EARTHQUAKE PERFORMANCE OF A THREE STORY ACTUAL SUB-STANDARD BUILDING

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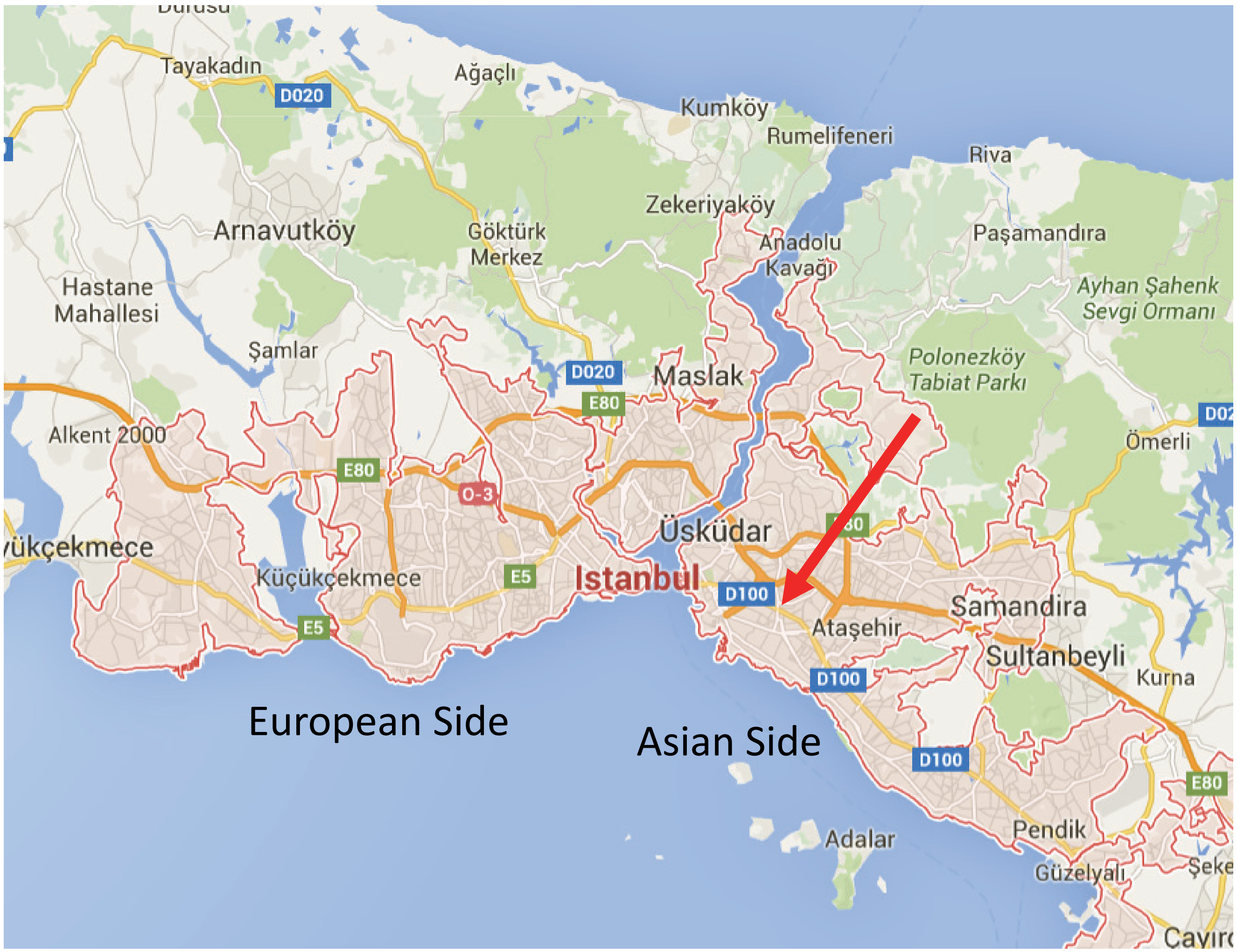
Introduction

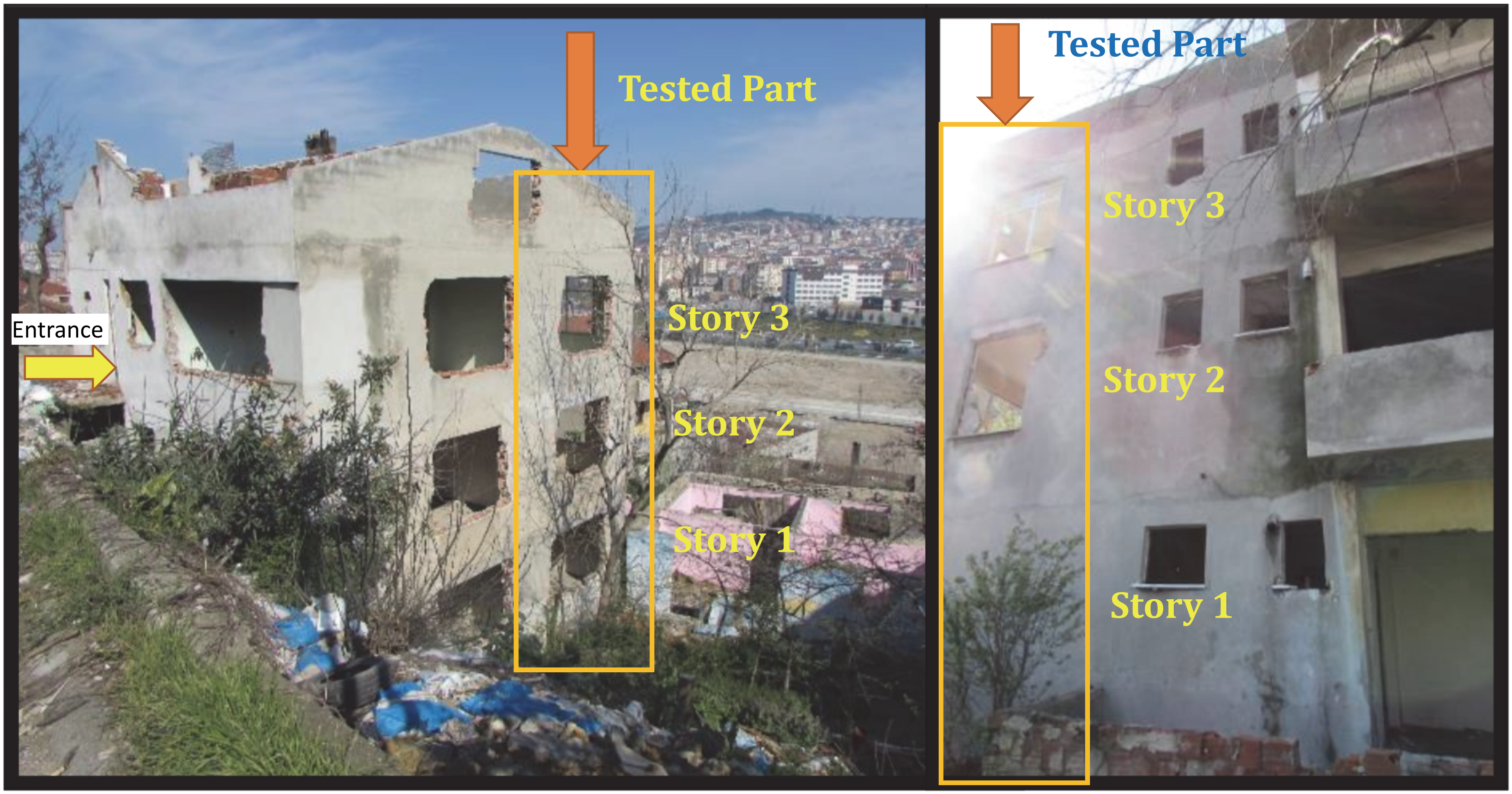
A three-story full-scale sub-standard reinforced concrete building part was tested under its self-weight and reversed cyclic lateral displacements.

