded and failure did not occur in any element. Based on the results obtained from time-history analysis, it can be concluded that the buildings designed for DC M and incorporating 500 MPa steel may develop an unfavourable sideway mechanism at this earthquake level. The use of HYS assisted in enhancing the general performance of the buildings and from that perspective their application in the construction industry appears to be justified. It has to be clarified that a PGA of 0.50g corresponds to a region with a medium-to-high seismicity in which buildings of such importance would have been designed using specific earthquake resistant provisions.

ACKNOWLEDGEMENTS

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NOTATION

Latin Characters

d_{ei} : displacement determined by linear analysis based on the design response spectrum

f_c : compressive strength of concrete

f_y yield stress of steel

f_{yl} : yield strength of the longitudinal reinforcement

f_{yw} : yield strength of the transverse reinforcement

V : base shear

W : total weight of the structure

Greek Characters

γ_I : importance factor

δ_t : target displacement

θ : Interstorey sensitivity coefficient

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