A Review of the Nonlinear Static Assessment Procedure in the Turkish Earthquake Code

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SUMMARY
Vulnerability assessment of existing reinforced concrete structures has become one of the major goals to be achieved by the authorities, not only in Turkey but also in most countries where seismic action threatens the building stocks. The current seismic design and assessment code of Turkey, in action since 2007, is the first code in the country that provides rules for assessing and strengthening existing buildings. The code involves several issues that are quite different than the general tendencies in the existing similar codes and guidelines, such as the use of linear methods for assessment, the use of strains for definition of damages in the reinforced concrete members, or introduction of a nonlinear static procedure as one of the methods to be employed in nonlinear assessment chapter.
The goal of this paper is to review the validity of the nonlinear static assessment procedure given in the Turkish Earthquake Code by comparing the assessment results with real structures from north-western Turkey, where the 1999 earthquakes occurred.

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1. NONLINEAR STATIC ASSESSMENT PROCEDURES

Following the 1994 Northridge and 1995 Kobe Earthquakes, performance-based design and assessment approaches have become widely popular (SEAOC, 2000). Performance-based assessment of a given structure is usually keyed to the capacity curve, a 2D plot that shows the displacement of a monitoring node, typically one of the top nodes, versus the base shear. This is a response curve of a multi degree of freedom (MDOF) building frame that has been converted to a representative single degree of freedom (SDOF) idealization so that it can be compared with the design spectrum. The components of the procedure were first put together by Freeman (1998), where the MDOF to SDOF representation concept dates back to the studies Gülkan and Sözen (1974) and Shibata and Sözen (1976).

Most seismic assessment procedures in codes or guidelines consist of two main parts: a) definition of the target displacement, b) definition of the damage when the structure reaches the target displacement.

The target displacement is calculated by plotting the capacity curve over the demand curve in Spectral Displacement vs Spectral Acceleration format. The performance point is either calculated by using equivalent displacement or equivalent energy rules, as in the Eurocode (CEN, 2003) and Turkish Earthquake Code (TEC, 2007), or by using an iterative procedure that requires a balance among the ductility, spectral displacement demand and the equivalent viscous damping for that ductility (ATC 40, 1994 and ATC55, 2005). A recent study by Bal et al. (2010) proves that for a near-collapse case study structure examined in Sakarya, Turkey, that was damaged after the 1999 Earthquakes of Kocaeli and Düzce, the difference in damage assessment due to the differences in target displacements may be of significant importance. Another study by Bal et al. (2010) has examined the nonlinear static assessment procedures statistically and concluded that the target displacement values found by the improved Capacity Spectrum Method (ATC 55, 2005), and by the N2 method of Eurocode (CEN, 2003), can exhibit a very large scatter in data as compared to the results of nonlinear time history analyses, raising concerns about the reliability of the target displacement approach that is estimated by using the pushover analyses.

The second part of the assessment procedures is relation of the target displacement to a damage level. The differences among the assessment codes and guidelines is much more in the case of this part of the assessment. The first difference appears to be in the damage assessment in the member level. The Turkish Earthquake Code (TEC, 2007) suggests the use of strain limits, for example, while the Eurocodes (CEN, 2003) work with chord rotations. While structural damage can be captured by material strains, the sensitivity of these strains to the analysis and modeling options as well as to the cyclic alterations, however, make their reliability very questionable. The chord rotation is a more stable parameter to be calculated, the limit states must be empirical, leading to high scatter in cases when the RC elements are dissimilar to pool from which the empirical relations were derived.

1.1. Analyses and comparisons

There is the inherent randomness of the earthquake events and relevant structural response, and there exists conservatism in the codes, thus a clear comparison of real damage state experienced following an earthquake and the damage estimation by using code-based approaches is not feasible. There is however a comparison that can be made, comparing the damage obtained by using the nonlinear time history analysis and nonlinear static procedures. This simple comparison tends to eliminate several uncertainties since the level of such uncertainties would be the same for both computer models. Some of these uncertainties are: i) inter- and intra-event uncertainties regarding the earthquake event, ii) unexpected soil phenomena, iii) significant soil-foundation-structure interaction, iv) high radiation damping or other forms of damping, v) contribution of the non-structural elements to the overall response, vi) uncertainties in the distribution of the material properties over the structure, and vii) differences in the calculated and real mass.

