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SEISMIC PERFORMANCE OF A PARTIALLY-COMPLETED-AND-OCCUPIED REINFORCED CONCRETE BUILDING

N. Ravi Kiran[(1)], Y. Singh[(2)]

[(1)] Research Scholar, Centre of Excellence in Disaster Mitigation and Management, IIT Roorkee, rnandyala@dm.iitr.ac.in
[(2)] Professor, Department of Earthquake Engineering, IIT Roorkee, yogenfeq@iitr.ac.in

Abstract

Prescriptive seismic codes perform well as long as they are used for designing conventional structures. Despite their shortcomings, prescriptive codes are more popular, owing to their simplicity. But, they are inherently restrictive and stifle innovation. Most inexperienced and naïve engineers believe that following the recommendations of the seismic code in letter (but not in spirit) will ensure achieving the design goals enshrined in the seismic code. In many cases, the prescriptive codes give a false sense of achieving the design goals, to the inexperienced engineers, even when they are clearly (to a discerning structural engineer) not achieved.

Many practising engineers in India understand well the behaviour of structures for gravity loads, but not earthquake loads. They do not realise that what is true for gravity loads may not be true for earthquake loads. In the past decade, several buildings in India which were approved for construction to a certain height were stopped mid-way and occupied, never to be constructed to the full height envisaged in the original design. From the gravity-design point-of-view, such buildings are safe. The general practice has been that such buildings were not systematically re-evaluated from seismic capacity point-of-view, as they were deemed safe, by extension of gravity-design practices. Since none of the clauses of the Indian seismic design code talks about such cases, it is assumed that there is no need to re-evaluate the seismic capacity of such buildings.

Clause 6.1.7 of IS 1893 part 1 directs users to ensure that any structural addition to an existing structure doesn't reduce the seismic capacity of the structural elements of the existing structure. The common misinterpretation of this clause is that only structural additions are to be assessed to ensure that the structural capacity of the modified structure meets the code requirements. Through a case study, it is shown that a building designed according to the provisions of the seismic code, when stopped mid-way (due to financial constraints) and occupied, and never to be constructed to the original design height, will not meet the performance requirements of the seismic code and hence has to be retrofitted. To this end, an assessment of a 28-storey building designed as per IS 1893 part 1 is undertaken using Eurocode 8 Part 3 approach (due to lack of equivalent recommendations in Indian codes). Also, similar assessments are carried out when the building is occupied after the construction of 9-storeys and 18-storeys respectively, to reflect the fact that construction of some buildings was stopped mid-way due to financial crisis. All the structural elements in the 9-storey building are the same as the structural elements in the 28-storey building, up to the 9th storey. Similarly, all the structural elements in the 18-storey building are the same as the structural elements in the 28-storey building, up to the 18th storey. The results show that the 9-storey and 18-storey buildings do not meet the performance criteria of the seismic code.

Keywords: IS 1893; IS 15988; DBE; MCE, EN 1998

1. Introduction

India's exponential urban growth is leading to the horizontal as well as the vertical growth of its handful of major cities. Mumbai being the commercial capital of India, has more tall buildings than the rest of India combined. The Remaking of Mumbai Federation (RoMF) partnering with the Council on Tall Buildings and Urban Habitat (CTBUH) is actively promoting the concept of sustainable tall buildings in urban India. The CTBUH 2010 World Conference jointly organised by RoMF and CTBUH is a step in that direction.

The greatest challenge - for most of the seismically active countries around the world - lies not in the design of new earthquake-resistant buildings, but the assessment and retrofitting of the existing seismically vulnerable building-stock. Therefore, it is unwise to add new seismically vulnerable buildings to the existing seismically vulnerable building-stock. The seismic risk of a building-stock is exacerbated multiple fold when the newly added seismically vulnerable buildings are tall. Hence, it is prudent to review the current Indian codes relevant to the seismic design of tall buildings and assess the performance of existing tall buildings under a possible seismic event. An attempt is made here to highlight the shortcoming of one of the clauses of the Indian seismic code, using a case study.

2. Case study

The 2001 Bhuj earthquake has changed the structural design paradigm in India. Although IS1893:1984 (fourth revision) [1] listed Bhuj in seismic zone V, the building designs in and around Bhuj did not comply with the earthquake code until after the 2001 earthquake. Most buildings in India were designed for gravity loads alone before the release of the fifth revision of the earthquake code, IS1893 part 1:2002 [2]. The sixth revision, IS1893 part 1:2016 [3] retained the seismic design philosophy introduced in the 2002 version.

Many practising engineers in India understand well the behaviour of structures for gravity loads but not earthquake loads, as the earthquake resistant design of civilian buildings is relatively a new phenomenon in India. They do not understand that what is true for gravity loads may not be true for earthquake loads. In the present work, displacement-based assessment of a 28-storey building, designed as per the forced-based design recommendations of IS1893 part 1:2002 [2] and detailed as per IS 13920 [4], is undertaken. The same building is assessed when the building is occupied after the construction of 9-storeys and 18-storeys. The seismic assessment of the 9-storey and 18-storey buildings is done to reflect the real life situations where the construction of some buildings was stopped mid-way due to the 2008 global economic recession.