The tested building is part of an existing building built more than 20 years ago and it was selected to reflect the main characteristics and deficiencies of existing sub-standard buildings in Turkey (low strength concrete, large stirrups spacing, improper hook details).

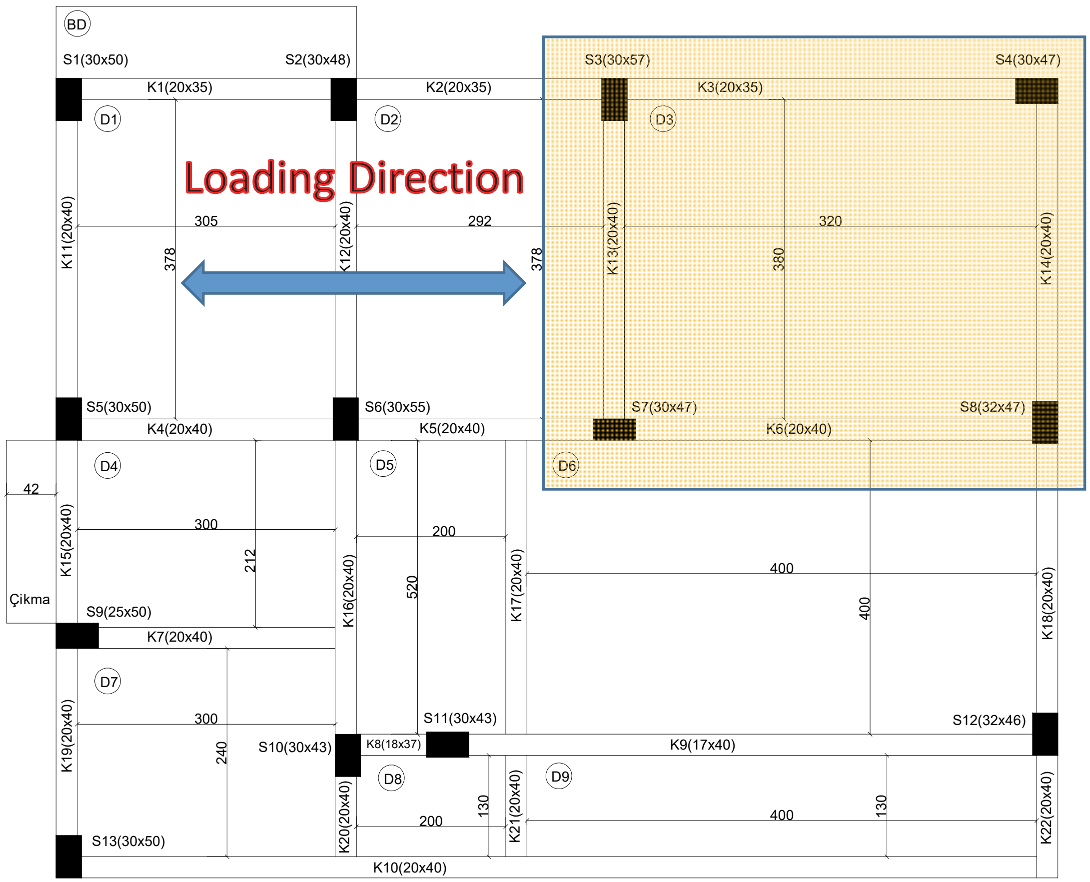
Location of Test Building



Test Building



Demolition for Test Building



Tested Part

Site Preparations