Two case study structures have been analyzed by using both nonlinear time history and nonlinear static analysis methods. SeismoStruct software (SeismoSoft, 2012), capable of modeling RC structures with distributed plasticity models, is used for analyses. The first case study structure was subjected to Sakarya record of the 1999 Earthquake while the second was in Gölcük, and was subjected to the East-West component of Yarimca record of the same earthquake. The records and their acceleration spectra are given in Figure 1 to Figure 3.

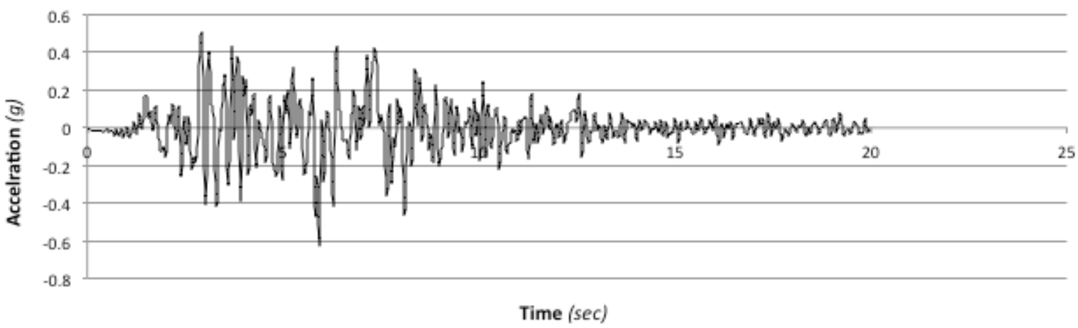

Figure 1. Sakarya record applied on the first case study structure

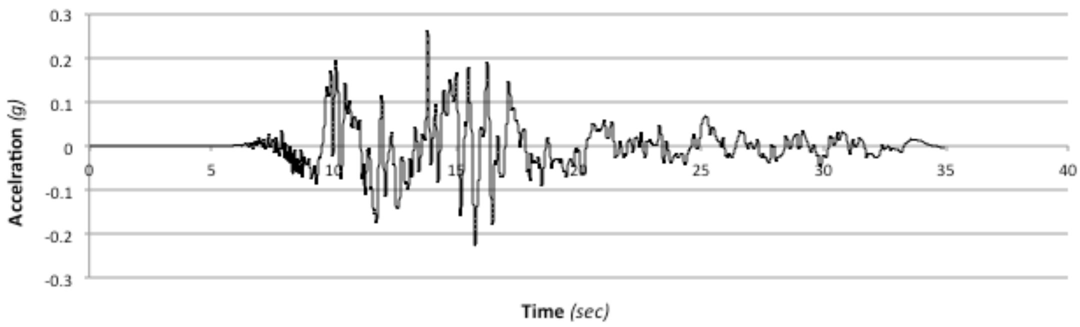

Figure 2. Yarımca record applied on the second case study structure

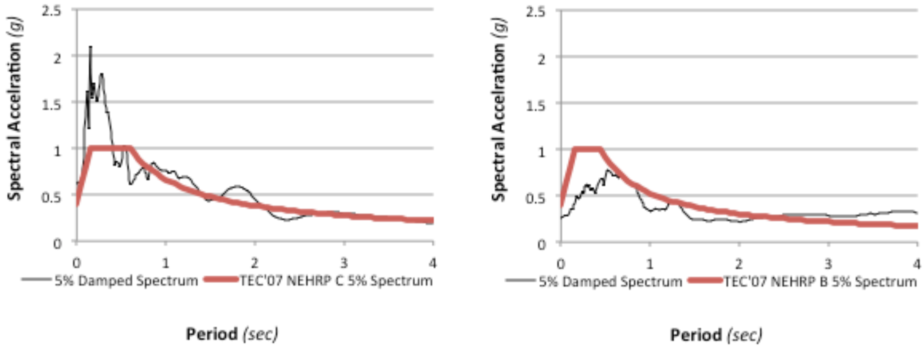

Figure 3. Comparison of the acceleration spectra for the Sakarya record (left) and Yarımca record (right)