The building chosen for the present study was designed for the Indian seismic zone II, using Response Spectrum Method, as per the guidelines of IS 1893 (Part 1) [2]. Although the overall design philosophy of the 2002 and 2016 versions of the code, in principle, remains the same, there are some minor differences in the design approach. The 2002 version suggests to take the gross stiffness of the members for the analysis of the buildings while the 2016 version suggests to take the effective stiffness of the members for the analysis of the buildings, in line with accepted international design practices. The case-study (28-storey) building has been designed using gross stiffness of members in the analysis, while the assessment has been carried out using effective stiffness of the members, using Eurocode 8 approach. The latest revisions of the Indian seismic code for the design of new reinforced concrete buildings as well as the seismic code for the assessment of existing reinforced concrete buildings [5] are force-based. The advantages of displacement-based assessment over force-based assessment are well accepted by the international scientific and design community. Hence, the assessment is done as per the guidelines of Eurocode 8 part 3. The purpose of the 9-storey and 18-storey models is to check whether the building would remain safe if an earthquake strikes when the building is built up to 9 storeys (approximately 1/3rd the height) or 18 storeys (approximately 2/3rd the height), and occupied. The 9-storey, 18-storey and 28-storey building models have been assessed as per the guidelines of Eurocode 8 [6, 7], since the current Indian code on the seismic assessment of existing buildings doesn't have guidelines on displacement-based seismic performance assessment.

The case-study building is a 28-storey reinforced concrete building with a floor-to-floor height of 3.6m throughout the height of the building. The building has five basement floors for parking which are not modelled in this study. Also, there is a floor above the 28th floor of the building which houses lift rooms and other services, which is ignored as well in the model for the assessment of the building. The site of the building is located in a hilly area, 570m above the mean sea level. Since the building is founded on rocky terrain with very high safe bearing capacity; raft foundation is used as the foundation for the building.

The importance of modelling and the choice of analysis procedure in the assessment of structures cannot be overstated. The case-study building is modelled in and assessed using the software ANSRuop [8], a substantially extended and improved version of ANSR-I [9]. Currently, Nonlinear Time History Analysis is widely accepted as the most reliable among the available analysis methods, for the performance evaluation/ assessment of structures; especially for tall buildings [10, 11, 12, 13]. Therefore, the seismic performance assessment of the case-study building is carried out using Nonlinear Time History Analysis.

3. Assumptions in the modelling, analyses and assessment of the case-study building

The key assumptions in the nonlinear material modelling and the analyses used in the present study are:

1. All the members are modelled using 3D prismatic beam elements. Non-rectangular walls are modelled with a single prismatic element per storey, at the shear centre of the wall. A point hinge model with bilinear M-θ curve is used for seismic loading. Nonlinear Time History Analyses are performed using the modified Takeda hysteresis rule, with the unloading and reloading parameters $\alpha = 0.3$ and $\beta = 0$.
2. The elastic stiffness $(EI)_{eff}$ of a member is taken as the mean secant-to-yield-point stiffness of the two member end sections in flexure. The calculation of $(EI)_{eff}$ is based on the shear span at the yielding member end(s) and on the member axial force due to the presence of gravity loads alone.
3. Shear span of various members is taken as follows:
   a. For Primary Beams/Columns: half the clear span within the plane of bending.
   b. For Secondary Beams: clear span between the supports.
   c. For Wall elements (between two successive stories): 50% of the wall height from the bottom section of the storey to the top of the wall in the model.

   For primary beams supporting secondary beams at intermediate points, the shear span is taken as the clear span between the columns supporting the primary beam.
4. The effective flange width of a T- or L-beam, in tension or compression, on either side of its web is taken as half of the smallest value among the beam shear span and the distance to the nearest adjacent parallel beam. Slab reinforcement bars parallel to the beam and falling within this 'effective width' are assumed to act as the longitudinal reinforcement of the beam section.
5. The flexural strength and stiffness of columns are considered as independent in the two orthogonal horizontal directions of loading. The yield moment M_y of each column element is calculated using the axial force at each time step and updated.
6. Rigid elements are used to model eccentric connections between members.
7. Joints are assumed as rigid. However, the effect of the member fixed-end rotations on the secant-to-yield-point stiffness and the ultimate chord rotation is accounted for, along with the slippage of longitudinal reinforcement through or from a joint.
8. The in-plane floor flexibility is incorporated in the model at the level of individual floor panels in plan, by considering the in-plane parameters of the panel boundary beams including the effects of the floor (diaphragm) panels on both sides of a beam, as suggested in [14].
9. Staircases are excluded from the model.

10. P-Δ effects are considered in the model.
11. Lumped masses are used in the model.
12. Rayleigh damping coefficients are calculated using a common damping ratio of 5% and the angular frequencies corresponding to the average and half the average of the first two periods with the highest base shear, in the two orthogonal horizontal directions.
13. Foundations are modelled as fixed as they are assumed capable of providing fixity to columns.
14. Demand-to-Capacity Ratios (DCR) are used to evaluate damage at member ends. Flexural capacity of members is estimated in terms of the ultimate chord rotations, using the empirical relations given in Annex A of Eurocode 8 Part 3 [7]. Shear capacity is evaluated considering diagonal tension failure after yielding and diagonal compression failure before or after yielding. The demand-to-capacity ratios of columns in the two orthogonal horizontal directions of loading are combined via the SRSS rule. In the calculation of the demand-to-capacity ratio both the demand and capacity values are updated at each time increment. The maximum value of the demand-to-capacity ratio during the entire response is reported at the end. The value of the demand-to-capacity ratio near 1.0 signifies incipient failure.