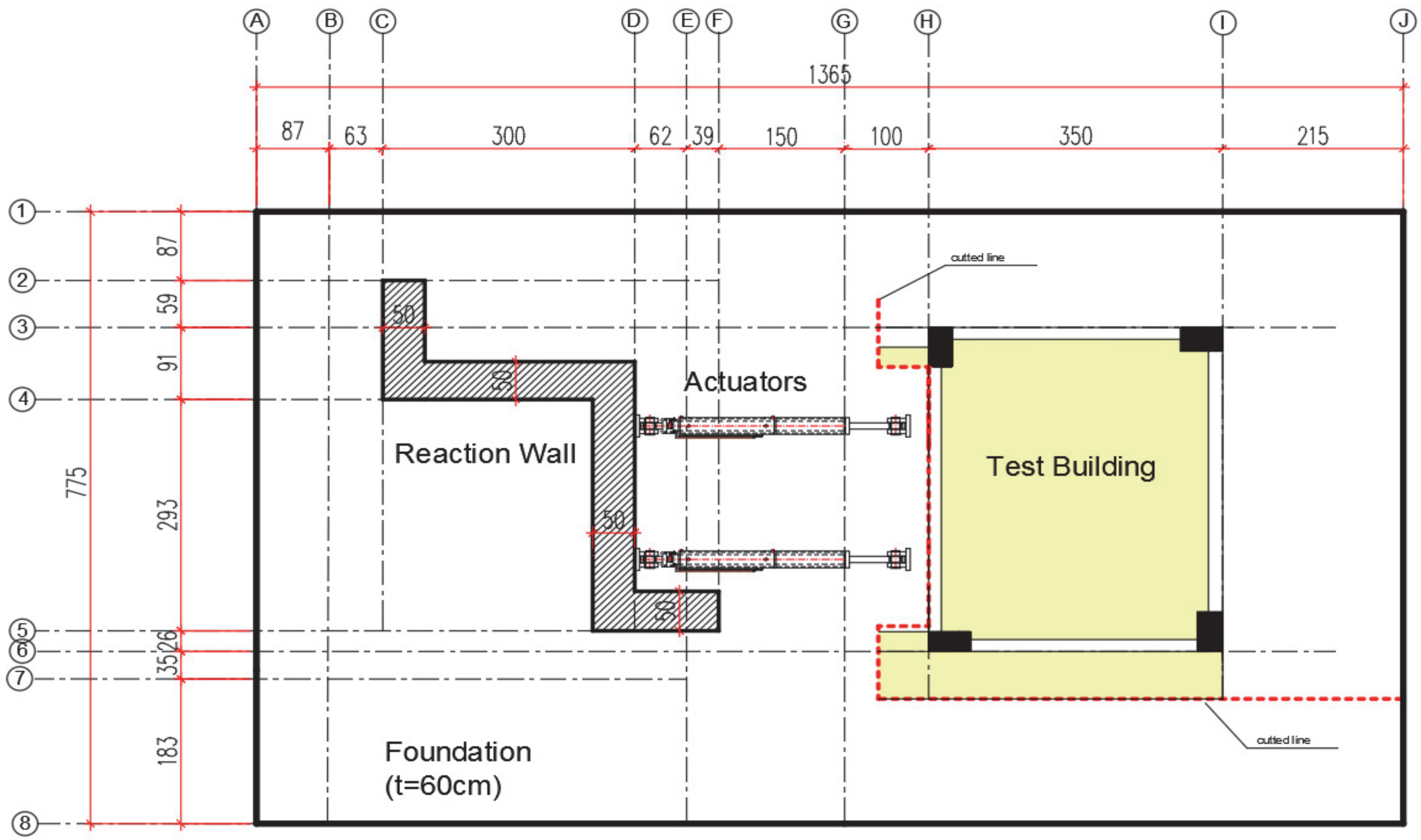
- Demolition

Site Preparations

- Demolition

Site Preparations

- Test site layout



Test Building

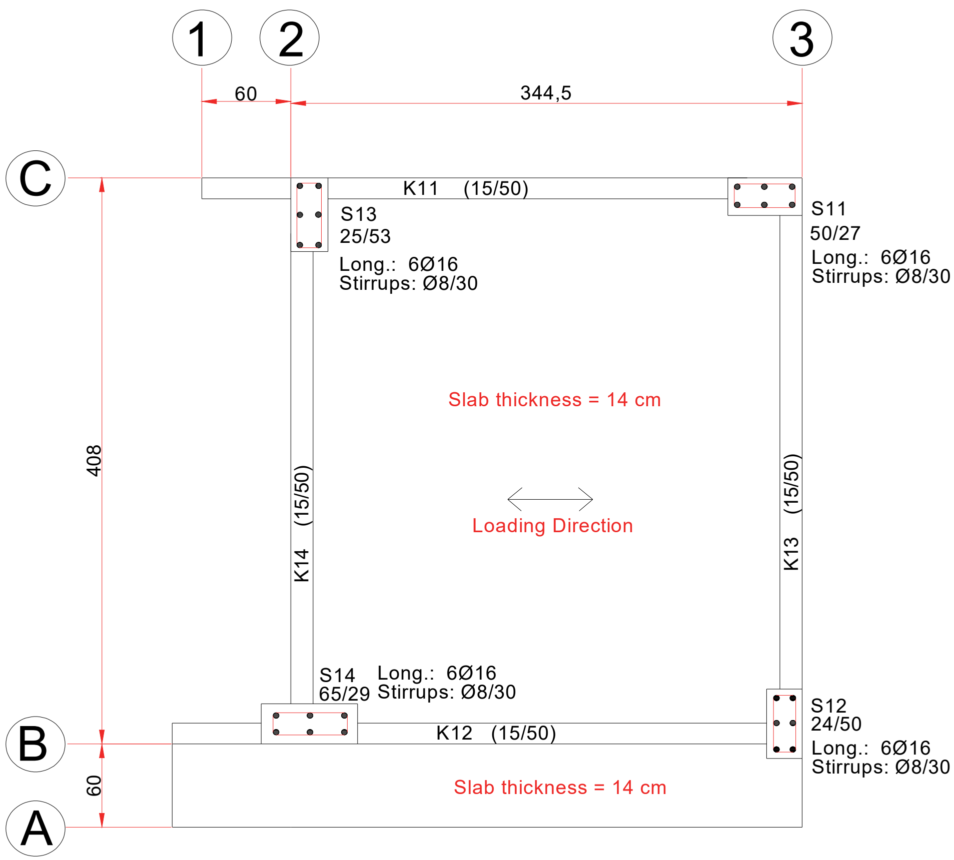
fc=17 MPa

Reinforcing Bars

Column Long. f_y=280 MPa
(lap splices without hook, 32ϕ-60ϕ)
Beam Long. f_y=444 MPa
(lap splices with hook)
Stirrups f_y=365 MPa
(closed tie 90 degree hook)

All members are flexural critical.