2. CASE STUDY BUILDINGS EXAMINED

2.1. Case Study 1

The subject building is located in Gölcük, Kocaeli and was still under construction during the 1999 Earthquakes of Kocaeli and Düzce (Mw=7.4 and 7.2, respectively). It is a five- story building. The building was not damaged during the August 17, 1999 Kocaeli Earthquake. The building is bare frame allowing an easier evaluation because partition walls do not exist (Figure 4 and Figure 7). The cast-in place reinforced concrete building designed according to requirements of the 1975 Turkish Seismic design code, where the lateral load coefficient (i.e. design base shear over the total building weight) varied between 8 to 10%. The building had overall plan dimensions of 14.70 m by 9.80 m and was approximately regular in plan. The regular story height is 2.80 m.

The typical floor framing consists of the reinforced concrete two-way slabs with 0.12 m thickness and 0.20x0.60 m beam dimensions with constant size at all floors. The typical columns are rectangular in cross section 0.30x0.60 min size with 10-Φ16 at the basement and ground floors, 0.25x0.60 m in size with 8-Φ16 at first and second floors and 0.25x0.50 m in size with 8-Φ16 at the third floor. The transverse reinforcement in columns are about 0.10 m at the confinement zones and 0.25 m at the column central zones. The concrete strength was found 16 MPa for the slab, beams and columns. The steel rebar strength was obtained as 420MPa for all structural elements.

The bare frame structure is located on the stiff soil deposit, which can be classified as NEHRP B type soil profile. Foundation system is observed as the continuous footings with 0.60x1.00 m foundation beam size.

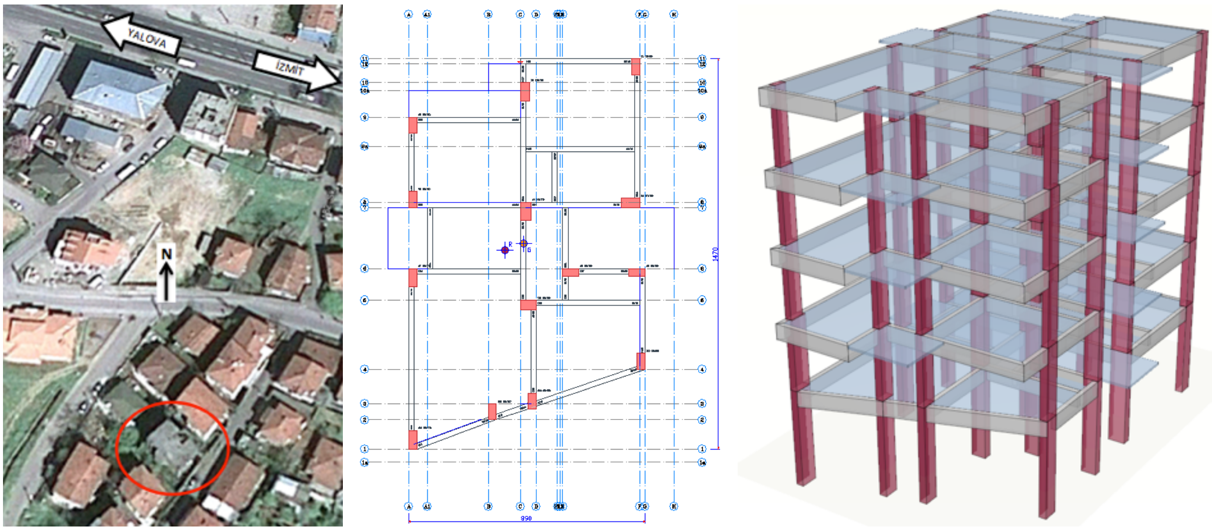

Figure 4. The satellite view of the case study (left), formwork plan of the ground floor (middle) and the 3D view of the structure (right)