3.1 Simplifications in the mathematical model

Due to the problems encountered during the time history analyses of the models due to their enormous size and complexity, simplifications were made in the mathematical models, to reduce the size and complexity of the models, by not modelling staircases and ignoring foundation flexibility.

3.2 Cross-section and reinforcement details of structural elements

A typical floor slab and beam layout of the case-study building is shown in Fig. 1. Figure 2 shows a typical floor layout of the mathematical model in ANSRuop. The slab and beam reinforcements are maintained the same in all the floors in the mathematical model although there are slight variations in the design of the actual building. The shearwalls are also modelled uniform throughout the height of the building although there are some minor variations in the reinforcement details over the height in the design of the actual building. However, the changes in the column dimensions and reinforcement details over the height, in the design of the actual building, are accurately reflected in the model. As mentioned earlier, the contribution of slab to the effective flange width of the beam is modelled.

3.3 Seismic Hazard considered for Nonlinear Analysis

The zone factor for seismic zone II is 0.1, which as per the 2002 version of IS1893 part 1 [2] reflects the realistic value of the effective peak ground acceleration considering the Maximum Considered Earthquake (MCE). The 2002 version considers MCE as the largest reasonably conceivable earthquake that appears possible along a recognised fault or within a tectonic province, irrespective of the return period of the earthquake. It also assumes that the Design Basis Earthquake (DBE) hazard as one half of the MCE hazard.

3.4 Limit States for Performance Assessment

The choice of the Limit States (LSs) for Performance Assessment as well as the probabilities of exceedance of the seismic hazard ascribed to the various LSs, depends on the programme adopted for the seismic assessment and retrofitting. In the assessment of the case study building, the No Collapse (NC) LS for the MCE motion and the Significant Damage (SD) LS for the DBE motion are considered. Annex A of Eurocode 8 Part 3 suggests that the required chord rotation capacities at the SD LS can be taken equal to 0.75 times the ultimate chord rotation capacity of the members [7]. In a Linear Time History Analysis, estimated chord rotation demands are proportional to the Peak Ground Acceleration (PGA) of the input ground motion. So, the SD LS will govern, if the PGA of the input ground motion is greater than 75% of the PGA of the input ground motion for the NC LS. Conversely, if it is below 0.75. In either case, checking both LSs is redundant. This conclusion may be extended to nonlinear time history analysis as well, provided that the ratio of the PGA values of the input ground motions of the SD and NC LSs is not close to 0.75 [14]. In the present case study, the ratio of the PGA values of the input ground motions of the SD and NC LSs is 0.5. Hence, checking the NC LS alone will suffice.

3.5 Ground motions used for Nonlinear Analyses

The 9-storey, 18-storey and 28-storey building models are subjected to nonlinear time history analyses using modified historical bi-directional (horizontal) ground motions. Each ground motion emulates the two horizontal components of seven historic earthquakes, with each component modified to fit the 5%-damped elastic acceleration response spectrum. Both the components of each horizontal bi-directional input ground motion are normalised to a PGA of 0.1g. Using both the components of each horizontal bi-directional input ground motion, a set of four input ground motions can be generated by combining each component in both the positive and the negative sense of the component direction with the other component in both the positive and the negative sense of the component direction. Similarly, a set of four ground motions can be generated by interchanging the components. Thus, the components of each horizontal bi-directional ground motion can be used to produce a set of eight ground motions. So, 7×8 = 56 earthquakes can be generated using the seven historical time history records. However, the building being somewhat regular, as is evident from the Centre of Mass and Centre of Stiffness values reported in section 4.1, the combinations in which the component in the X direction of the building applied in the positive sense alone are considered for the nonlinear analyses, giving four orientations of the components of each horizontal bi-directional input ground motion. So, 7×4 = 28 nonlinear time history analyses have been carried out on each model.

The following seven historic earthquake records are modified and used for the assessment.

1. The 1976 Tolmezzo earthquake (Friuli, Italy).
2. The 1979 Ulcinj earthquake (Montenegro).
3. The 1979 Herceg Novi earthquake (Montenegro).
4. The 1986 Kalamata earthquake (Greece).
5. The 1989 Loma Prieta earthquake (California, U.S.A) recorded at the Capitola building.
6. The 1979 Imperial Valley earthquake (California, U.S.A) recorded at Bonds Corner.
7. The 1940 Imperial Valley earthquake (California, U.S.A) recorded at El Centro.

As mentioned earlier, the assessment is carried out using ANSRuop as per the guidelines of Eurocode 8 parts 1 and 3. ANSRuop has the option to calculate the demand-to-capacity ratios in flexure and shear of the columns as per Eurocode 8 part 3, apart from the options to calculate the demand-to-capacity ratios in flexure for columns using the yield chord rotation as capacity or ultimate chord rotation as capacity. The results of the demand-to-capacity ratios in flexure calculated as per the No Collapse Limit State of Eurocode 8 part 3 should yield comparable results to that of the demand-to-capacity ratios calculated using the ultimate chord rotation as capacity. The demand-to-capacity ratios in flexure for columns are calculated using the above three options. Although the No Collapse Limit State is the topic of interest of this assessment, the yield chord rotation as capacity option is used to see if the building enters the inelastic range in flexure and the ultimate chord rotation as capacity option is used to see if the No Collapse Limit State and the ultimate chord rotation as capacity option yield comparable results. The demand-to-capacity ratios in shear for columns for the No Collapse Limit State are calculated too.

