BENCHMARK ASSESSMENT OF PROTOTYPE RC BUILDING
ACCORDING TO EN1998-3

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ABSTRACT

A prototype six-story RC building, typical of 1970-80s Israeli construction, was assessed for the design earthquake specified by the Israeli code for low, medium and high seismicity zones, with PGA 0.09g, 0.18g and 0.26g, using EN 1998-3:2005, as adapted to Israeli Standard 413-3. All analysis methods find the columns to be shear-deficient in all three zones. In flexure, nonlinear time-history analysis shows the Significant Damage Limit State to be met for low or medium seismicity, but the Damage Limitation State not, and for high seismicity the Significant Damage Limit State to be violated but Near Collapse to be met. Linear analysis with q=1 leads to the same conclusions, but is not applicable for high seismicity. The q-factor approach with q=1.5 is overly safe-sided in flexure. Pushover analysis gives slightly less conservative results than nonlinear time-history analysis.

Keywords: Concrete buildings; Eurocode 8; Nonlinear analysis; Seismic assessment; Seismic evaluation

1. INTRODUCTION

A prototype six-story RC building, with beam-column frames on the perimeter and flat slab frames in the interior (Fig. 1), typical of 1970-80s Israeli apartment construction, was assessed to EN 1998-3:2005, as adapted to Israeli Standard 413-3, for the design earthquake specified in Israel for low, medium and high seismicity. Fig. 2 shows the corresponding elastic spectra with peak ground acceleration (PGA) 0.09g, 0.18g and 0.26g. The concrete has mean strength 22 MPa; the longitudinal steel has mean yield stress 400 MPa and the ties 220 MPa. Knowledge Level 2 (Normal Knowledge) applies. Floor loads, beyond self weights, are 3.5 kN/m² permanent and 2 kN/m² live load. The subgrade reaction modulus, kss, is 120000/b kPa/m, with b (m) the minimum size of the footing in plan.

2. ANALYSIS AND MODELLING

The types of analysis applied were:
1. Linear analysis with the 5%-damped elastic spectrum for the deformations and equilibrium in the plastic hinge mechanism for the shear forces, with applicability conditions in Eurocode 8;
2. Non-linear static (pushover) analysis, under uniform and inverted triangular (1st mode) forces;
3. Non-linear response-history analysis, for seven spectrum-compatible pairs of ground motions;
4. The behavior factor approach with the 5%-damped elastic spectrum divided by a q-factor of 1.5.

In nonlinear time-history analysis seven pairs of natural ground acceleration time-histories were used as concurrent horizontal components of the seismic action, modified according to the single impulse method (Al Atik & Abrahamsson 2010) to be compatible with the elastic response spectra (Fig. 2).

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In linear analysis and the behavior factor approach maximum seismic action effects due to the individual horizontal components of the seismic action are combined through the SRSS rule. Action effects estimated individually in this way were conservatively assumed to take place at the same time. In pushover analysis Eurocode 8 requires to apply concurrent lateral force patterns in the two horizontal directions which correspond to 100% of the target displacement in each horizontal direction and to 30% of the target displacement in the orthogonal one. This was accomplished in three stages:
1. Preliminary uni-directional pushover analyses were carried out, separately in each direction.
2. The magnitude of uni-directional lateral forces which produced 100% and 30% of the target displacement in their own direction during Stage 1 was determined.
3. The lateral forces producing 100% of the target displacement in one direction were incrementally applied concurrently with those giving 30% of the target displacement in the orthogonal one. The only way to take accidental eccentricities of masses (5% of the orthogonal plan dimension) into account in nonlinear analysis is by shifting masses from their nominal position in the model, in the same direction at all floors. As the two horizontal components are applied concurrently, masses are shifted concurrently in both horizontal directions. Certain masses on or near each one of the axes of symmetry were increased over one half of the floor plan and their symmetric ones over the other half were reduced by equal amounts, so that the 5% overall eccentricity is achieved. For consistency across models the same eccentric models are used in linear analyses; so, the simplified handling of accidental eccentricities allowed by Eurocode 8 if linear analysis is used (the results of static analysis for the story torques produced by the bi-axial eccentricities superimposed in absolute value terms to those due to the concentric horizontal components, even when the modal response spectrum method is used for these components) does not become a source of differences in the results of analysis methods. Thanks to the double symmetry in plan, shifting of the masses is done only in the positive direction of both horizontal axes. The unfavorable results among all four quadrants in plan apply then in all quadrants.