2.2. Case Study 2

The subject building is located in Adapazarı, Sakarya and was also under construction during the 1999 earthquakes,. The structure is a five-story cast in place RC building. The building was significantly damaged during the August 17, 1999 Kocaeli Earthquake. It was a typical structure constructed in Adapazarı in which the ground floor is arranged for commercial and upper stories for residential purposes (Figure 6 and Figure 10). The building has overall plan dimensions of 16.40 m by 10.90 m. It consists of a sub-floor that was designed to serve inside the first commercial floor that has the height of 5.75 m. The subfloor has the height of 3.25 m. The regular story height is 2.80 m. Infill walls were used only in one bay in the ground floor and partly in some perimeter frames in the first upper floor, so they can be ignored. The structure was designed according to requirements of the 1975 Turkish Seismic design code. The typical floor framing consists of reinforced concrete two-way slabs with 12 cm thickness. The beam dimensions in all floors are 0.2x0.6 m. The transverse reinforcement spacing in columns is about 0.10 m at the confinement zones and 0.2 m at the column central zones. The concrete strength was found as 20 MPa for the slabs, beams and columns. The reinforcing bar strength was obtained as 420MPa for the structural elements. The structure is located on the relatively soft soil deposit, which can be classified as NEHRP C type soil profile. Foundation system is s the mat foundation with 0.40 m slab and 0.50 m width by 1.0 m depth continuous beams.

Figure 5. Satellite view of the location of the 2nd Case Study building

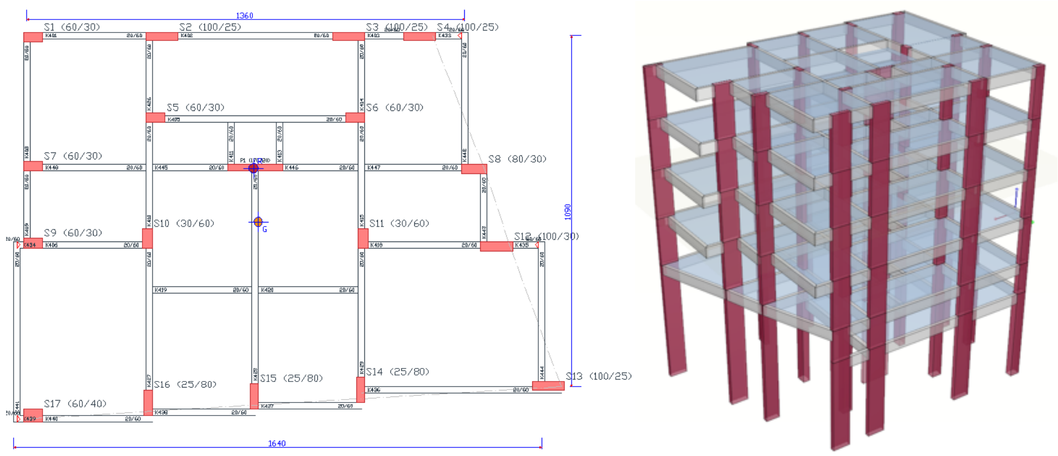

Figure 6. Formwork plan of the 2nd case study building (left) and its 3D view (right)

3. ANALYSIS RESULTS AND DISCUSSIONS

There are two case study structures examined in this study. The first structure is rather simple, but the second one has plan and elevation irregularities. There is a sub-floor created inside the ground floor, leading to very high levels of torsion. The structure experienced very high levels of residual drifts in the columns along the front façade, and extensive damage in the ground floor (see Figure 10).

Figure 7. General view of the 1st Case Study structure

The structure did not suffer any significant damage. The nonlinear time history analysis, based on the E-W component of the Yarimca record (see Figure 2), provides a damage estimation of “Minimum Damage”, a finding that is in line with the real damage state.

The nonlinear assessment procedure followed, as given in Figure 8, suggests a Life Safety damage level since several beams and columns reach certain damage levels. The time history analysis, on the other hand, indicates no damage in any of the elements. This inconsistency in findings may have several reasons, such as the strains being used as damage indicators, or the type of the nonlinear analysis.