4. Results and observations

4.1 Centre of Mass and Centre of Stiffness

From Fig.2, it can be seen that the eccentricities in X and Z directions, 0.79 m and 0.26 m respectively, are far less than 1.81 m and 1.34 m (5% of the floor-dimension perpendicular to the direction of the seismic action), the values of the code prescribed accidental eccentricity in the respective directions.

4.2 Periods of vibration

The periods of vibration (in seconds) of the first 3 modes and the effective modal mass (in percentage) in the plan directions (X and Z), in each mode, of the 9-storey, 18-storey and 28-storey buildings are presented in Table 1, from which it can be observed that the second and third modes of all the three building models are well separated while the first and second modes of all the three building models can be treated as not closely spaced as per the conventional definition of closely spaced modes. Using the measured data on real buildings during moderate earthquakes, most of the international seismic codes usually give an approximate formula for the estimation of the period of the 1st mode of vibration which is proportional to the height of the building raised to the power of ¾. Using this conventional knowledge, one can expect that the ratio of the periods of the first mode of the 18-storey and 9-storey buildings is less than 1.7 and that of the 28-storey and 9-storey buildings is less than 2.3. However, it is observed that the ratio of the periods of the first mode of the 18-storey and 9-storey buildings is more than 2 and that of the 28-storey and 9-storey buildings is more than 3. The fact that the 9-storey and 18-storey buildings are much stiffer than what is required of them, had they been designed as 9-storey and 18-storey buildings as per the guidelines of the Indian seismic code, in the first place, can be attributed as the primary reason for this deviation from the conventional wisdom. Due to this peculiar situation, the 9-storey and the 18-storey buildings attract much higher base shear than what they would, had they been designed as 9-storey and 18-storey buildings in the first place. As can be seen from the Table 2 below, although the seismic mass of the 9-storey building is around 1/3rd of that of the 28-storey building, the MCE lateral force coefficient of the 9-storey building is 3.7 times that of that of the 28-storey building. Thus, the base shear of the 9-storey building (partially-completed-and-occupied 28-storey building) is higher than the 28-storey building designed to meet the requirements of the seismic code.

Table 1 – First 3 modes of the 9-storey, 18-storey and 28-storey building models

	9-storey model			18-storey model			28-storey model		
	Mode 1	Mode 2	Mode 3	Mode 1	Mode 2	Mode 3	Mode 1	Mode 2	Mode 3
Period (in sec)	1.64	1.47	1.24	3.36	3.03	2.48	6.04	5.48	4.48
Effective Modal Mass in X (%)	55.16	0.94	24.72	56.49	0.31	16.48	59.70	0.19	12.81
Effective Modal Mass in Y (%)	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Effective Modal Mass in Z (%)	0.16	78.86	1.40	0.51	72.80	0.03	0.42	72.48	0.14

Table 2 – Base Shear of the 9-storey, 18-storey and 28-storey building models

	9-storey model	18-storey model	28-storey model
Seismic Weight (in kN)	219184	430997	639647
S_a/(Zone Factor × g) (as per [2])	0.611	0.297	0.165
DBE lateral force coefficient	0.0305	0.0149	0.0083
MCE lateral force coefficient	0.0611	0.0297	0.0165
DBE Base Shear (in kN)	6685	6422	5309
MCE Base Shear (in kN)	13392	12801	10554

Also, it is evident that the first two modes are predominantly translational (along X and Z directions) for all the three building models and the effective modal mass in X direction in the first mode is increasing with the increase in the height of the building while the effective modal mass in Z direction in the second mode is decreasing with the increase in the height of the building. Thus, it can be inferred that the contribution of the first mode response to the overall building response is higher in the case of the 28-storey building compared to the 9-storey and 18-storey buildings.

4.3 Performance assessment as per Annex A of Eurocode 8 part 3

The definitions of column damage index (demand-to-capacity ratios) for flexure and column damage index for shear as defined in [14] are adopted for the assessment of the case-study building. The demand-to-capacity ratios in flexure and shear of the 9-storey, 18-storey and the 28-storey models computed for a set of fourteen bi-directional horizontal MCE level ground motions are presented here. Due to space constraints, only a few important figures are presented here. The column damage indexes in flexure considering utlimate chord rotation as the capacity, the column damage indexes in shear for the NC Limit State as per Eurocode 8 part 3 and the column damage indexes in flexure considering yield chord rotation as the capacity, for the 9-storey, 18-storey and 28-storey building models, under the MCE level ground motions, are presented through the Figs. 3 to 17. From Figs. 3, 4, 5, it is evident that all the building models, i.e. the 9-storey, 18-storey and 28-storey buildings meet the performance requirements in flexure (considering utlimate chord rotation as the capacity), when subjected to MCE level ground motions. From Figs. 6, 7, 8, it is evident that the 9-storey building does not meet the performance requirements in shear, while from Fig. 9, it is clear that the 9-storey building enters the inelastic range (considering yield chord rotation as the capacity), when subjected to MCE level ground motions. From Figs. 10, 11, 12, it is evident that the 18-storey building does not meet the performance requirements in shear, while from Fig. 13, it is clear that the 18-storey building is very close to entering the inelastic range (considering yield chord rotation as the capacity), when subjected to MCE level ground motions. From Figs. 14, 15, 16, it is evident that the 28-storey building meets the performance requirements in shear, while from Fig. 17, it is clear that the 28-storey building is close to entering the inelastic range (considering yield chord rotation as the capacity), when subjected to MCE level earthquake ground motions.