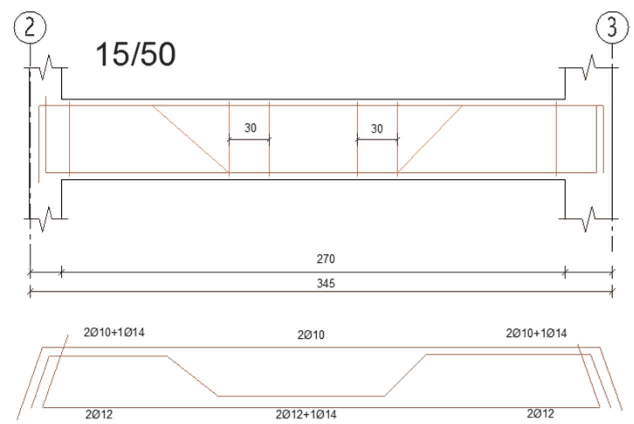
Test Building



Structural Plan

Beam Reinforcement Details

K11, K12,K21,K22,K31,K32

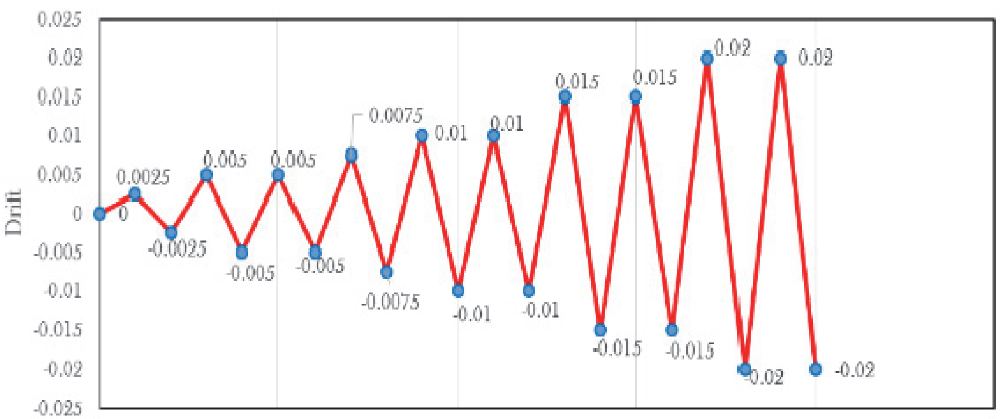


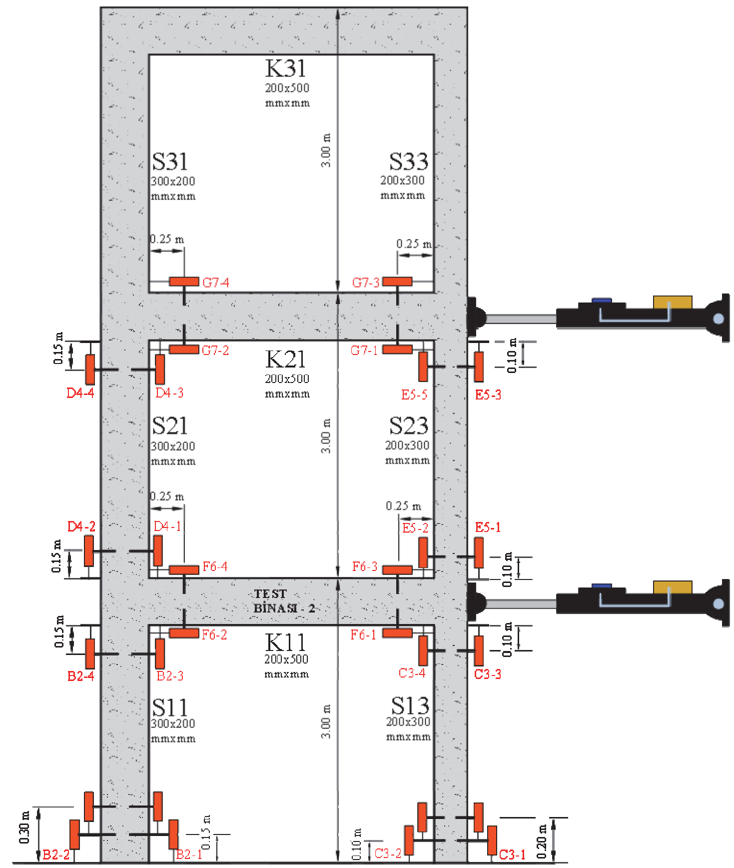
H_{story}=2.7 m

Test Setup: Loading

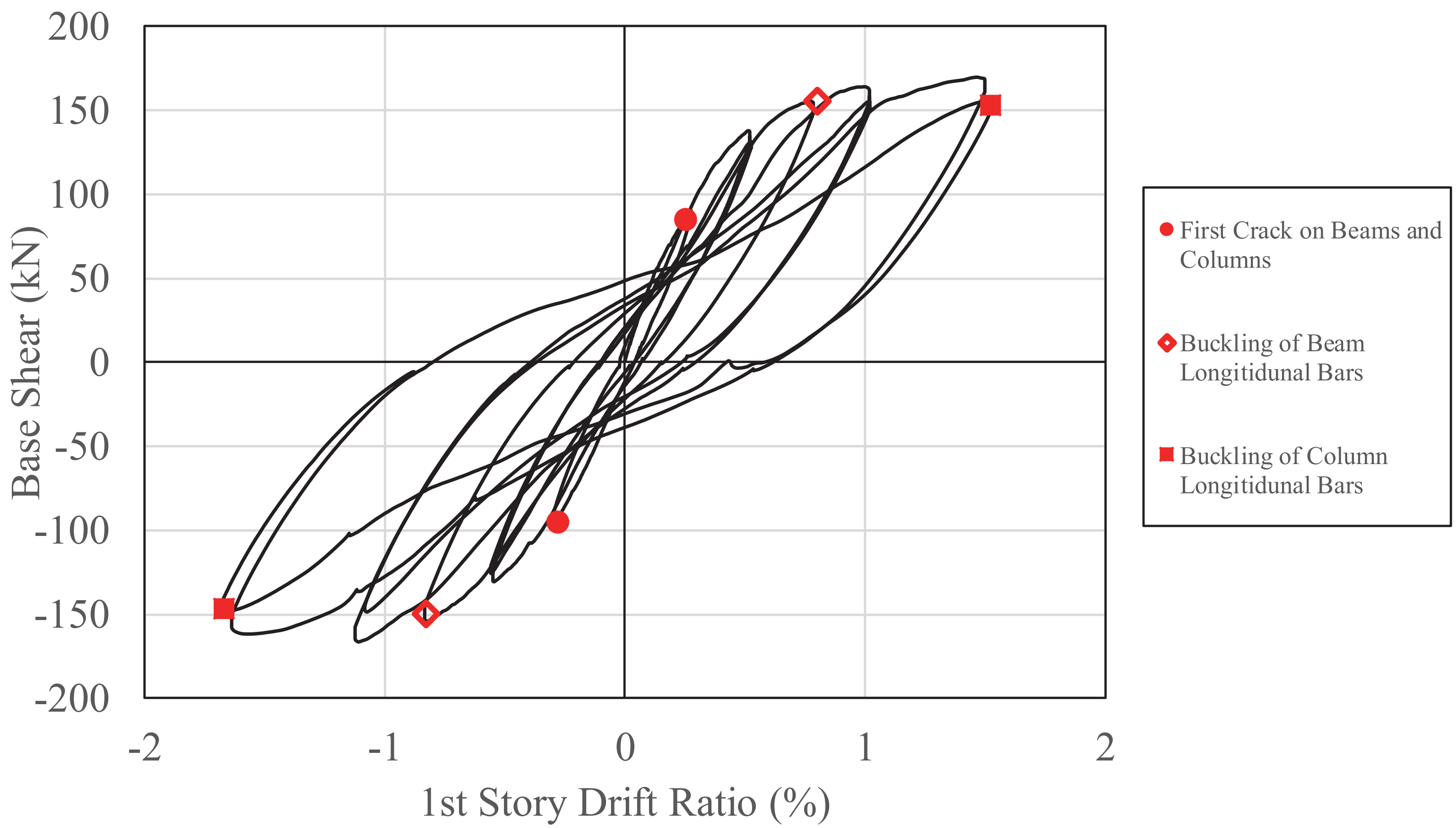
- Reversed cyclic loading with three hydraulic actuators (300 kN load and 800 mm displacement capacities)
- Diplacement controlled loading
- Load distribution in elevation(2P-P) kept constant