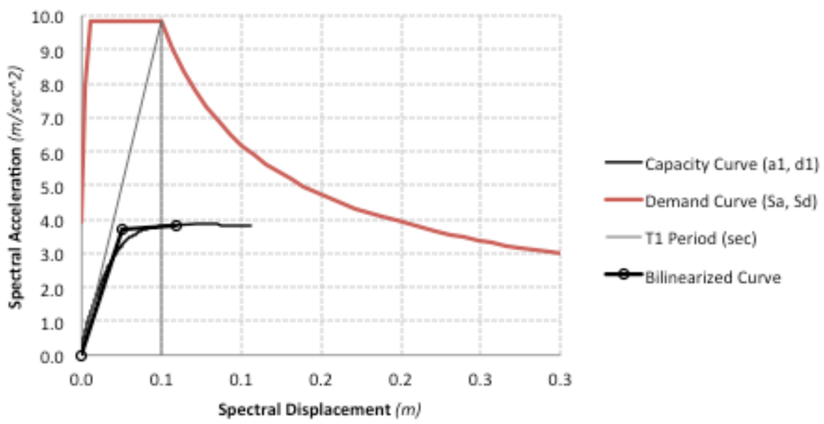

Figure 8. Estimation of the target displacement for the 1st Case Study structure

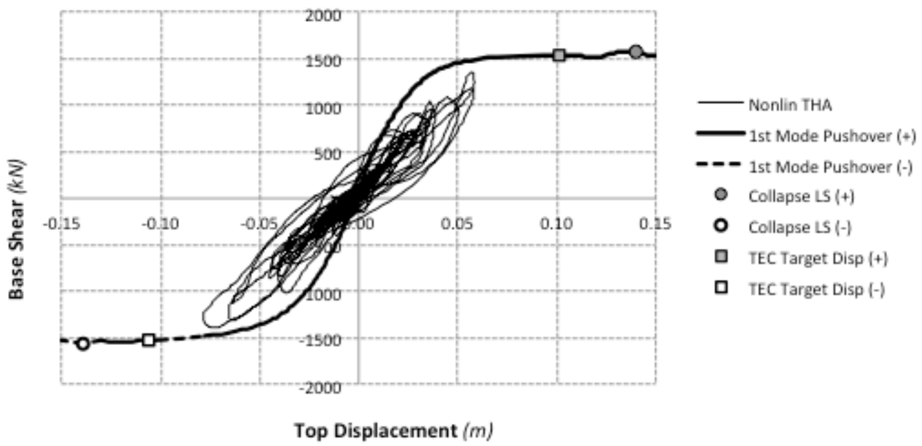

Figure 9. Comparison among the nonlinear time history analysis results, pushover analyses and the estimation of the target and collapse displacements for the First Case Study

The 2nd case study structure is a rather complex building due to the irregularities it possesses, however it is an ideal case for the exercise aimed in this study since it was under construction at the time of earthquake. Additionally, the structure reached the drift levels that would lead to a total collapse, but due to the lower axial loads and nominal p-delta effects on the displaced columns, the structure was somehow "frozen" in a state just before the physical collapse, allowing surveyors a detailed survey of the damage distribution.

Estimation of the target displacement of the 2nd case study, along the shorter axis of the structure as it was subjected in 1999, given in Figure 11, follows the methodology suggested in the Turkish Earthquake Code (TEC, 2007). The procedure simply follows the equivalent displacement rule since the fundamental period of the structure, 0.68sec, is longer than the corner period of the design spectrum, 0.6sec.

The structure was also analyzed by using the only available record from the 1999 Kocaeli Earthquake close-by, the E-W component of the Sakarya record. The structure must have been subjected to a different record due to the differences in position and in soil properties as compared to that of the recording station, but the available record is still an acceptable approximation for the purposes of this study. A recent scientific project, funded by Turkish National Science Foundation (Kutanis et al., 2011), examines the soil properties in the region to come up with plausible seismic action to which this structure was subjected.