5. Conclusions

From the results, it is clear that the 28-storey building satisfies the No Collapse Limit State for the Maximum Considered Earthquake motions and the Significant Damage Limit State for the Design Basis Earthquake motions (since the PGA ratio is less than 0.75). However, that is not the case with the 9-storey and 18-storey buildings. The shear demands on some of the columns in the row of columns next to the front elevation exceed the shear capacity of columns for the No Collapse Limit State for the Maximum Considered Earthquake motions, in case of the 18-storey building and the shear demands on some of the columns in the back elevation and the row of columns next to the front elevation exceed the shear capacity of columns for the No Collapse Limit State for the Maximum Considered Earthquake motions, in case of the 9-storey building. The reason for shear being more critical as the number of storeys decreases can be attributed to the decrease in shear resistance with decreasing axial load or to the shorter periods (and hence higher force demands) in the shorter buildings or to both. Both reasons are applicable to the 18-storey building. However, the second reason does not apply for the 9-storey building since some of the shear–critical members are beyond flexural yielding (i.e., under the seismic action level considered, the structure is beyond the Damage Limitation (DL) Limit State). Hence, the shear force demands in those members depend on the member yield moments and not on the elastic shear force demands, that increase with the decreasing period.

Thus, from the analyses results, it can be concluded that the design of the 28-storey building meets the performance requirements. However, if the construction is stopped at 1/3rd or 2/3rd the height of the building and occupied, then the resulting buildings are uncharacteristically stiff compared to buildings of same height designed to meet the requirements of the seismic design code. Such partially-constructed-and-occupied-buildings are unlikely to meet the performance requirements of the seismic design code. Particularly, the column damage indexes in shear, of such partially-constructed-and-occupied-buildings, under MCE level ground motions need to be re-evaluated.

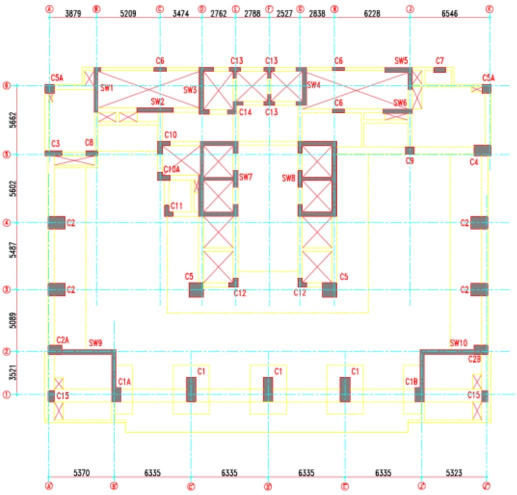

Fig. 1 – Typical floor slab and beam layout

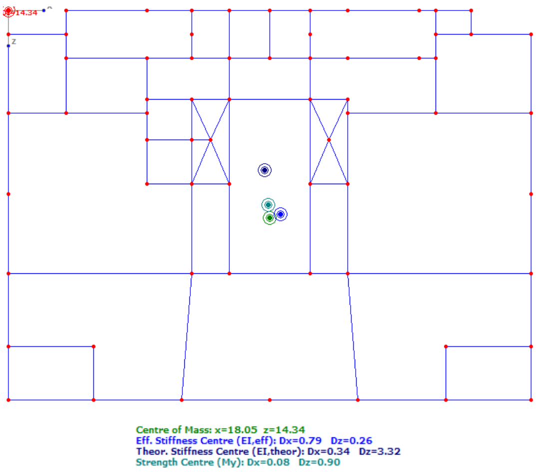

Fig. 2 – Centre of Mass and Centre of Stiffness

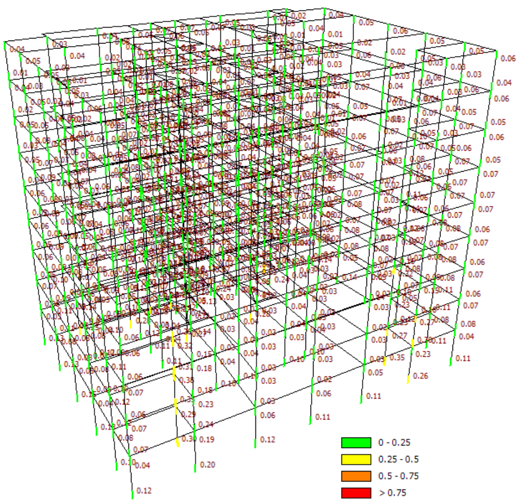

Fig. 3 – Column demand-to-capacity ratios in flexure (ultimate); average values from the nonlinear time history analyses of the 9-storey building subjected to the 14 bi-directional horizontal ground motions

