EARTHQUAKE PERFORMANCE OF A THREE STORY ACTUAL SUB-STANDARD BUILDING

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Abstract

A three-story full-scale sub-standard reinforced concrete building was tested under self-weight and reversed cyclic lateral displacements to examine its behavior during earthquakes. The tested building is part of an existing building more than 20 years old, which was demolished partially to obtain the test structure. The building reflects the main characteristics and deficiencies of existing sub-standard buildings in Turkey. A measurement system, which collected data from approximately 64 channels, consisting of data loggers, load cells, displacement transducers and strain gauges was used. Since the building was damaged significantly at 1.5 % inter-story drift ratio (longitudinal bars of beams and columns were buckled, extensive and large cracks were observed), the test was terminated. In this paper, details of this sub-standard building and test results are presented. Additionally, available nonlinear modeling techniques and seismic performance estimations of the Turkish Seismic Design Code (2007) and ASCE 41-13 (2014) are compared with the experimental results.

Introduction

The buildings constructed with poor reinforcement details and low strength concrete generally suffer from earthquakes due to their low deformation and lateral load capacities. For reliable seismic evaluation of such sub-standard buildings, engineers require reliable assessment methodologies. The available assessment methodologies define the modeling and analysis approach to be followed and the criteria to be used for performance level identification. The current damage limits used for performance level identification mainly depend on post-earthquake observations and results of laboratory tests performed on members (such as beams and columns) and frames that consists of beams and columns. However, among various other uncertainties, post-earthquake investigations cannot provide in-depth information about the experienced loading and response of the building during this loading. In the case of laboratory tests, the physical and financial constraints limit performing full-scale tests that can more realistically represent the seismic behavior of actual structures. Moreover, the laboratory tests are generally realized under controlled and idealized conditions, so that, the uncertainties available for boundary conditions and variation of material and workmanship qualities in the structure cannot be effectively considered. Consequently, full-scale field tests carried on actual buildings becomes a viable and complimentary option to post-earthquake investigations and laboratory tests which provide data for definition of seismic performance levels and damage limits.

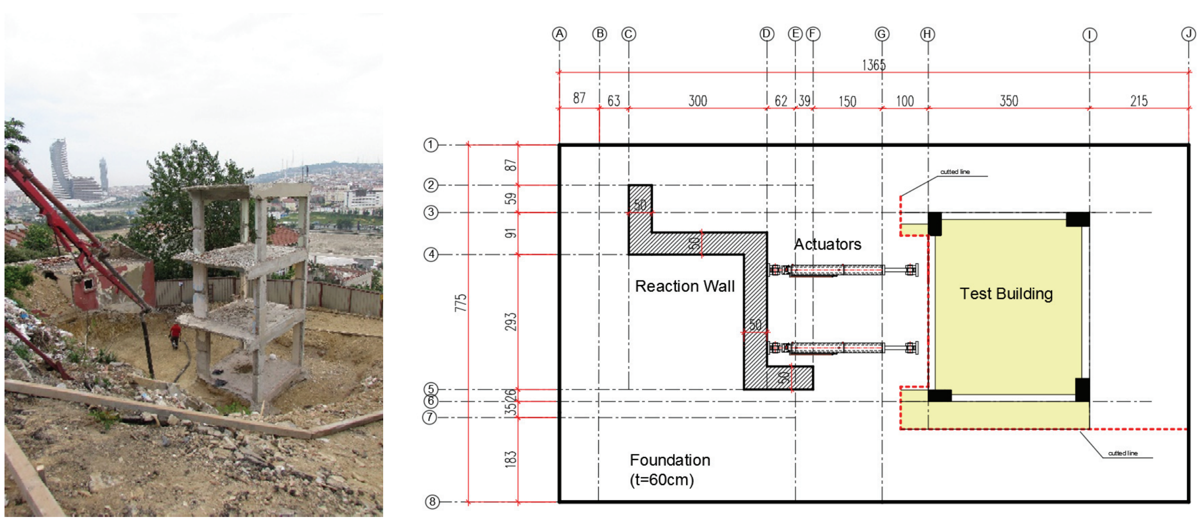
This paper presents the test results of an actual reinforced concrete (RC) building tested on site and subjected to reversed cyclic lateral loading. The building reflects the main characteristics and deficiencies of existing sub-standard buildings in Turkey. The test building is part of an existing actual building in Istanbul constructed at earlier 1990s. In addition to test observations on overall behavior and development of damage at increasing drift ratios, predictions of the seismic performance assessment approaches recommended by ASCE 41-13 (2014) and Turkish Seismic Design Code (TSDC, 2007) are also compared with the test results and observations. Further details for testing procedure and results can be found elsewhere (Comert et al. 2016).