Test Setup:
Drift Pattern and Measurement System



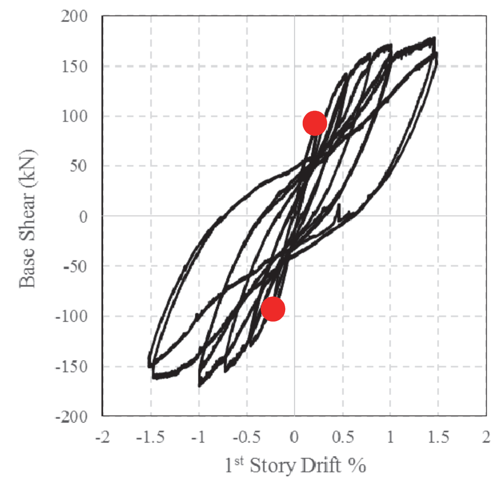


Observations and Test Results



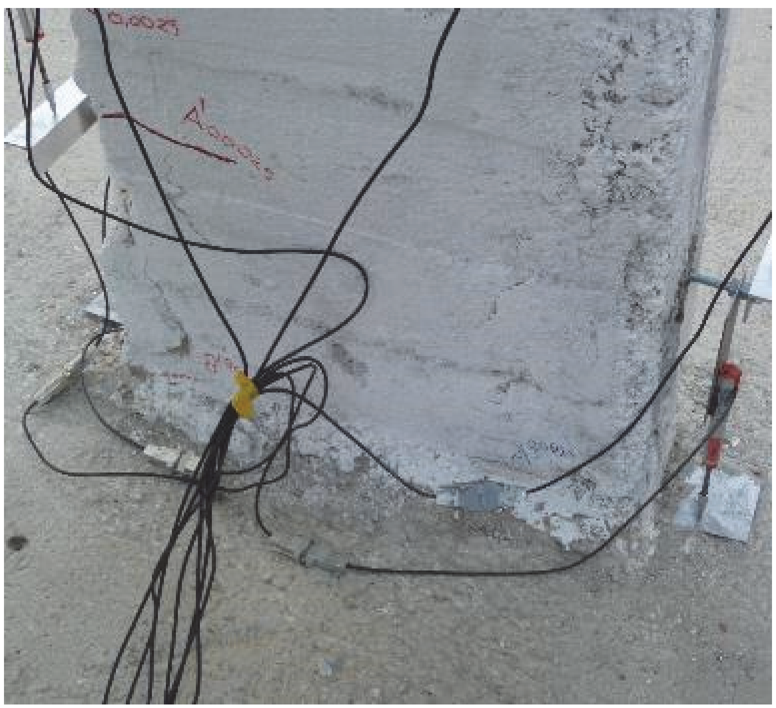
Observations and Test Results

% 0.25 Drift Ratio – 1cycle

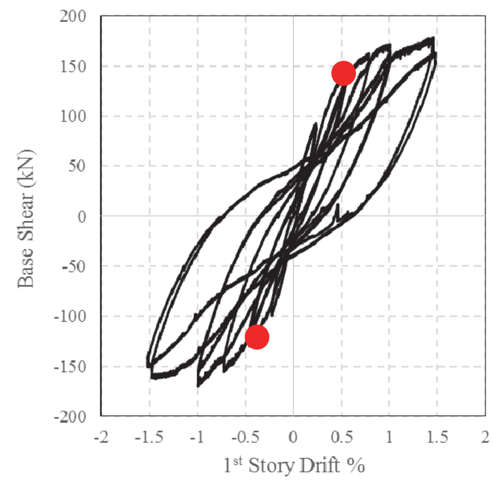


Beams
W_{max}=0.2 mm

Columns
W_{max}<0.1 mm



Observations and Test Results



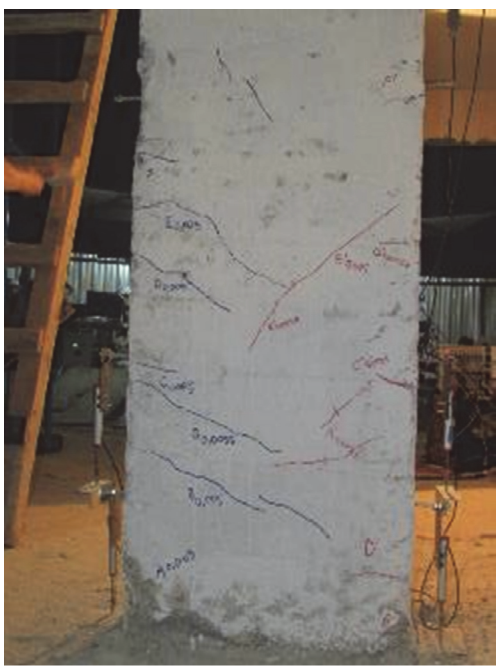
% 0.50 Drift Ratio - 2 cycles

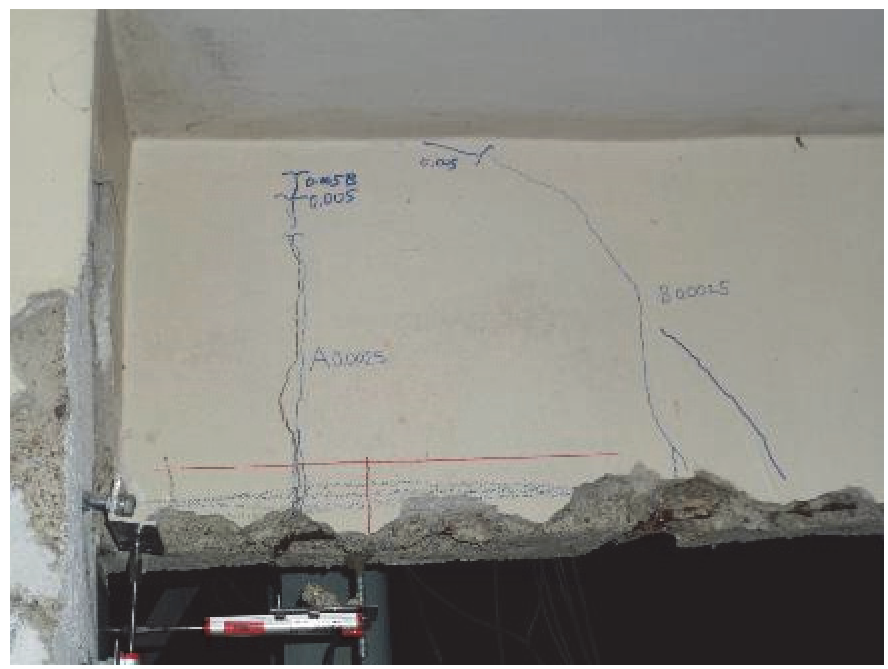
Beams

W_{max}=1.2 mm