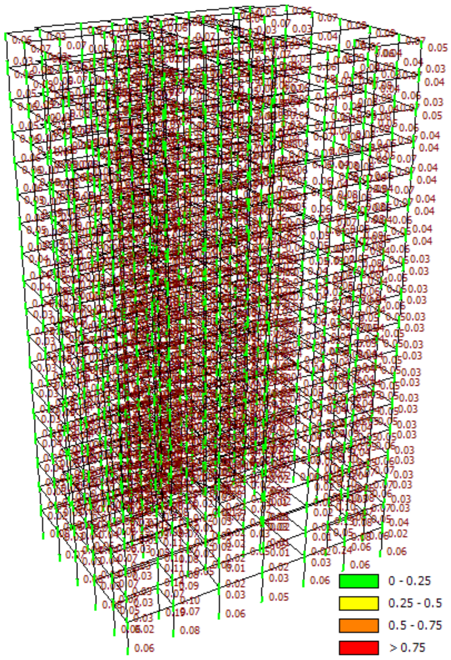

Fig. 4 – Column demand-to-capacity ratios in flexure (ultimate); average values from the nonlinear time history analyses of the 18-storey building subjected to the 14 bi-directional horizontal ground motions

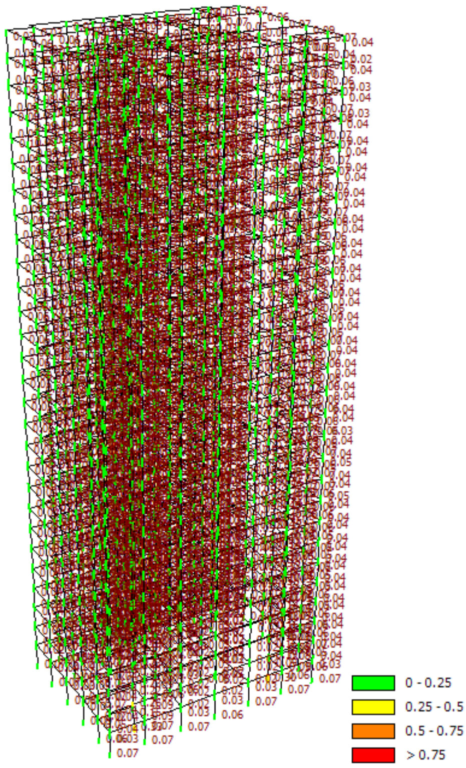

Fig. 5 – Column demand-to-capacity ratios in flexure (ultimate); average values from the nonlinear time history analyses of the 28-storey building subjected to the 14 bi-directional horizontal ground motions

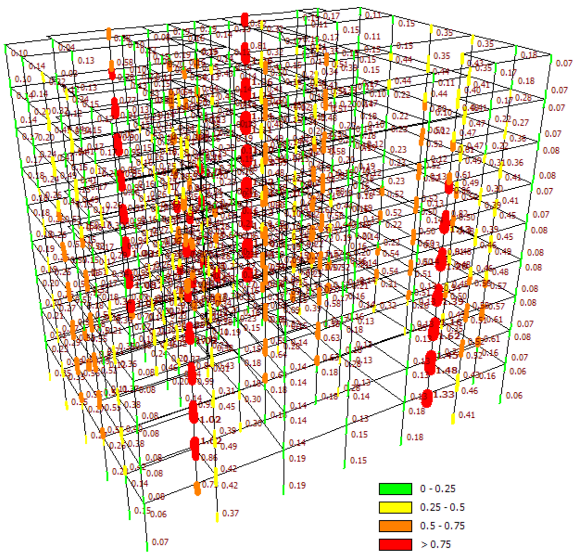

Fig. 6 – Column demand-to-capacity ratios in shear for the No Collapse Limit State as per Eurocode 8 part 3; average values from the nonlinear time history analyses of the 9-storey building subjected to the 14 bi-directional horizontal ground motions

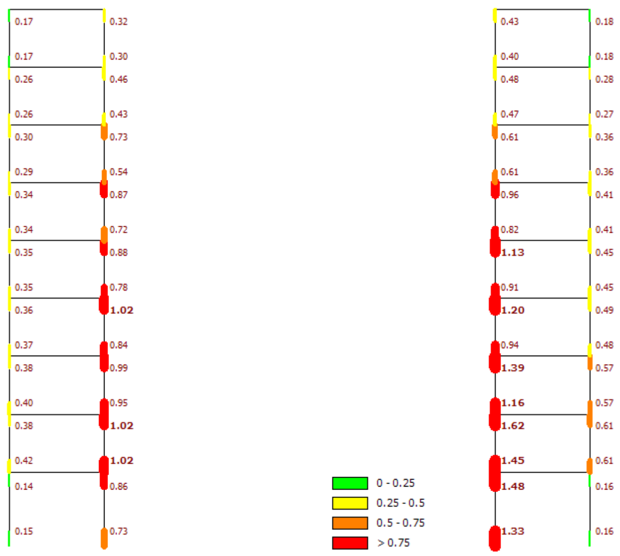

Fig. 8 – Column demand-to-capacity ratios in shear of the row of columns next to the front elevation; average values from the nonlinear time history analyses of the 9-storey building subjected to the 14 bi-directional horizontal ground motions

Fig. 7 – Column demand-to-capacity ratios in shear of the columns of the back elevation; average values from the nonlinear time history analyses of the 9-storey building subjected to the 14 bi-directional horizontal ground motions