Properties of Test Building

The test was carried out at the Fikirtepe district of Istanbul city. There are hundreds of sub-standard buildings in this area representing common deficiencies of structures in Turkey. Based on the Urban Transformation Law (Law No. 6306 on the Transformation of Areas Under Disaster Risk) that aims to reconstruct or retrofit seismically vulnerable buildings on a mass scale, the area has recently been designated as a high seismic risk area and was abandoned for demolition at the time of testing. One of these buildings was selected to execute the test program. The general appearance and plan of the building are given in Fig. 1. A reaction wall was designed and constructed to the test site to apply lateral load representing seismic actions (Fig. 2b). In order not to exceed the load capacities of the reaction wall and the hydraulic actuators, the existing building was demolished partially so that the remaining part was tested (as shown in Fig. 2). After partial demolishing of the existing building, since the original foundations of existing building was seen to be deficient and irregular, the test site was leveled and lean concrete was poured, which was followed by construction of a 0.6 m thick mat foundation (Fig. 3).

(a) (b)

Figure 1. (a) General view (b) plan of test building (dimensions in cm).



(a) (b)

Figure 2. (a) Partial demolishing of the building and (b) plan of test area (dimensions in cm).

Figure 3. Stages of strengthening of foundation of test building.

The test building had 2.7 m story height at all three stories. The plan dimensions were approximately 3.5 m by 4.1 m. The beams in the loading-direction of the building extended for an additional length of 0.6 m from the column surfaces in order to supply adequate anchorage length to the beam longitudinal bars, since the adjacent bay of the building was demolished. Non-structural elements such as hollow clay brick infill walls and plaster covering beam and column surfaces were removed for preventing potential interference with the structural system. The material characteristics of building were determined through tests conducted on specimens that were extracted from the building. Concrete cores (100 mm × 200 mm cylindrical cores) were sampled from each story and tested under uniaxial compression, which indicated an average concrete compressive strength of 17 MPa. Reinforcement of the structural members consisted of plain bars, which was typical for the time of construction. Uniaxial tension tests carried out on steel samples obtained from the building provided average yield strength values of 290 and 370 MPa for the longitudinal and transverse bars, respectively. The reinforcement details of the test building were also obtained by using destructive and non-destructive investigations. The beams had 150 mm × 500 mm cross-section dimensions and they had two 10 mm and one 12 mm diameter longitudinal bars at the top and two 12 mm diameter bars at the bottom of the support cross-sections. Three of the four columns at each of the first and second stories of the test building had cross-sectional dimensions of 250 mm × 500 mm, whereas the fourth column (S14) had 300 mm × 650 mm cross-section. All columns had six 16 mm bars as longitudinal reinforcement. In the third story of the building, all columns had approximately 250 mm × 400 mm cross-sectional dimensions and incorporated six 16 mm longitudinal bars, except one column that had four 16 mm bars (S31 column). With these member cross-section dimensions and longitudinal reinforcements, the building had a strong column-weak beam type moment capacity hierarchy at the beam-column joints. The plan and reinforcement details of the tested part of the building are given in Fig. 4. The column longitudinal bars in the first story of the building were continuous down to the foundation, instead of being spliced above the foundation level, probably because the building did not have a proper foundation system. The transverse reinforcement of beams and columns comprised 8 mm diameter stirrups tied at an approximate spacing of 300 mm, with absence of confinement regions at potential plastic hinge locations. As commonly observed in sub-standard buildings, the hooks of the closed stirrups were bent 90 degrees.