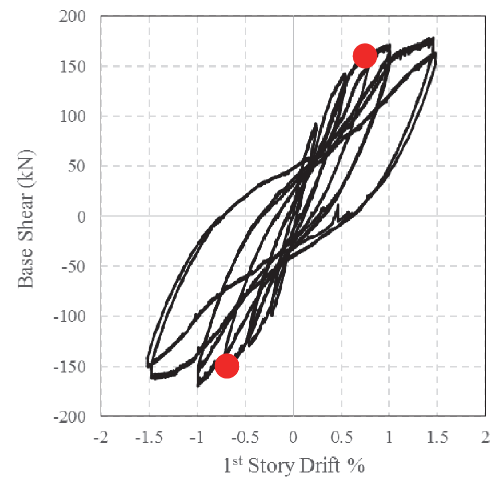
Columns

W_{max}=0.6 mm





Observations and Test Results



% 0.75 Drift Ratio - 1 cycle

Beams

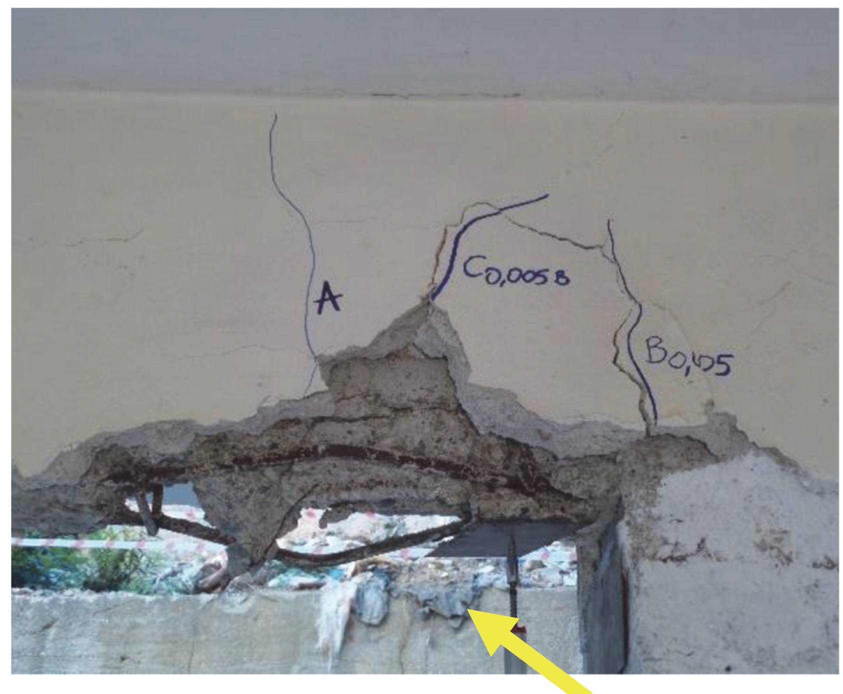
W_{max}=2.2 mm

Crushing at positive peak

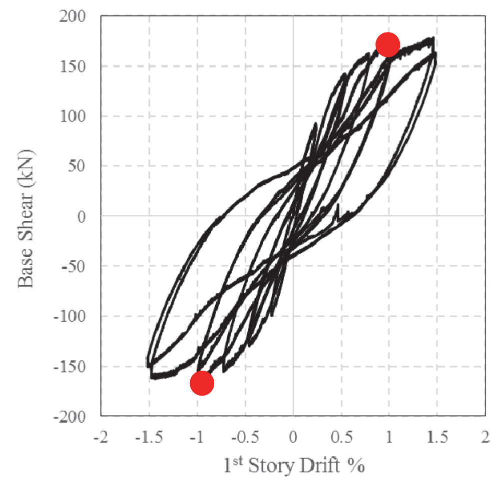
Bucking of bars at negative peak (K11)

Columns

W_{max}=1.6 mm



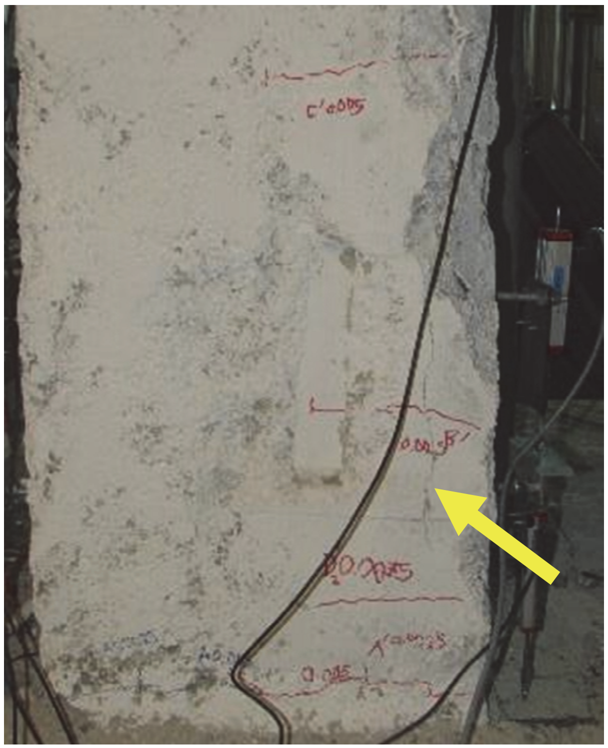
Observations and Test Results



% 1.00 Drift Ratio - 2 cycles

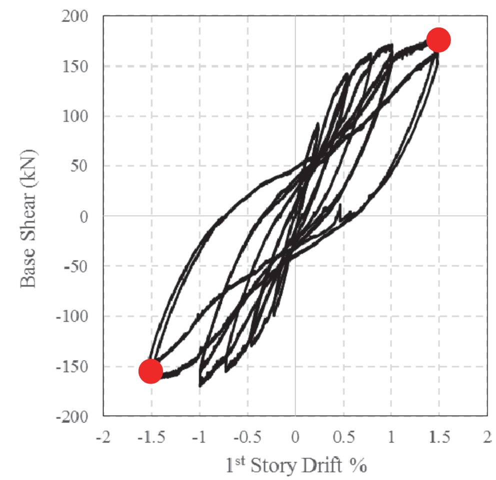
Beams
W_{max}=6 mm
Bucking of bars (K12)

