Field testing of substandard full scale RC buildings for seismic performance assessment: Quasi-static tests

M. Comert, C. Demir, A.O. Ates, E. Tore, O. Ozeren,
A. Moshfeghi, S. Khoshkholghi, K. Orakcal, A. Ilki

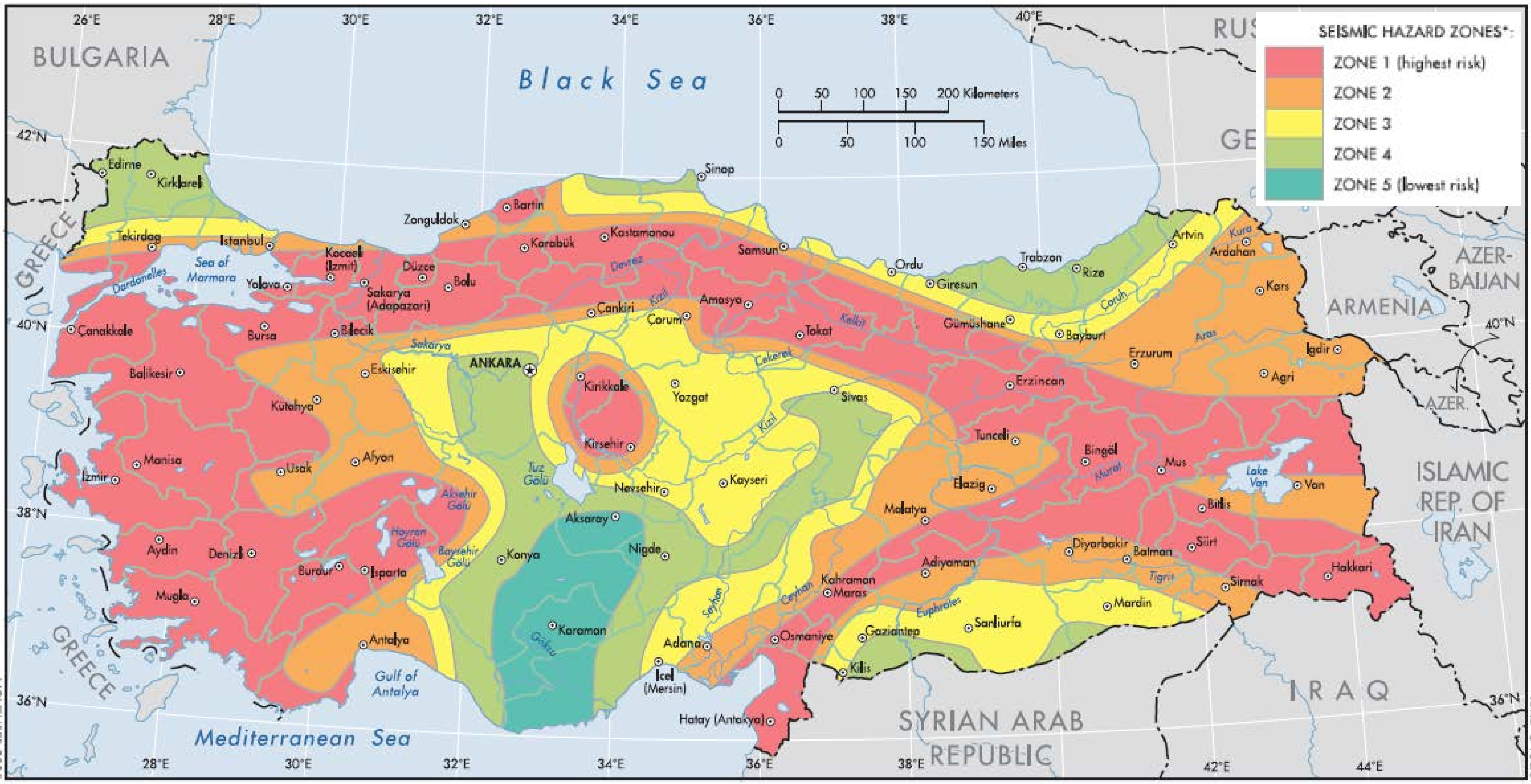
Content

- Outline of the Project
- Introduction
- Building Characteristics
- Site Preparations
- Test Setup
- Observations and experimental results
- Predictions and comparisons

Project Outline

- **Project name:** Seismic performance assessment of existing buildings through full-scale tests
- **Aim:** To supply experimental data on seismic performances of structures of actual scale
- **Content:** Site tests on two full-scale sub-standard buildings
- **Supported by:** Istanbul Development Agency, Istanbul Technical University and sponsors
- **Duration:** 9 months + 3 months extension
- **Team:** 3 professors, 2 post-docs, 6 PhD candidates, 5 MSc students, 1 undergraduate student
- **Advisors:** 6 professors from Japan and Turkey
- **Budget:** Approximately 300 000 USD
- **Some figures:** 200 m^3 concrete poured, 10 t steel bars for construction, 5.5 t kg steel for test setup