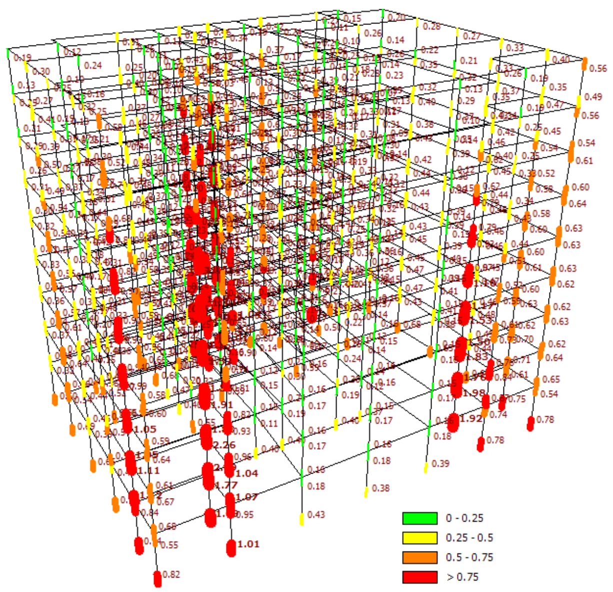

Fig. 9 – Column demand-to-capacity ratios in flexure (yielding); average values from the nonlinear time history analyses of the 9-storey building subjected to the 14 bi-directional horizontal ground motions

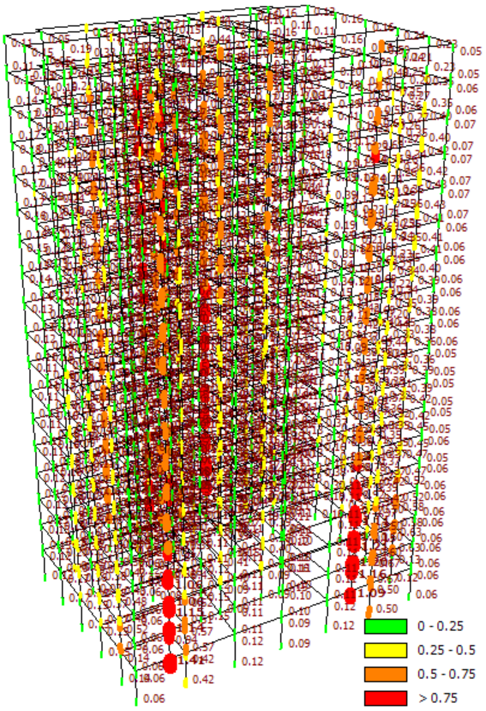

Fig. 10 – Column demand-to-capacity ratios in shear for the No Collapse Limit State as per Eurocode 8 part 3; average values from the nonlinear time history analyses of the 18-storey building subjected to the 14 bi-directional horizontal ground motions

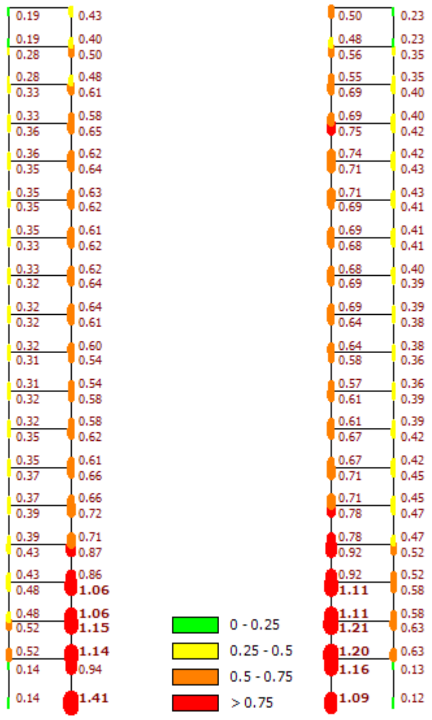

Fig. 12 – Column demand-to-capacity ratios in shear of the row of columns next to the front elevation; average values from the nonlinear time history analyses of the 18-storey building subjected to the 14 bi-directional horizontal ground motions

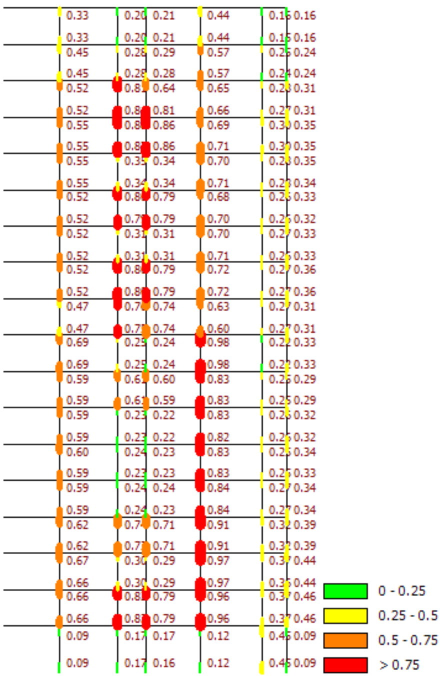

Fig. 11 – Column demand-to-capacity ratios in shear of the columns of the back elevation; average values from the nonlinear time history analyses of the 18-storey building subjected to the 14 bi-directional horizontal ground motions