Test Setup and Measurement System

Lateral loads were applied by using three servo-controlled hydraulic actuators with 300 kN load and 800 mm displacement capacities, that were attached to the buildings at the first and second story slab levels. The attachment of each actuator to the story was maintained by post-tensioning a steel plate and the plate of the actuator swivel head, by using threaded steel rods that run above and below the reinforced concrete slab and pass through the holes drilled on the beams at both extremities of the building. Rather than using one actuator at each story level, two actuators were attached to the second story of the test building to avoid possible torsional effects. The test was conducted under displacement and force control. During the test, the first story level actuator was force controlled so that the applied force was kept as half of the total force applied by the second story actuators, which were displacement controlled. A 64 channel measurement system was used to measure the story forces, displacements and strains at potential plastic hinge regions of columns and beams. The lateral load applied to the structure was monitored by using actuator internal load cells. Deformations at the potential plastic hinging regions of the beams and columns were measured by using displacement transducers with a measurement length of h/2, where h is the cross-section depth of the beam or column. General layout of the measurement system can be seen in Fig. 5. The displacement cycles were applied considering specific drift ratio levels of the first story of the buildings. Target drift ratios were defined as following: 0.25% (one cycle), 0.5% (two cycles), 0.75% (one cycle), 1.0% (two cycles), 1.5% (two cycles).

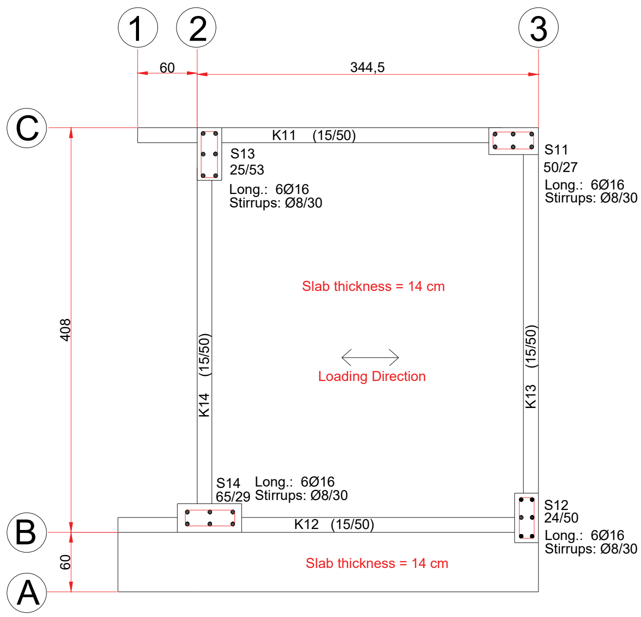


Figure 4. Plan and reinforcement details of the test building (dimensions in cm).

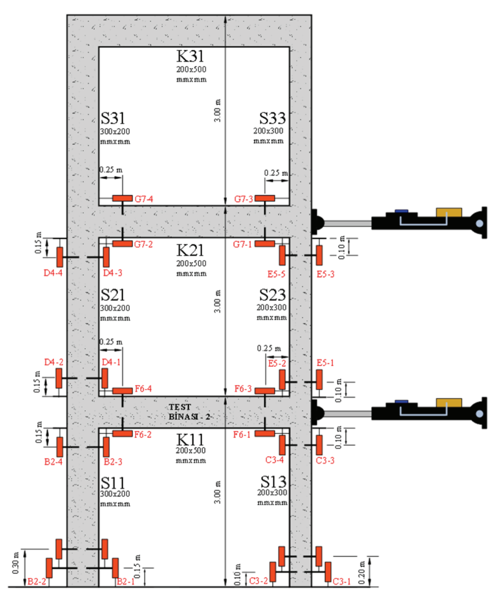


Figure 5. Measurement system.