Seismicity of Turkey

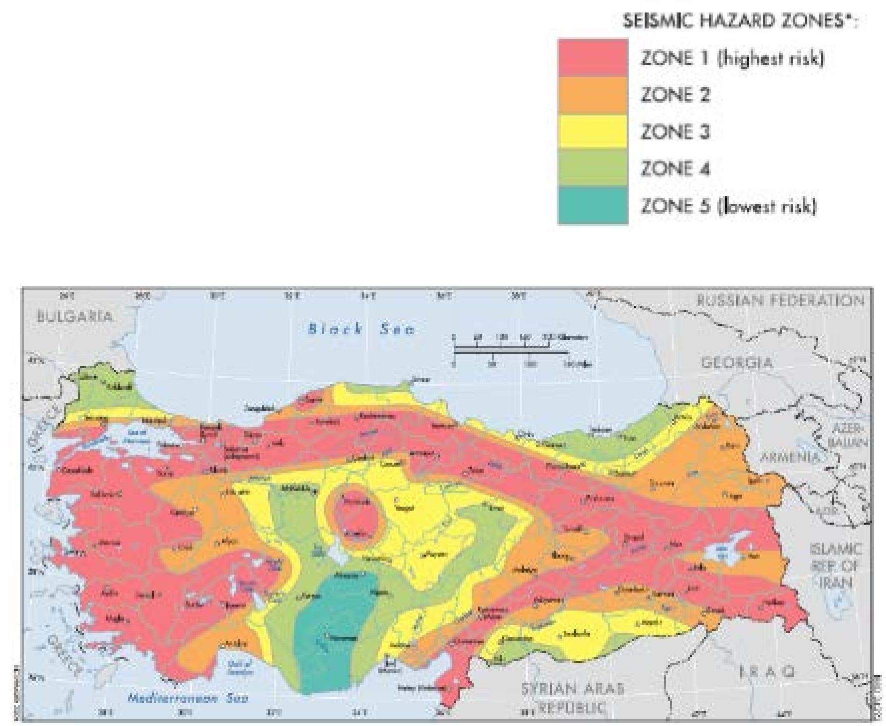


Source: World Bank

Seismicity of Turkey

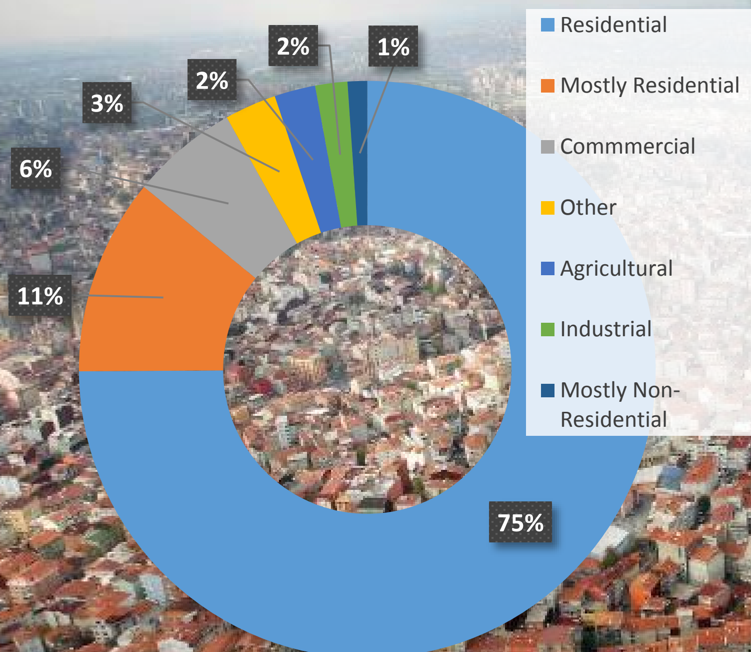
Earthquake Zones	Area (m2)	Percentage of Population
Zone 1	328995	45
Zone 2	186411	26
Zone 3	139594	15
Zone 4	97894	13
Zone 5	32051	2
Total	784945	100

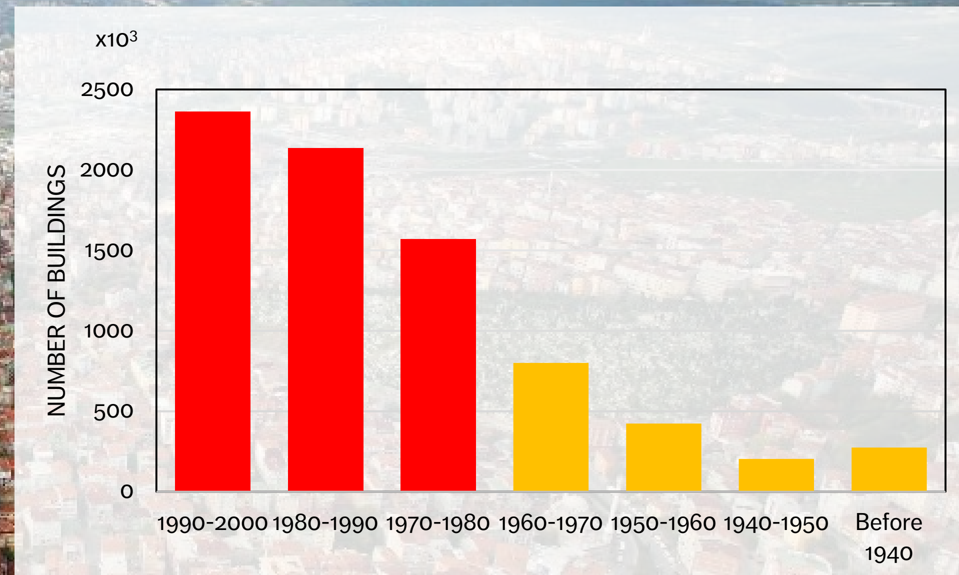
Source: Turkish Earthquake Foundation



According to 2000 Building Census

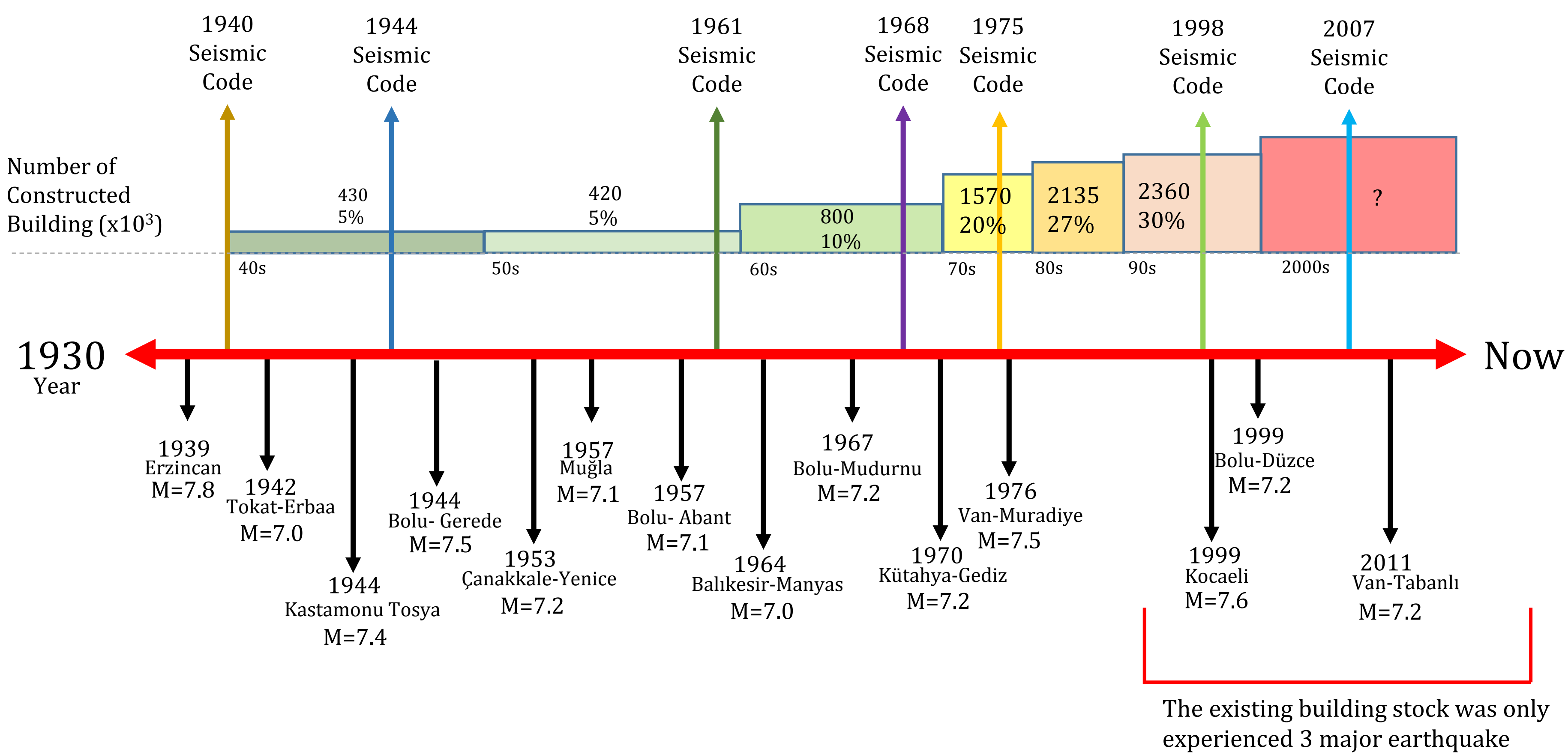
Building Stock of Turkey





78 % of buildings was constructed
between 1970-2000 years

Development of Seismic Code and Major Earthquakes



Properties of Building Stock

Huge part of buildings in Turkey was constructed between 1970-2000 years

Between these years;

- Some part of buildings was constructed as non-engineered due to inadequate legal inspections.