Prismatic beam/column elements are used in the model, with masses lumped at the nearest node. Flat slabs are modeled as beams with effective width one-third of the length of the spans on either side of the idealized horizontal element between the columns or beams in which the flat slab element frames (Hwang & Moehle 2000). Columns are modeled with independent elements in each bending plane. The secant-to-yield-point stiffness specified in Annex A of EN1998-3 is used as initial stiffness of every member. The shear span of beams or columns is taken as half the clear length from one beam-column joint to the next within the plane of bending. Beams framing into a column at one end but indirectly supported on another beam at the other, have shear span equal to the clear span of the beam. For the calculation of the effective stiffness, the effective flange width of T- or L-beams in tension and compression is equal to 50% of the shear span or of the distance to the adjacent parallel beam (whatever is less) on either side of the web; slab bars that fall within this flange width and are parallel to the axis of such a beam are included in the top reinforcement of the beam’s end section.

Yield properties of columns are computed from the initial value of axial force, due to gravity loads. Joints are considered as rigid, but slippage of bars of members framing in the joint, is reflected in the member’s effective stiffness according to EN 1998-3. The in-plane flexibility of floor diaphragms is taken into account by assigning an in-plane flexural stiffness and an axial stiffness to the beams around a diaphragm panel already in the model for out-of-plane bending, according to Fardis (2009).

Soil impedance $K_\varphi$ against rotation about axis x of a footing in plan is assigned to any support node:

$$K_\varphi = k_s \frac{A_t b_t^2}{5.75}$$  \hspace{1cm} (1)

where $A_t$ is the area of the footing in plan and $b_t$ its size at right angles to axis x. Similarly for rotation about axis z. With this case value footings are much closer to hinged than to fully fixed.

P-$\Delta$ effects were taken into account in the nonlinear analyses. The hysteretic rules used in the nonlinear response-history analyses are of the modified Takeda type, (Fardis 2009) with unloading parameter $\alpha = 0.3$ and reloading parameter $\beta = 0$.

Mass- and initial-stiffness proportional Rayleigh viscous damping was used in the nonlinear response-history analyses, with 5% damping ratio specified at the fundamental period and at one-half its value.

3. MODAL ANALYSIS AND VERIFICATIONS BASED ON LINEAR ELASTIC ANALYSIS

Table 1 lists the periods and participating mass ratios, neglecting or including accidental eccentricities. Fig. 3 compares shapes of the first mode per horizontal direction with or without eccentricities. Mode shapes are strongly affected by the eccentricities, but natural periods are less sensitive. The lateral force procedure does not apply, as the fundamental period is longer than four times the corner period $T_c$ between the constant-acceleration and the constant-velocity regions of the spectrum.
Table 1. Natural modes and periods.

<table>
<thead>
<tr>
<th>mode</th>
<th>Without accidental eccentricities</th>
<th>With accidental eccentricities</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T (s)</td>
<td>modal mass X</td>
</tr>
</tbody>
</table>
| 1    | 2.05  | 0            | 2.12  | 16.1%         | translational | translational-
torsional |
| 2    | 1.92  | 85.8%        | 2.03  | 59.1%         | translational | torsional |
| 3    | 1.78  | 0            | 1.73  | 9.3%          | pure torsion | torsional |
| 4    | 0.70  | 0            | 0.73  | 0.7%          | translational | torsional |
| 5    | 0.64  | 8.8%         | 0.69  | 7.2%          | translational | torsional |
| 6    | 0.61  | 0            | 0.59  | 1.2%          | pure torsion | torsional |
| 7    | 0.41  | 2.3%         | 0.43  | 0             | translational | torsional |
| 8    | 0.36  | 2.3%         | 0.39  | 2.3%          | translational | torsional |
| Sum  | 97%   | 97%          | Sum  | 95.9%         | 97.1% |

Figure 3. First (top) and second (bottom) mode shapes without (left) or with (right) accidental eccentricities.

Fig. 4 shows that the elastic-moment-demand-to-capacity-ratio for beams or columns in low seismicity does not vary by more than a factor of 2.5 among primary seismic elements (flat slab elements do not count, as they are secondary elements) and linear analysis applies. The same holds for medium seismicity, but not for high, as the range of the ratio exceeds 2.5. Chord rotation demands from modal response spectrum analysis with q=1 meet the verification in flexure for low or medseismicity at the Significant Damage Limit State but not at the Damage Limitation one (see Figure 5 for columns).
Figure 4. Ratio of elastic moment from modal response spectrum analysis of the building in low seismicity zone to the moment resistance, for applicability of linear analysis: (top) beams; (bottom) columns in biaxial bending.