Test Results

The test was performed on 26th–27th of July, 2014. The reversed cyclic loading pattern was applied up to a drift ratio of 1.50%. Since the building damage was severe at this drift ratio, the test was finalized for the sake of safety. The base shear vs. 1st story drift ratio response of the building is presented in Fig. 7. Marks on this figure indicate important stages of the test. Throughout the test, no significant shear damage was observed and the overall response was dominated by flexural behavior at the critical end regions of beams and columns. The damage was first seen (at 0.25% drift ratio) at the support regions of the first story beams and columns (K11 and K12 beams and S11 and S14 columns in Fig. 4), which were parallel to the loading direction. Then, the damage started to accumulate at the bottom ends of the first story S11 and S14 columns that were relatively stronger in the loading direction than the S12 and S13 columns. No significant damage was observed at weak columns (S12 and S13) of the first story and all of second and third story. In the following cycles, damages on the beams developed faster than the columns. Since concrete cover was thin (i.e. 5 mm), concrete spalling occurred at an early stage at beam end regions. The longitudinal bars of K12 beam started to buckle at 0.75% drift ratio cycles (Fig. 7a). Vertical cracks and cover crushing were seen at S11 and S14 columns at 1.0% drift ratio. The longitudinal bars of the S14 column buckled at 1.5% drift ratio (Fig.7b). Further details for test results can be found elsewhere (Comert et al. 2016).

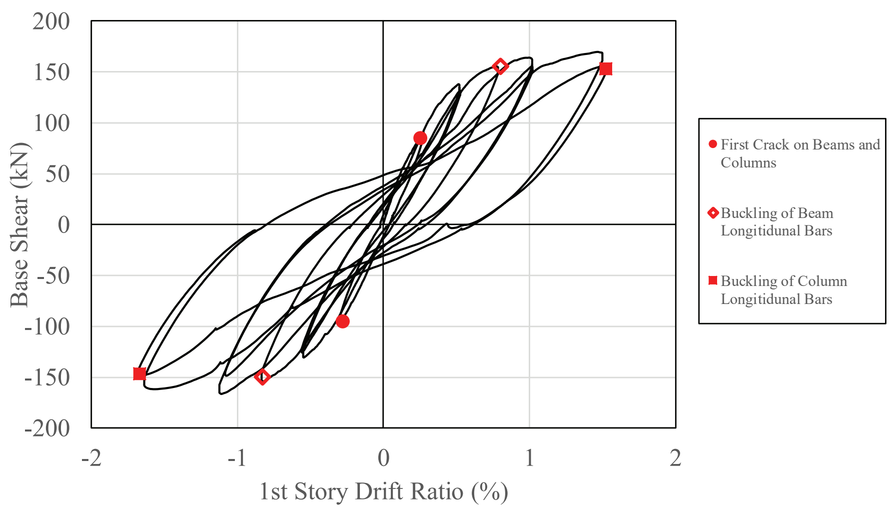
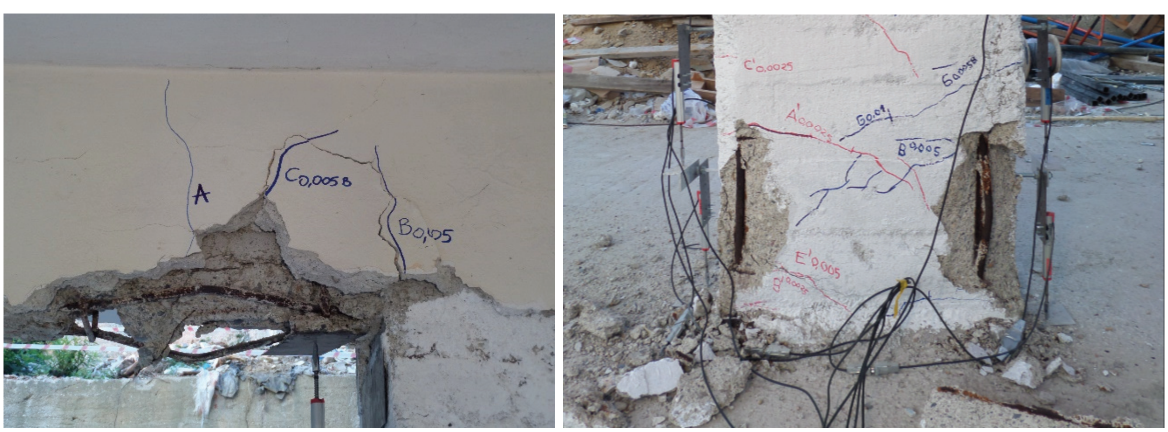


Figure 6. Base shear – 1st story drift response of test building.