- Some part was designed according to relevant seismic code but not constructed as designed.

- Some part was designed according to relevant seismic code and constructed as designed.

Most Common Deficiencies

Based on observations made after recent EQs

Low Concrete Quality

Before 2000's,
Most of buildings in Turkey were constructed with hand-mixed concrete.

Large Stirrup Spacing

Approximately 200-300 mm

Improper Stirrup Hook Details

90° Hooks
and
Inadequate Hook
Length

Plain Bars

Plain bars, inadequate lap splice lengths, missing hooks

Test Buildings

TB2

TEST BUILDING 2

REPRESENTATIVE SUB-STANDARD BUILDING

Low Strength Concrete

Plain Bars

Large Stirrup Spacing

Improper Hook Details

TB1

TEST BUILDING 1

PART OF AN ACTUAL BUILDING

Low Strength Concrete

Plain Bars

Large Stirrup Spacing

Improper Hook Details

What are the differences ?

TB1;

- Weak Beam – Strong Column
- Lower Axial Load Level
- Higher Shear Demand Capacity Ratio

TB2;

- Strong Beam – Weak Column
- Higher Axial Load Level
- Lower Shear Demand Capacity Ratio

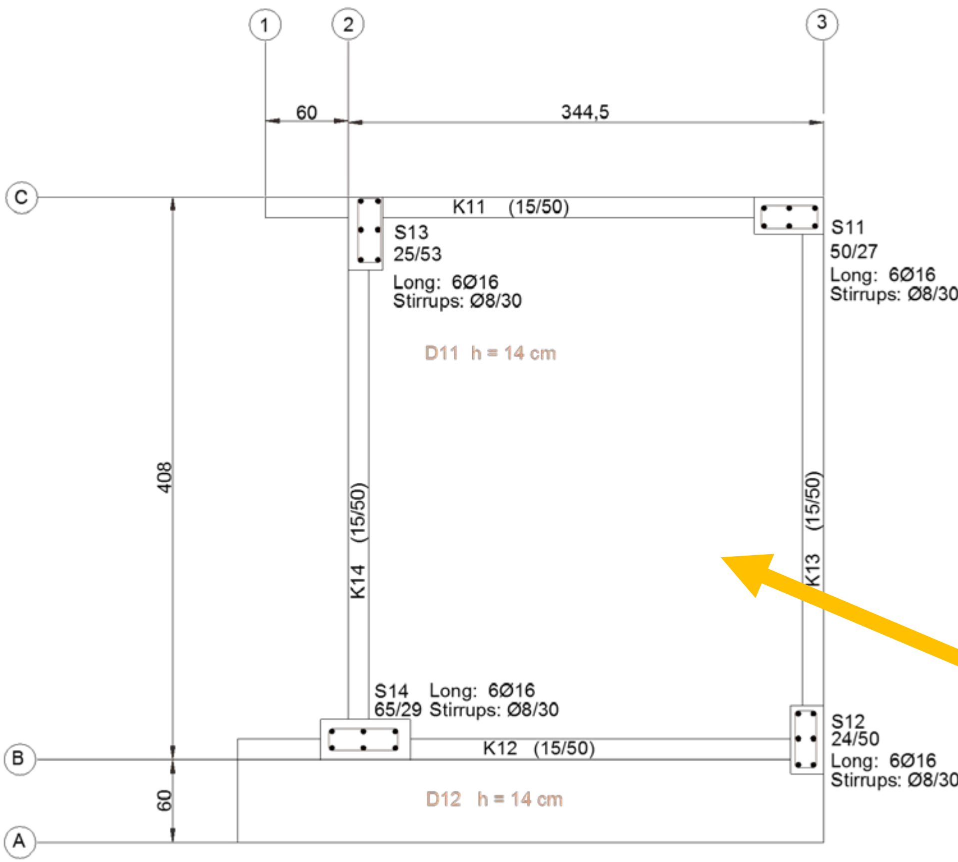
Test Building 1

Concrete
Compressive
Strength 13.5 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)