Columns
W_{max}=1.8 mm
Vertical Cracks



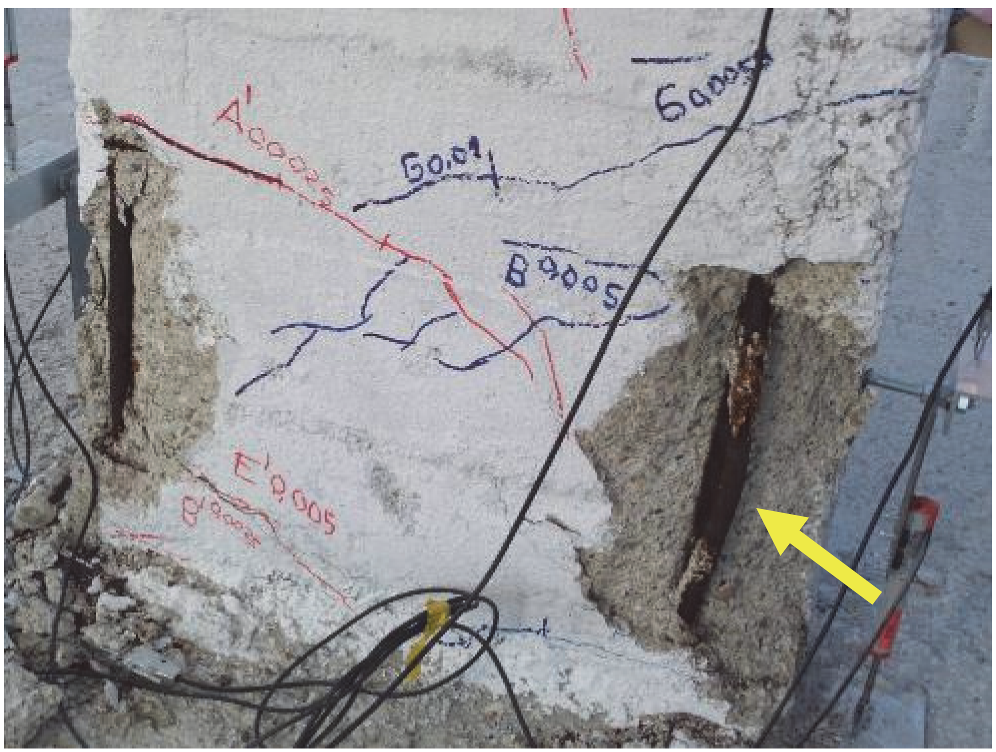
Observations and Test Results

% 1.50 Drift Ratio - 2 cycles

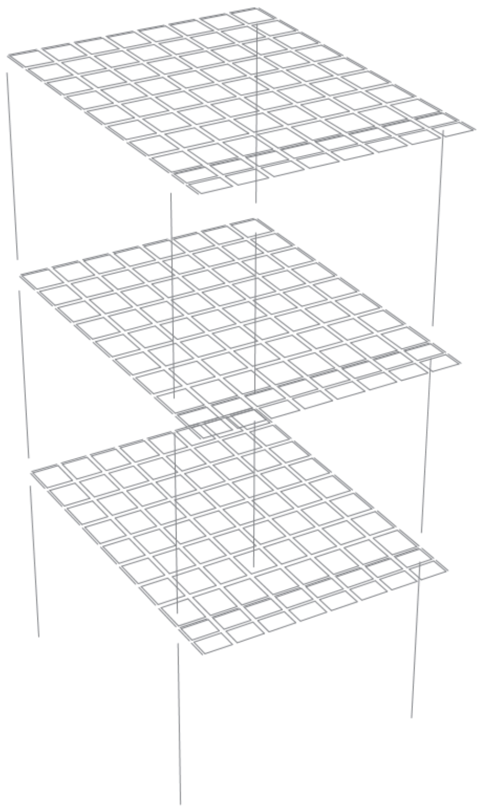


Beams
W_{max}=9 mm

Columns
W_{max}=3 mm
Bar Buckling (S14)



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



Material nonlinearity were defined with hinges that assigned to the end of members.

For TSDC (2007)
The backbone property of hinges were taken from section analysis with plastic hinge length assumption of 0.5H.

For ASCE 41-13(2014)
The backbone property of hinges were directly taken from ASCE 41-13 documents tables.

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

Turkish Seismic Design Code 2007 Section Damage Limits

Damage levels	Concrete strain limit*	Steel strain limit
IO	$(\varepsilon_c)_{MN} = 0.0035$ Cover Concrete	$(\varepsilon_s)_{MN} = 0.01$
LS	$(\varepsilon_{cg})_{GV} = 0.0035 + 0.01\,(\rho_s / \rho_{sm}) \leq 0.0135$ Core Concrete	$(\varepsilon_s)_{SL} = 0.04$
CP	$(\varepsilon_{cg})_{GC} = 0.004 + 0.014\,(\rho_s / \rho_{sm}) \leq 0.018$ Core Concrete	$(\varepsilon_s)_{FL} = 0.06$