(a) (b)

Figure 7. The damage seen in (a) K12 beam at 0.75 % drift ratio and (b) S14 column at 1.50% drift ratio.

Comparison of Test Results with Estimations of ASCE 41-13 (2014) and Turkish Seismic Design Code (2007)

The building is modeled by using a commercially available software (Perform 3D, 2011) to evaluate the performances of modeling approaches and member damage limits proposed by Turkish Seismic Design Code (2007) and ASCE 41-13 (2014). During all analyses, material and geometric nonlinearities are taken into account. The material nonlinearity is taken into account with lumped plastic hinges that were assigned to the potential plastic hinging zones of columns and beams. For evaluating the response prediction capability of the TSDC (2007), the nonlinear flexural behavior of columns and beams were obtained from section analyses (in means of moment – curvature relationships) and the plastic hinge length was taken as 0.5h (as also suggested by TSDC, 2007), where h is the depth of the member cross-section in bending direction. While assessing the accuracy of the Nonlinear Static Procedure defined in ASCE 41-13 (2014), the plastic hinge properties of columns and beams were both defined by making use of the backbone curves specified in this document. During the analyses, the distribution of lateral loads was assumed similar to the tests (P at the first story, 2P at the second story and zero lateral load at the third story). In addition to modeling and analysis criteria, both performance assessment documents define limits for predicting member damage (or performance) levels. The member damage (performance) limits defined in the documents correspond to states of minimum damage (Immediate Occupancy IO), significant damage (Life Safety LS) and extensive damage (Collapse Prevention CP). In Fig. 8, the predictions of both seismic performance assessment procedures for base shear vs. first-story lateral drift ratio responses are compared with the experimentally-measured cyclic response. Both modeling approaches overestimate lateral load capacity of the building approximately by 5% to 15%.

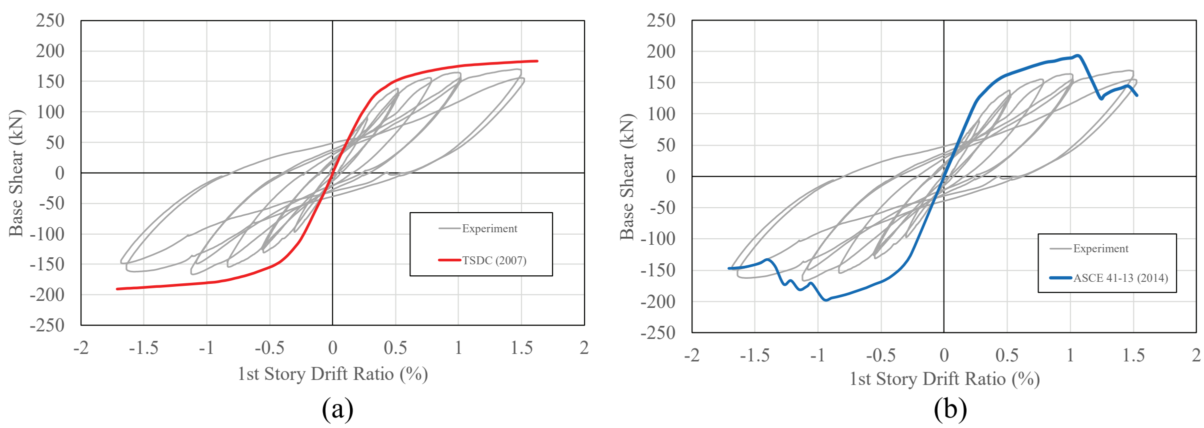


Figure 8. Comparison of test results and analysis results (a) TSDC (2007) and (b) ASCE 41-13 (2014).