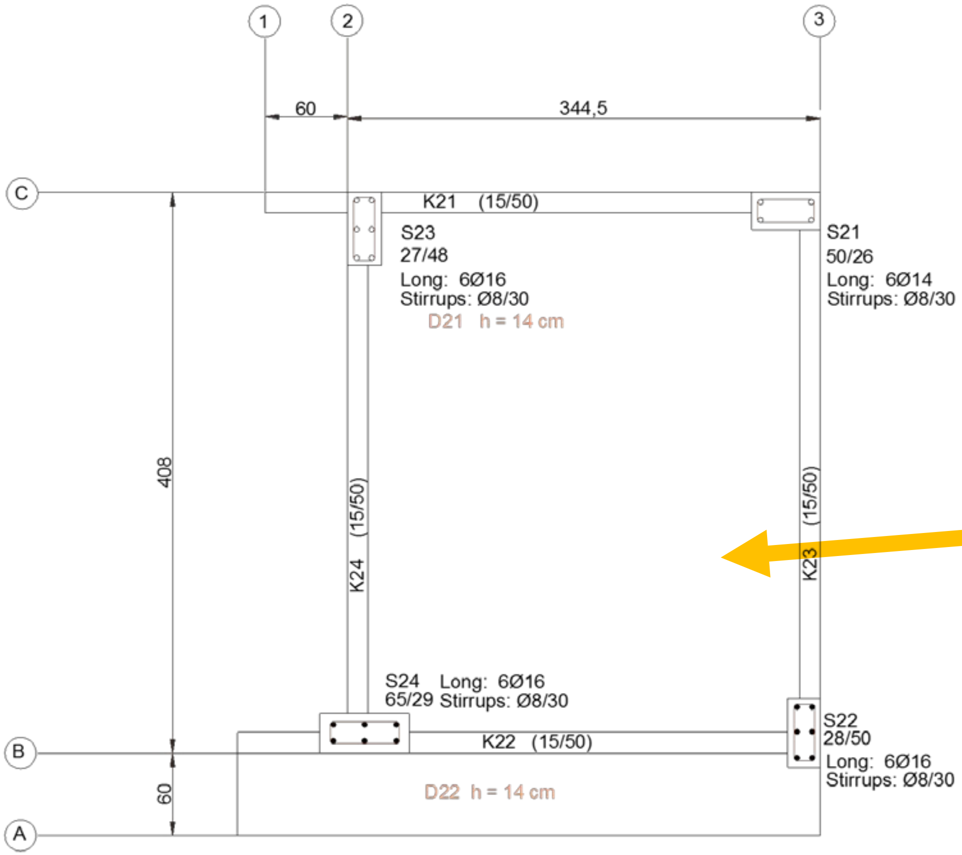
Test Building 1



1st Story Plan

H_{story}=2.7 m

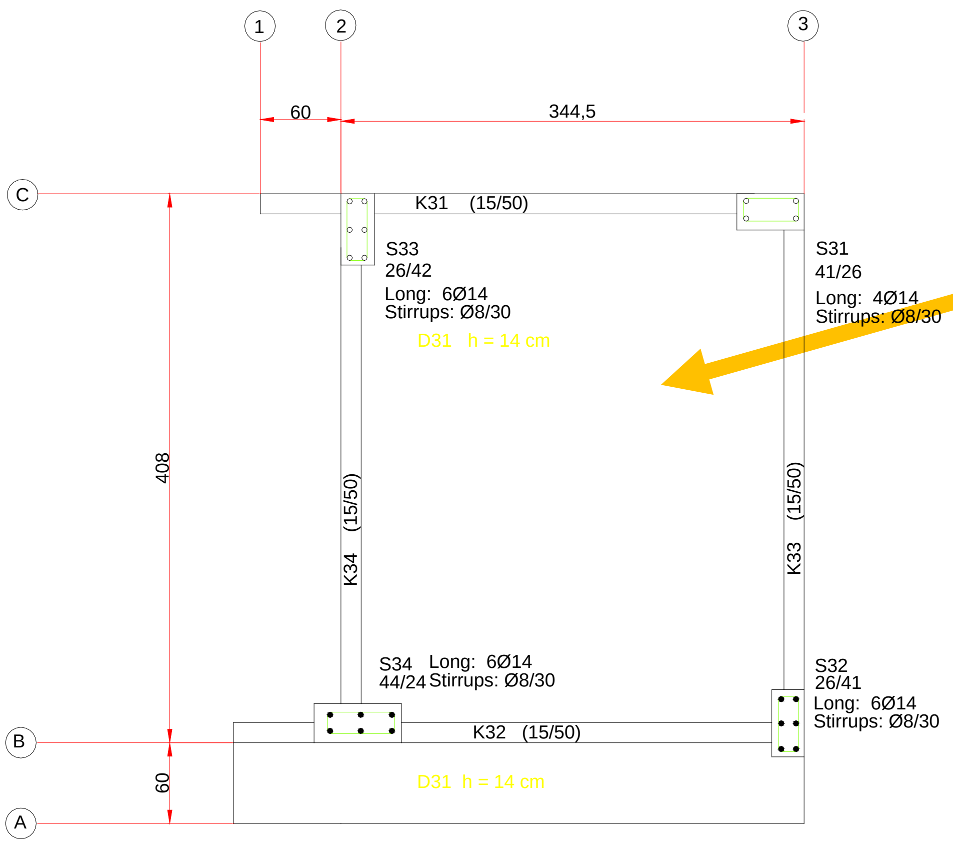
Test Building 1



2nd Story Plan

H_{story}=2.7 m

Test Building 1

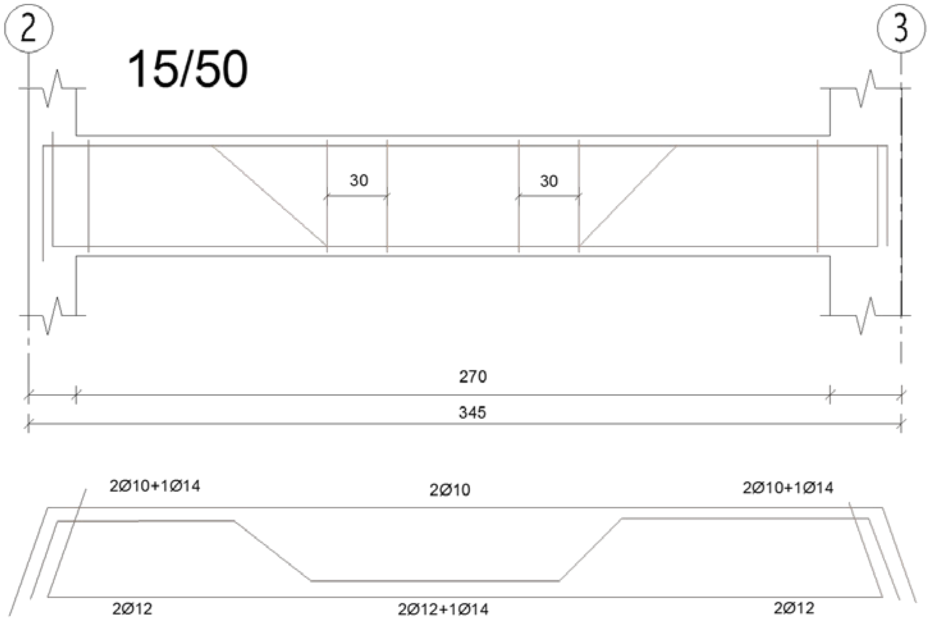


3rd Story Plan

H_{story}=2.7 m

Test Building 1

K11, K12,K21,K22,K31,K32



Beam Reinforcement Details

Test Building 1

Column
Axial Load Level: %10

Column
Shear Demand Capacity Ratio: 0.65

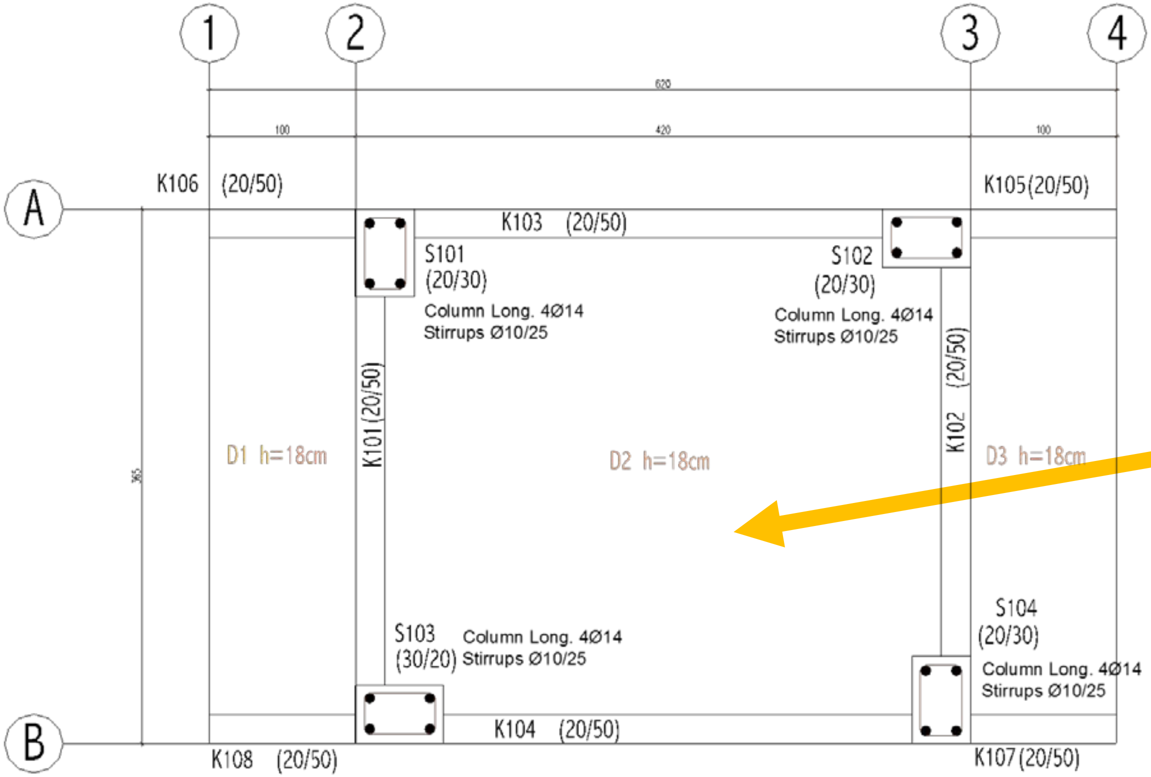
Test Building 2

Concrete
Compressive
Strength 10 MPa

Reinforcing Bars

Column Long. f_y=350 MPa
(lap splices with hook, 50ϕ)
Beam Long. f_y=350 MPa
(lap splices with hook)