Figure 6 shows that most columns (especially in the weak direction) do not meet the shear verification which accompanies linear analysis for the flexural deformation demands, i.e., with shear force demand estimated as in capacity design, independent of Limit State, analysis method or design seismic action.
Figure 5. Ratio of biaxial chord-rotation demands on columns from modal response spectrum analysis in low seismicity zone, to limits at Significant Damage (top) and Damage Limitation (bottom) Limit States.

4. ASSESSMENT BASED ON NONLINEAR ANALYSIS

The verifications in flexure show that, under the design seismic action, the Significant Damage Limit State is met with a good margin in the low seismicity zone, but the Damage Limitation State is not met
at column tops (Figure 7) and in few beams. In the moderate seismicity zone the Significant Damage Limit State is met, but the Damage Limitation State is not. The building in the high seismicity zone violates the Significant Damage Limit State but meets the Near Collapse one (Figure 8).

Figure 6. Ratio of capacity-design-shear to shear resistance in the two directions of the columns.
The shear resistance fails to meet the shear force demands due to the design seismic action from nonlinear response-history analysis, in few columns at several floors for low seismicity (Figure 9), in several ones at most floors for moderate seismicity and in most columns and floors for high seismicity. The "uniform" force pattern in pushover analyses gave more unfavorable seismic demands than the "modal" pattern (approximated, for consistency across the seismicity zones by the force pattern used in the lateral force procedure, although the latter applies only for low seismicity).
Figure 8. Average (over seven nonlinear response-history analyses) ratio of biaxial chord-rotation demands on columns in high seismicity zone to limits of Significant Damage (top) and Near Collapse (bottom) Limit State.
Figure 9. Ratio of shear force demand on columns in low seismicity zone, to the cyclic shear resistance in the plastic hinge, from (top) average over seven bidirectional nonlinear response-history analyses; (bottom) pushover analysis under bidirectional uniform force patterns.

Figure 10 shows capacity curves from the uniform force pattern on the same plot with composite (acceleration-displacement) spectra for low and high seismicity. Exceedance of a Limit State for the
first time at any beam or column is marked on each capacity curve. As the secant-to-yield-point stiffness specified in Annex A of EN1998-3 has been used as initial stiffness of every member, the target displacement due to the design seismic action is at the projection on each capacity curve of the intersection of its initial tangent and the composite spectra. In general, pushover analysis gave smaller estimates than nonlinear response-history analysis; the case in Figure 9 is one of the few exceptions.

![Figure 10. Capacity and demand curves in pushover analysis for uniform force pattern. Low or high seismicity](image)

5. ASSESSMENT BASED ON THE BEHAVIOR-FACTOR APPROACH

The q-factor approach with q=1.5 shows the building to be deficient in flexure and in shear at all three zones, to a degree and extent that increases in severity from low (Figure 11) to high seismicity. For shear, this is the right conclusion, but in flexure the verdict is overly safe-sided.

6. CONCLUDING REMARKS

- Under the design seismic action, shear resistance does not meet the shear demand in few columns of several floors for low seismicity (PGA 0.09g), in several columns of most floors for medium seismicity (PGA 0.18g) and in most columns of every floor for high seismicity (PGA 0.26g).
- The Significant Damage Limit State is met for low seismicity, but the Damage Limitation State is not. For medium seismicity the Significant Damage Limit State is met, but the Damage Limitation State is not. For high seismicity the Near Collapse is met but the Significant Damage one is not.
- Although linear analysis is not applicable at the high seismicity zone, it delivers the same verdict as the nonlinear response history method.
- The q-factor approach with q=1.5 concluded that the building is deficient in flexure and in shear at all three zones, to a degree and extent that increases in severity from low to high seismicity. So, for shear it gives the right answer, but is overly safe-sided for flexure.
- The "uniform" lateral force pattern in pushover analyses produced overall more unfavorable seismic demands over the structure, compared to the "modal" lateral force pattern.
- Overall, pushover analyses gave more favorable results than nonlinear response-history analyses. Detailed information on the input and the outcomes of all approaches is available from the authors.
Figure 11. Ratio of moment (top) or shear (bottom) demand on columns in low seismicity zone from modal response spectrum analysis with \( q = 1.5 \) to design values of moment (top) or shear (bottom) resistance.

7. REFERENCES