In order to compare theoretical and experimental responses at member level, observed damages and the damage limitations for S14 column and K12 beam determined by both approaches are presented in Tables 1 and 2. It should be noted that while TSDC (2007) defines member damage limits in terms of strains of concrete at ultimate compression fiber and reinforcing bars, ASCE 41-13 (2014) defines the limits in terms of plastic rotation of member. To compare results of both documents with the experimental ones, the code limitations were transformed to the total rotation of section by using cross section analyses. As seen in Table 1, for column S14, although the ASCE 41-13 (2014) procedure defines approximately 20% larger rotation limits compared to the TSDC (2007) method, both analysis procedures and damage limitations are in overall good correlation with the test data and observations. The rotation limits that define the member damage states in TSDC (2007) seem to be slightly more conservative than those specified in ASCE 41-13 (2014), when compared with each other and with the experimental observations presented in Table 1. For K12 beam, both procedures fail to estimate the observed level of damage (Table 2). The damage observed at this beam during the test, progressed much more rapidly than both TSDC (2007) and ASCE 41-13

(2014) predictions. For beams of the test building, the diameter of the beam longitudinal bars (d) is 12 mm and the spacing of stirrups (s) is approximately 300 mm. Together with the thin concrete cover, the s/d ratio that is in the order of 25 makes the beam longitudinal bars prone to buckling (as also observed during test). In predicting the beam damage state, both assessment approaches lack the consideration of buckling of the beam compression bar buckling, which leads to misprediction of the behavior, ductility, and damage state of the beam.

Table 1. Comparison of Member Damage Predictions with Test Results for Column S14

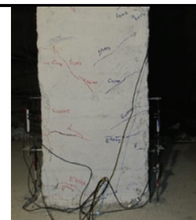
	TSDC (2007) (Rotation)	*ASCE 41-13 (2014) (Rotation)*	*Observed Damage*	*Damage Notes*
IO	0.005	0.0061		(Measured Rotation is 0.0075) Distributed flexural cracks were observed. Maximum residual crack width is measured as 0.5 mm.
LS	0.0071	0.01		(Measured Rotation is 0.0085) Concrete spalling. Maximum residual crack width is measured as 1.1 mm.
CP	0.009	0.011		(Measured Rotation is 0.012) Buckling of longitudinal bars.

Table 2. Comparison of Member Damage Predictions with Test Results for Beam K12

	TSDC (2007) (Rotation)	*ASCE 41-13 (2014) (Rotation)*	*Observed Damage*	*Damage Explanation*
IO	0.006	0.005		(Measured Rotation is 0.012) Buckling of longitudinal bars under compression.
LS	0.024	0.019	Experimental rotation did not reach this level.	-
CP	0.028	0.029	Experimental rotation did not reach this level.	-

Conclusions

In this study, the behavior of a sub-standard building under gravity loads and cyclic lateral displacement reversals is investigated through full-scale field tests and performance evaluation procedures.

The following results are obtained from the test observations and comparisons of test results with predictions of code-based seismic performance assessment procedures conducted to the building:

- The evolution of damage was highly affected by the moment capacity hierarchy between the beams and columns at the beam-column joints. The damage started at the beams of the test building and then accumulated at the bottom edge of first story columns.
- At 0.75% drift ratio, the beam longitudinal bars buckled and at 1.50% drift ratio, the column longitudinal bars buckled. Thereafter, the test was terminated due to safety reasons.
- Both modelling approaches showed good correlation with test results in a global sense. In terms of member damage limits, TSDC (2007) gives 20% conservative limitations when they are compared with the ASCE 41-13 (2014) limitations.
- Full-scale testing of existing buildings may provide a good basis for evaluating the capabilities, limitations, and reliability of available modeling approaches and performance assessment procedures, particularly for the buildings with sub-standard characteristics.

Acknowledgements

Authors are thankful to Istanbul Development Agency (Project No: TR10/12/AFT/0050) and ITU Scientific Research Fund Department for the financial support provided. The authors would like to thank to Japanese project consultants, Prof. Koichi Kusunoki, Prof. Toshimi Kabeyasawa, Prof. Kazuki Koketsu and Prof. Kiyoshi Takano. The contributions of Erkan Tore, Ozgun Ozeren, Alvand Moshfeghi, Soheil Khoshkholghi, Caglar Goksu, Pınar Inci, Ugur Demir, Ahmet Sahin and 2014 summer trainees are also gratefully acknowledged. The authors acknowledge Akcansa Co., Art-Yol Co., Boler Celik Co., Hilti-Turkey Co., Tasyapi Co., Urtim Co., and Kadikoy Municipality.

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