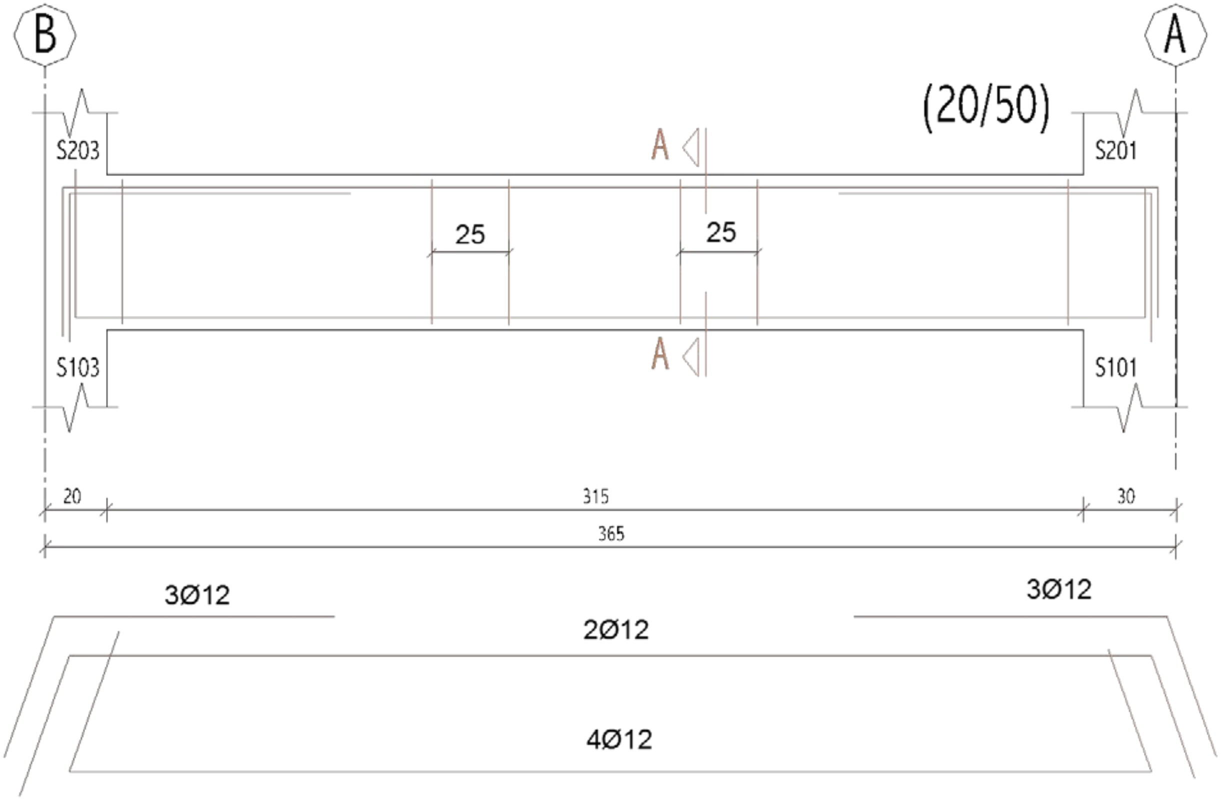
Test Building 2



Story Plan – All Stories

H_{story}=3.0 m

Test Building 2



Beam Reinforcement Details

Beams in Loading Direction

Test Building 2

Column
Axial Load Level: %25

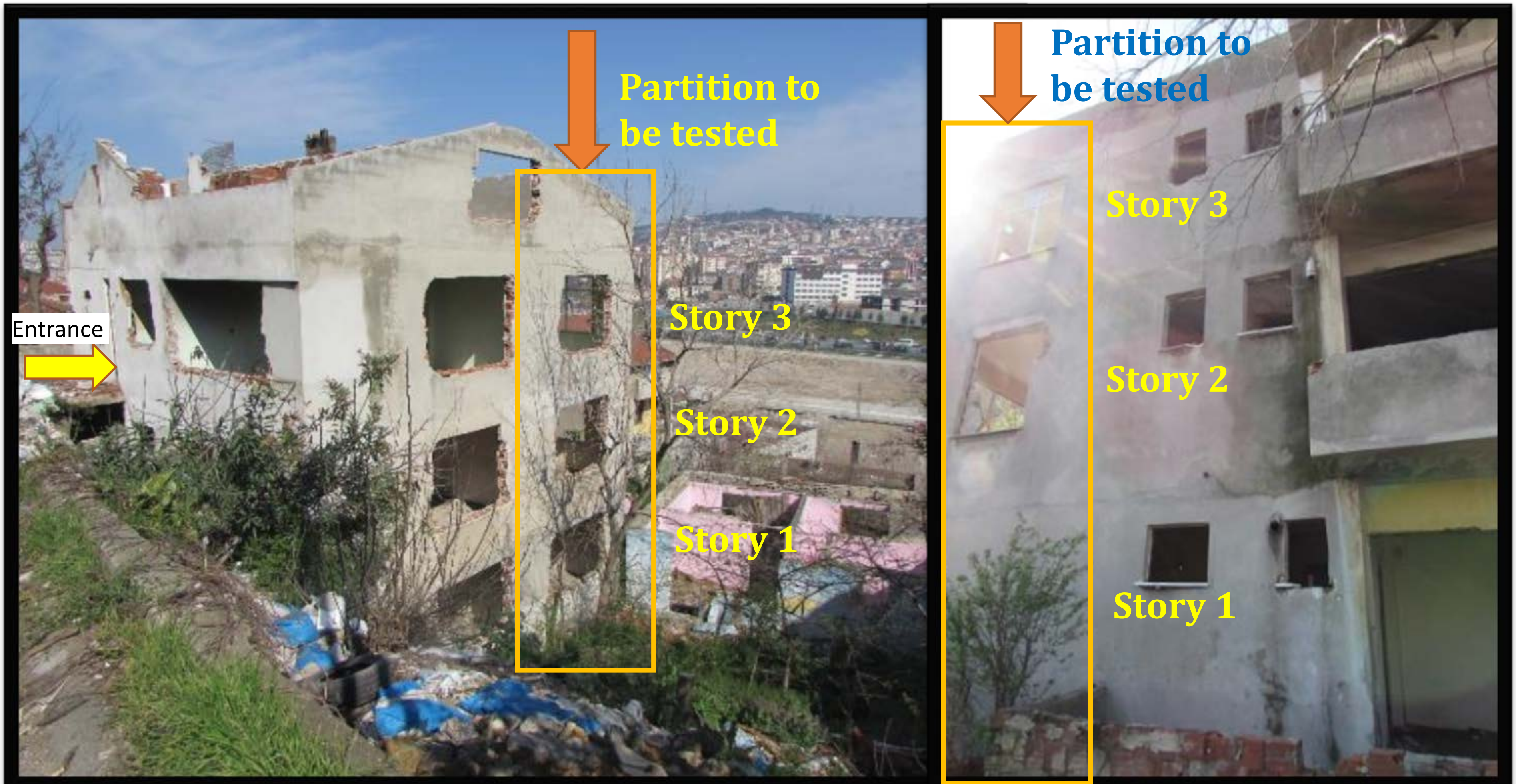
Column
Shear Demand Capacity Ratio: 0.30