Figure 10. Overview of the damages of the 2nd case study structure

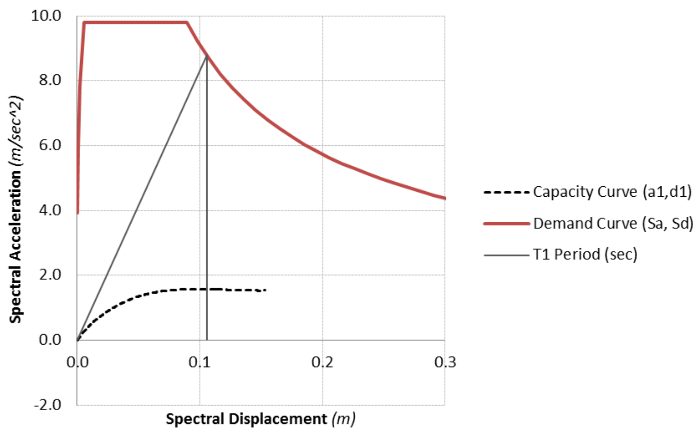

Figure 11. Estimation of the target displacement for the 2nd Case Study structure

The nonlinear time history analysis, following the damage descriptions in the Turkish Earthquake Code (TEC 2007), suggests that the structure would reach collapse limit state under this shaking. This finding is in agreement with the field observation since the structure was in collapse limit state in reality. The damage estimation by the nonlinear static procedure, on the other hand, suggests a life safety limit state, as shown in Figure 12. The maximum displacements reached in both directions are not significantly different for nonlinear dynamic and static analyses, as given in Figure 12, but the nonlinear static procedure simply underestimates the capacity of the structure.

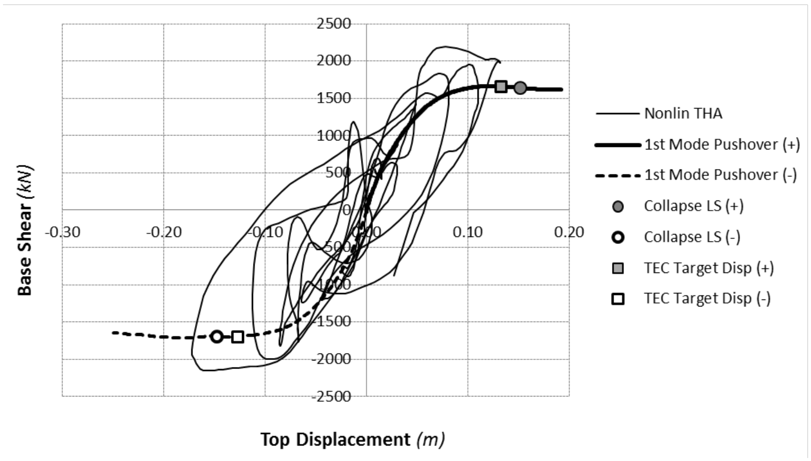

Figure 12. Comparison among the nonlinear time history analysis results, pushover analyses and the estimation of the target and collapse displacements for the Second Case Study

4. CONCLUSIONS

Use of nonlinear static procedures for assessing seismic vulnerability of existing structures is widely included in the existing codes and guidelines worldwide. These methods are based on fundamental earthquake engineering concepts that are valid in several cases but are not precise enough, if not completely wrong, in real cases experienced on the field. It can be even said that the nonlinear static procedures are little more than academic exercises, sometimes off the true mark, for several structures existing in the building stocks.

Two simple but real examples examined in this paper, chosen from the Turkish building stock in western Turkey, are cases that tend to void the assumptions and findings of the nonlinear static procedure suggested in the Turkish Earthquake Code (TEC, 2007). However they are not enough to reach general conclusions. The study presented herein needs to be extended to additional buildings, with different conditions and damage states. The details of the inconsistencies should also be examined in more detail so that some general suggestions can be derived. Provisions must be thoroughly validated before they are made part of codes or other binding documents.

AKCNOWLEDGEMENT
This publication is part of a scientific project, "108M303: Development of Performance Based Design and Evaluation Methods by Comparison with Observed Earthquake Performance of the Structures in Turkey", supported by the Turkish National Science Foundation (TUBITAK). This generous support is appreciated.

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