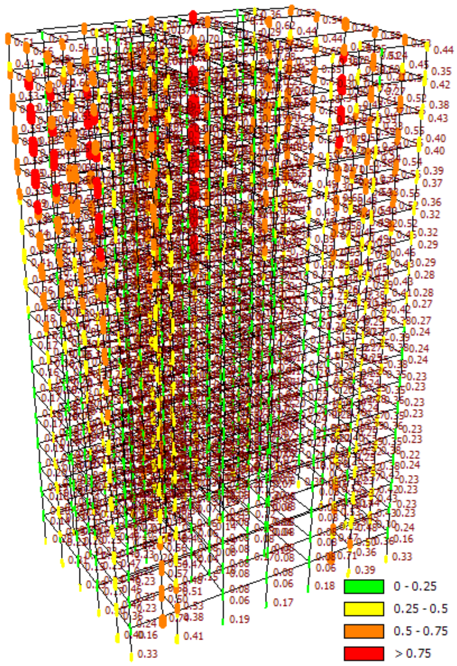

Fig. 13 – Column demand-to-capacity ratios in flexure (yielding); average values from the nonlinear time history analyses of the 18-storey building subjected to the 14 bi-directional horizontal ground motions

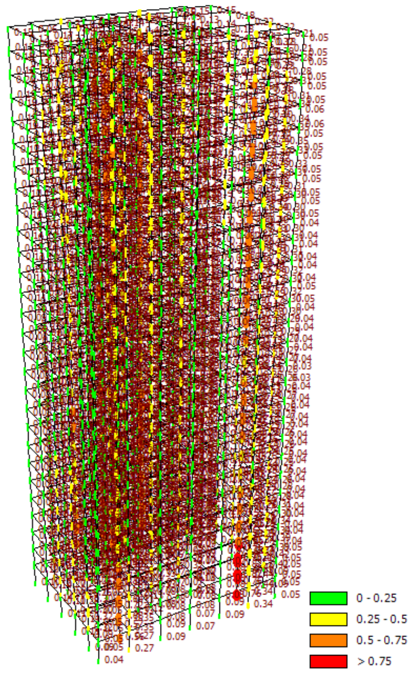

Fig. 14 – Column demand-to-capacity ratios in shear for the No Collapse Limit State as per Eurocode 8 part 3; average values from the nonlinear time history analyses of the 28-storey building subjected to the 14 bi-directional horizontal ground motions

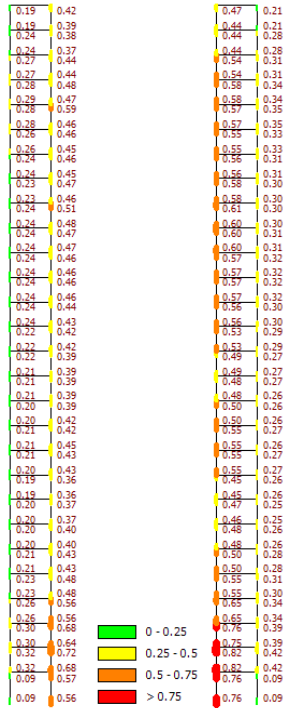

Fig. 16 – Column demand-to-capacity ratios in shear of the row of columns next to the front elevation; average values from the nonlinear time history analyses of the 28-storey building subjected to the 14 bi-directional horizontal ground motions

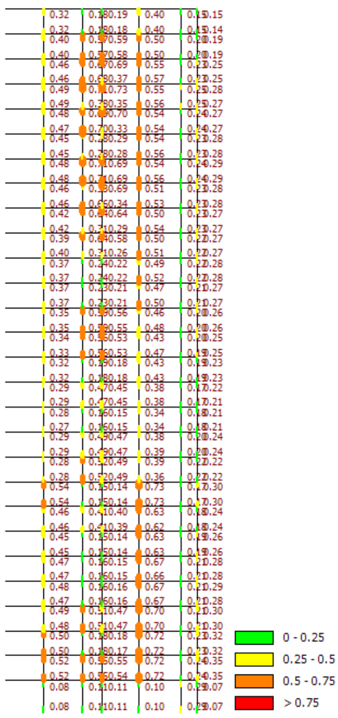

Fig. 15 – Column demand-to-capacity ratios in shear of the columns of the back elevation; average values from the nonlinear time history analyses of the 28-storey building subjected to the 14 bi-directional horizontal ground motions

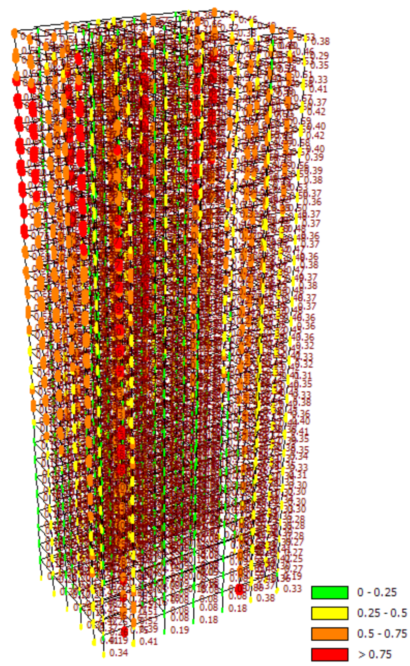

Fig. 17 – Column demand-to-capacity ratios in flexure (yielding); average values from the nonlinear time history analyses of the 28-storey building subjected to the 14 bi-directional horizontal ground motions

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