Site Preparations

- Test site @ Fikirtepe Urban Renewal Area

Site Preparations

- Test Building 1



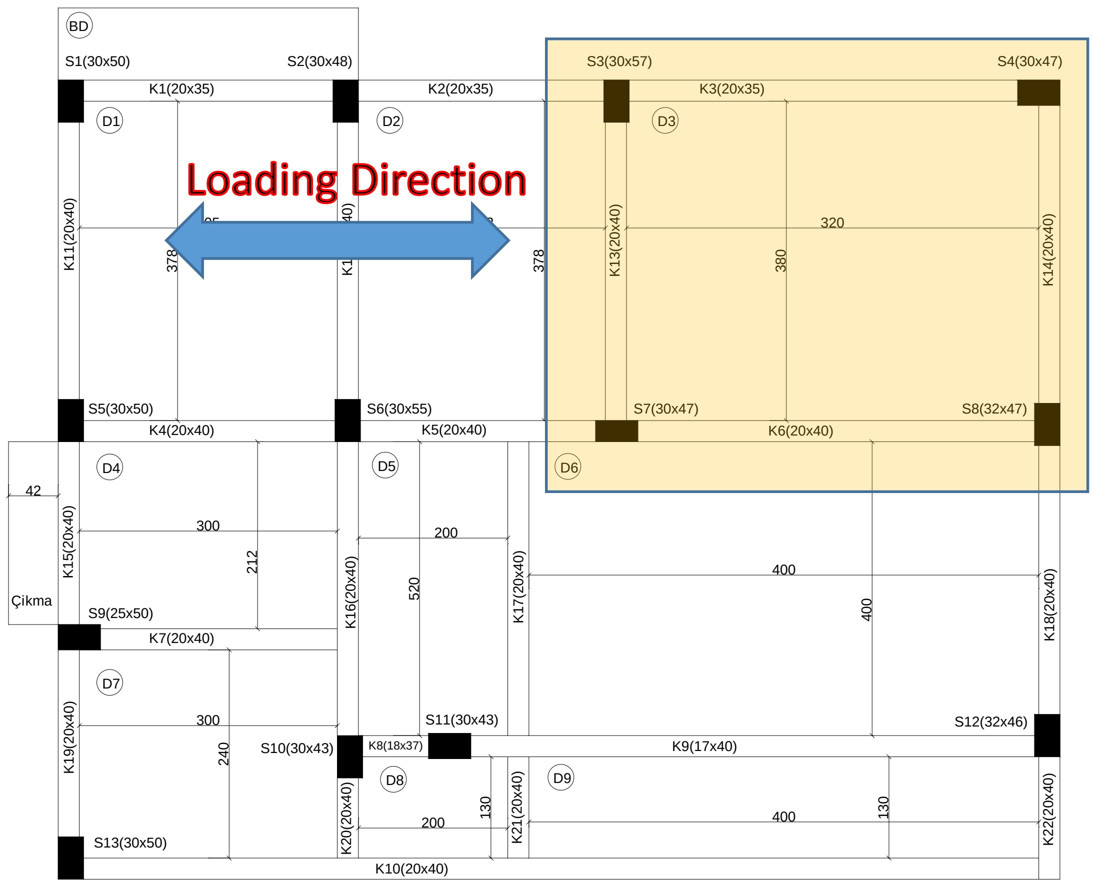
Site Preparations

- Site survey, material sampling, dimensions, reinforcement details, etc.




Site Preparations

- Demolution for Test Building 1



Test Building 1 (TB1)

Site Preparations

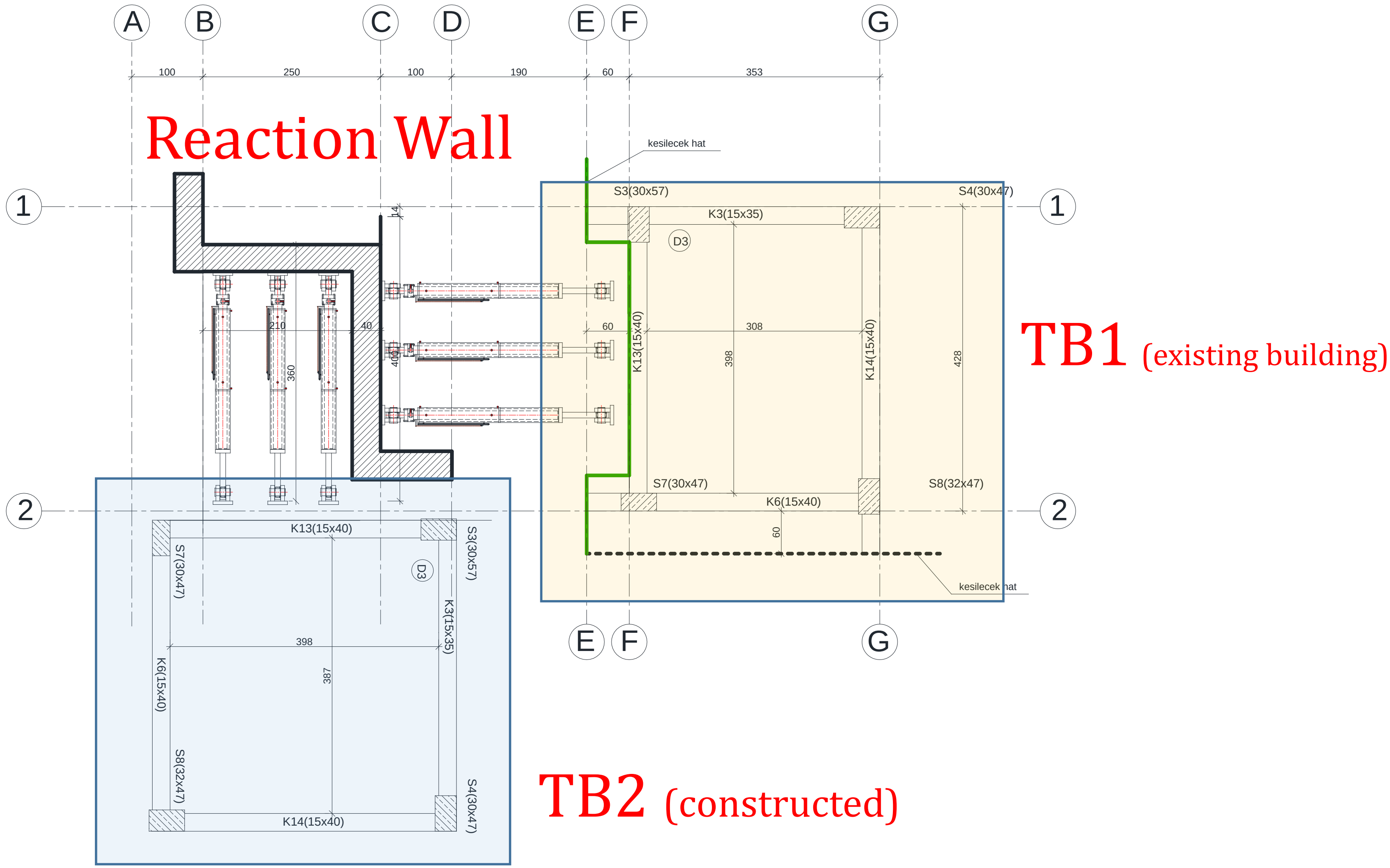
- Demolution

Site Preparations

- Demolution

Site Preparations

- Test site layout



Site Preparations

- Pouring of lean concrete

Site Preparations

- Foundation construction
- 60 cm thick

Site Preparations

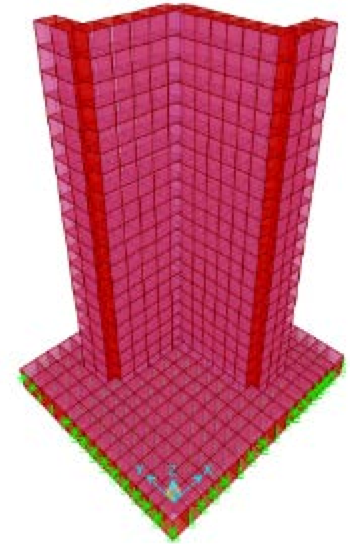
- Existing foundation of TB 1

Site Preparations

- New mat foundation for TB 1

Site Preparations

- Reaction wall construction
  - 50 cm thick wall
  - Wing form for testing two buildings consequtively