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

Asce 41-13 Section Damage Limits

Table 10-8. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns

Conditions			Modeling Parameters[a]			Acceptance Criteria[a]		
			Plastic Rotations Angle (radians)		Residual Strength Ratio	Plastic Rotations Angle (radians) — Performance Level		
			a	b	c	IO	LS	CP
Condition i.[b]								
$\frac{P}{A_g f_c'}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.035	0.060	0.2	0.005	0.045	0.060
≥0.6	≥0.006		0.010	0.010	0.0	0.003	0.009	0.010
≤0.1	=0.002		0.027	0.034	0.2	0.005	0.027	0.034
≥0.6	=0.002		0.005	0.005	0.0	0.002	0.004	0.005
Condition ii.[b]								
$\frac{P}{A_g f_c'}$ [c]	$\rho = \frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f_c'}}$ [d]						
≤0.1	≥0.006	≤3 (0.25)	0.032	0.060	0.2	0.005	0.045	0.060
≤0.1	≥0.006	≥6 (0.5)	0.025	0.060	0.2	0.005	0.045	0.060
≥0.6	≥0.006	≤3 (0.25)	0.010	0.010	0.0	0.003	0.009	0.010
≥0.6	≥0.006	≥6 (0.5)	0.008	0.008	0.0	0.003	0.007	0.008
≤0.1	≤0.0005	≤3 (0.25)	0.012	0.012	0.2	0.005	0.010	0.012
≤0.1	≤0.0005	≥6 (0.5)	0.006	0.006	0.2	0.004	0.005	0.006
≥0.6	≤0.0005	≤3 (0.25)	0.004	0.004	0.0	0.002	0.003	0.004
≥0.6	≤0.0005	≥6 (0.5)	0.0	0.0	0.0	0.0	0.0	0.0
Condition iii.[b]								
$\frac{P}{A_g f_c'}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.0	0.060	0.0	0.0	0.045	0.060
≥0.6	≥0.006		0.0	0.008	0.0	0.0	0.007	0.008
≤0.1	≤0.0005		0.0	0.006	0.0	0.0	0.005	0.006
≥0.6	≤0.0005		0.0	0.0	0.0	0.0	0.0	0.0
Condition iv. Columns controlled by inadequate development or splicing along the clear height[b]								
$\frac{P}{A_g f_c'}$ [c]	$\rho = \frac{A_v}{b_w s}$							
≤0.1	≥0.006		0.0	0.060	0.4	0.0	0.045	0.060
≥0.6	≥0.006		0.0	0.008	0.4	0.0	0.007	0.008
≤0.1	≤0.0005		0.0	0.006	0.2	0.0	0.005	0.006
≥0.6	≤0.0005		0.0	0.0	0.0	0.0	0.0	0.0

NOTE: f_c' is in lb/in.^{2} (MPa) units.

[a] Values between those listed in the table should be determined by linear interpolation.

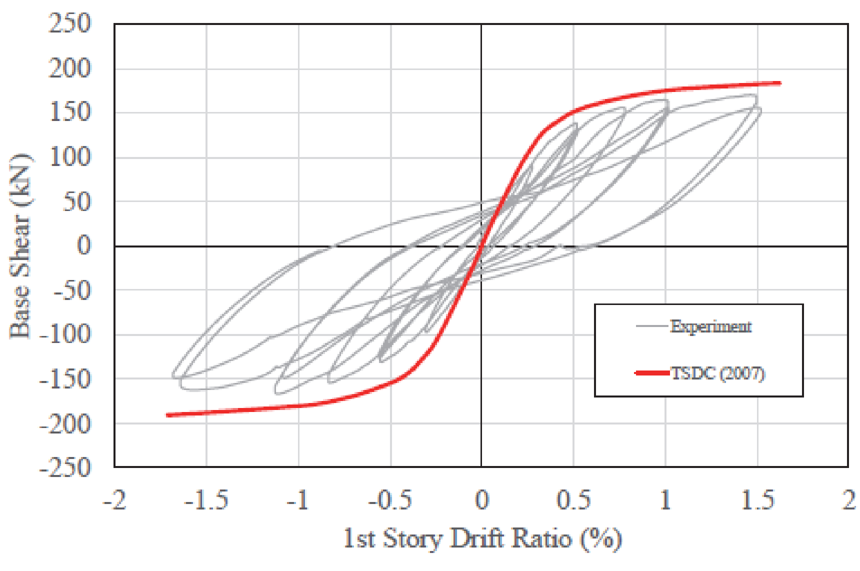
[b] Refer to Section 10.4.2.2.2 for definition of conditions i, ii, and iii. Columns are considered to be controlled by inadequate development or splices where the calculated steel stress at the splice exceeds the steel stress specified by Eq. (10-2). Where more than one of conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.

[c] Where $P > 0.7 A_g f_c'$, the plastic rotation angles should be taken as zero for all performance levels unless the column has transverse reinforcement consisting of hoops with 135-degree hooks spaced at $\leq d/3$ and the strength provided by the hoops (V_s) is at least 3/4 of the design shear. Axial load P should be based on the maximum expected axial loads caused by gravity and earthquake loads.

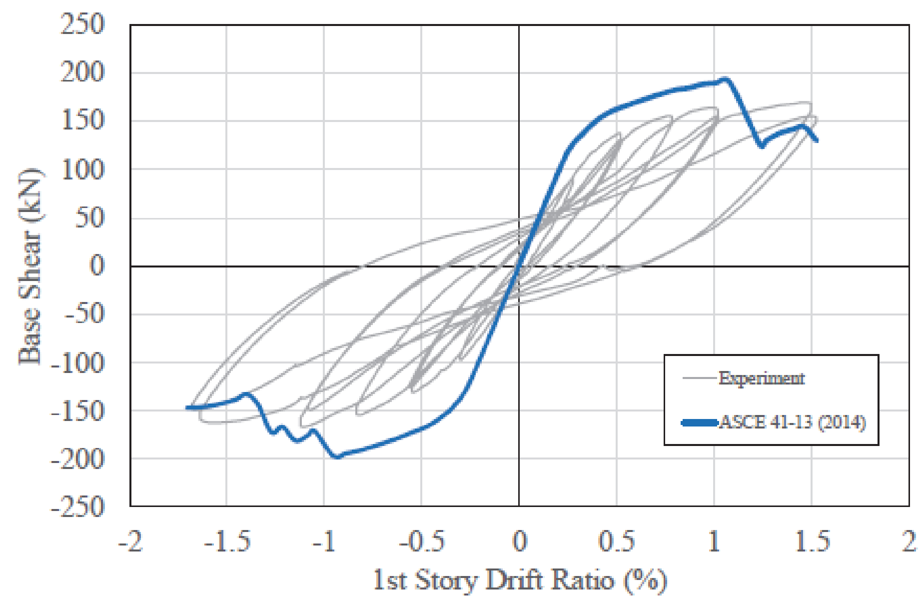
[d] V is the design shear force from NSP or NDP.

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

TSDC (2007)

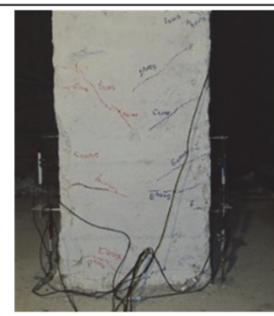


ASCE 41-13



Comparison of Damages

. 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.

Comparison of Damages

In the case of beams

	TSDC (2007) (Rotation)	ASCE 41-13 (2014) (Rotation)
IO	0.006	0.005
LS	0.024	0.019
CP	0.028	0.029

The beam compression bars buckled at 0.75 % drift ratio (The measured rotation at this drift is 0.012).

Both approach overestimate the damage of the beams.

Conclusions

- 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 column damage limits, TSDC (2007) gives 20% conservative limitations when they are compared with the ASCE 41-13 (2014) limitations. In case of beam damage limit, both assessment approach overestimate the beam damages.

Thank You

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