Site Preparations

- Construction of Test Building 2 (TB2)

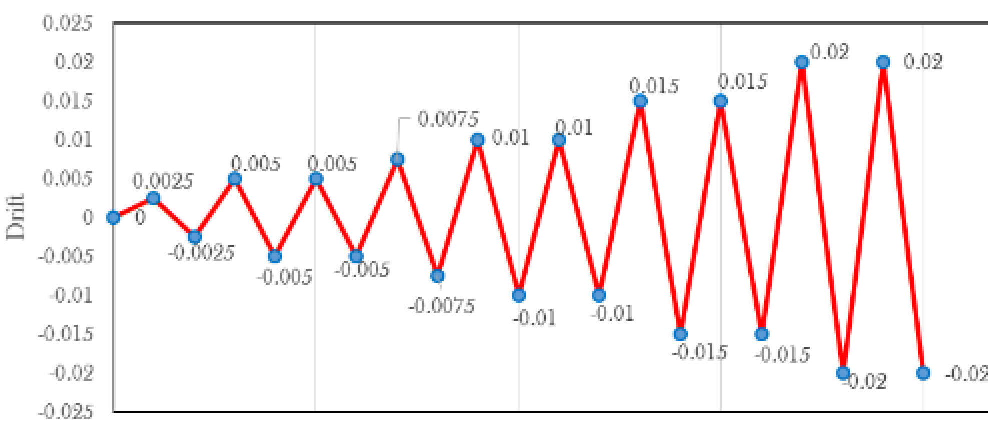



Test Setup: Loading

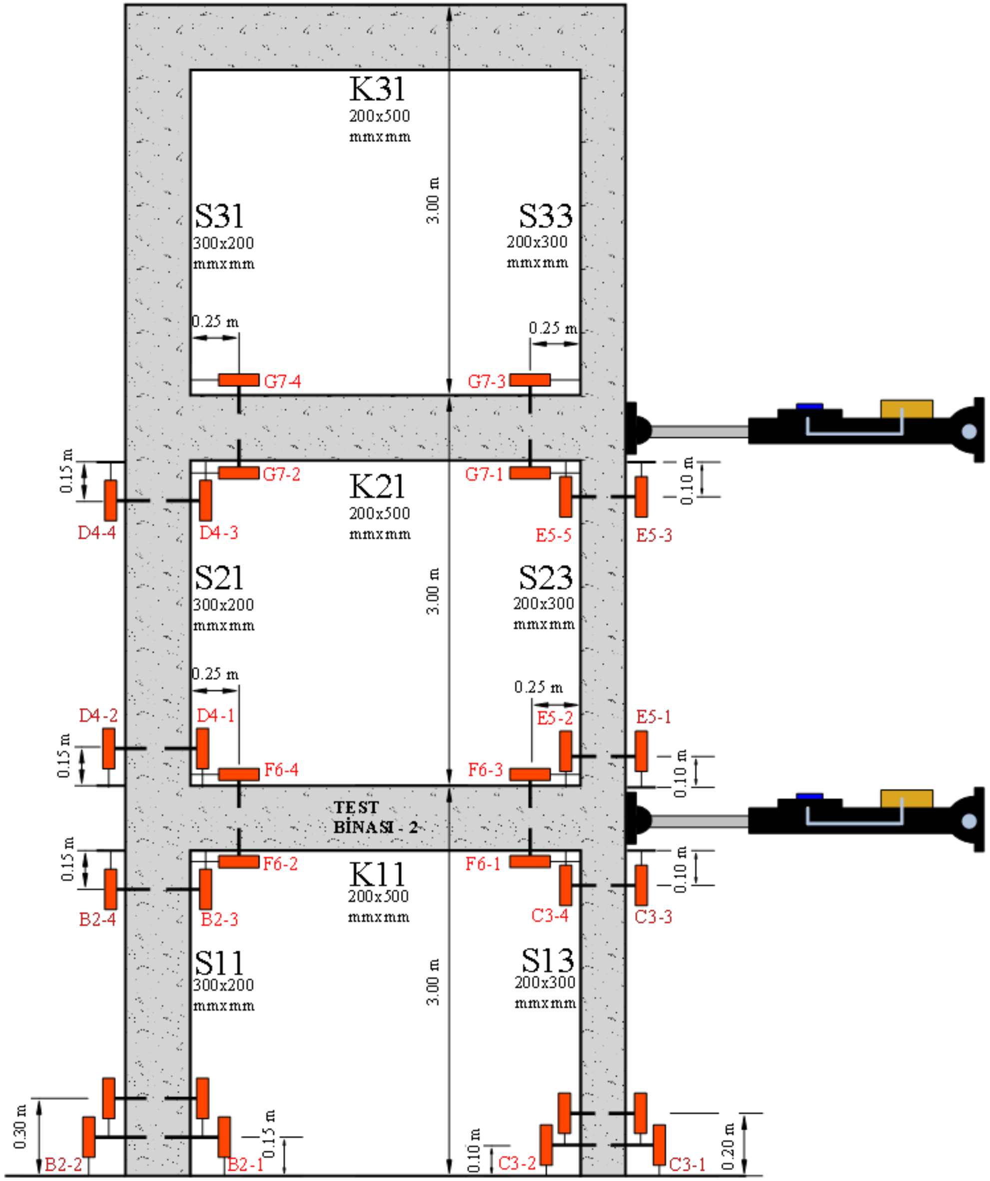
- Reversed cyclic loading with three hydraulic actuators (300 kN load and 800 mm displacement capacities)
- Diplacement controlled loading for TB1
- Displacement and load controlled loading for TB2
- Load distribution in elevation(2P-P) kept constant

Test Setup: Loading

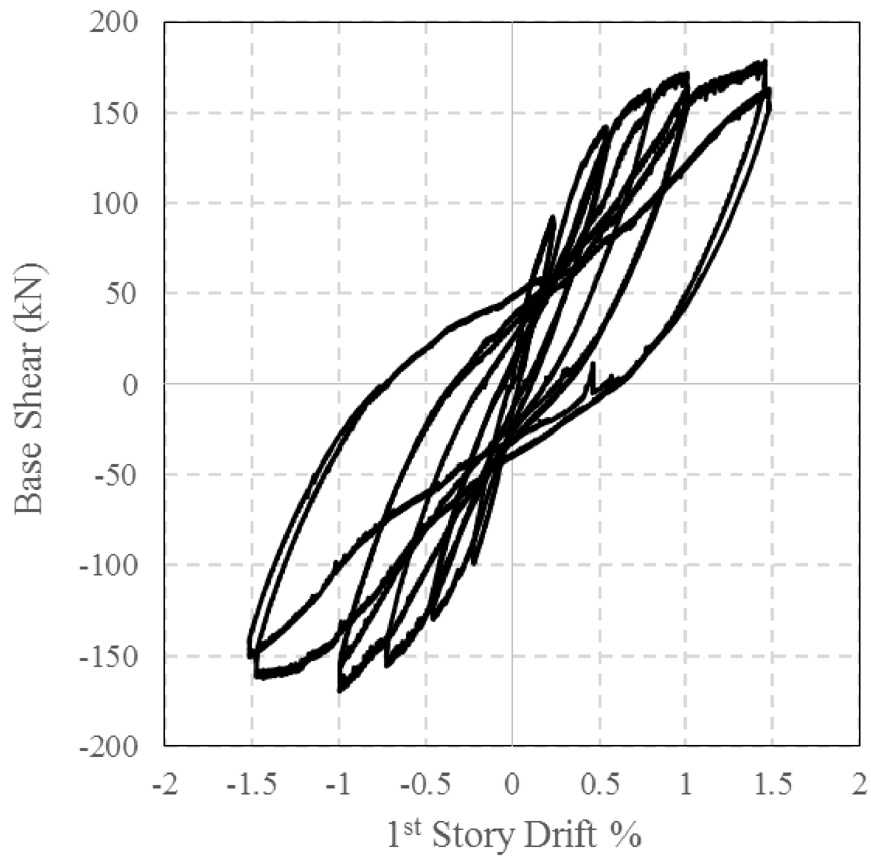
- TB1 loaded to 1.5% DR
- TB2 loaded to 4% DR Reversed cyclic until 3% DR then cyclic until 4% DR



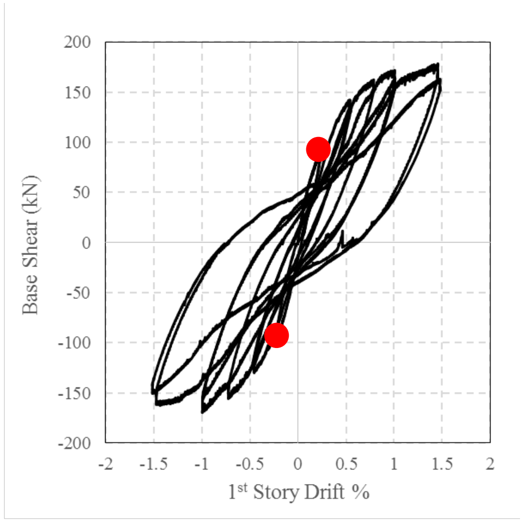
Test Setup: Measurement System



Observations and Test Results: TB1



Observations and Test Results: TB1



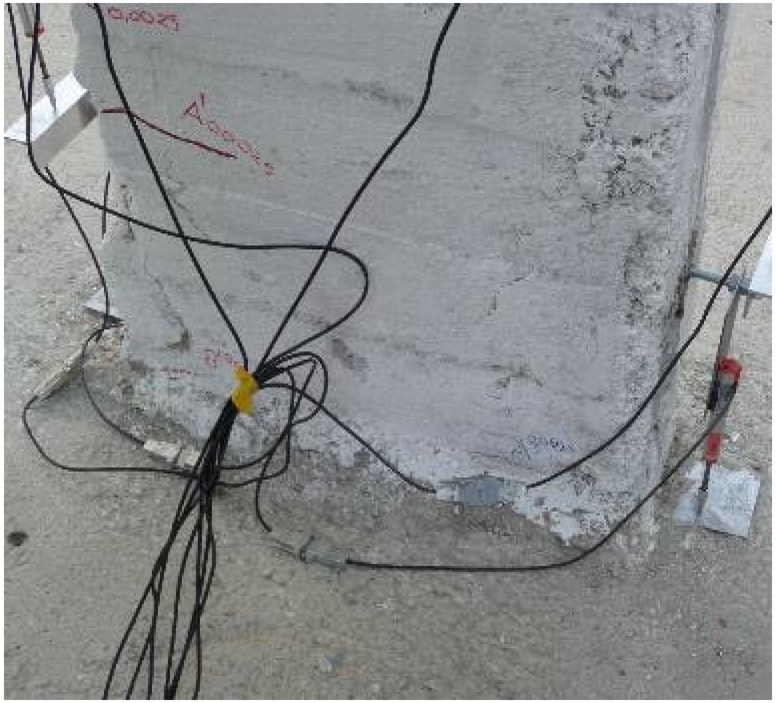
% 0.25 Drift Ratio – 1cycle

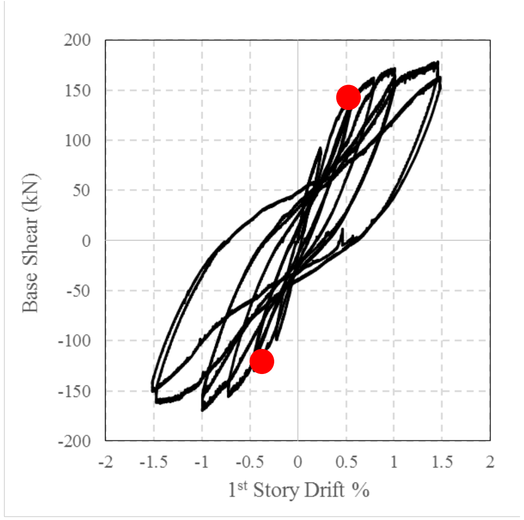
Beams

W_{max}=0.2 mm

Columns

W_{max}<0.1 mm



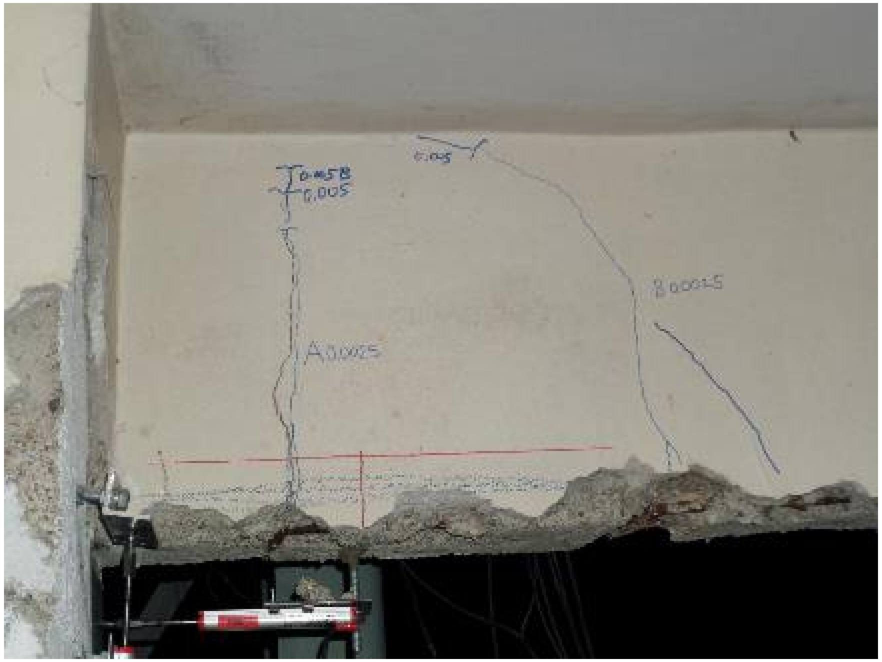


Observations and Test Results: TB1

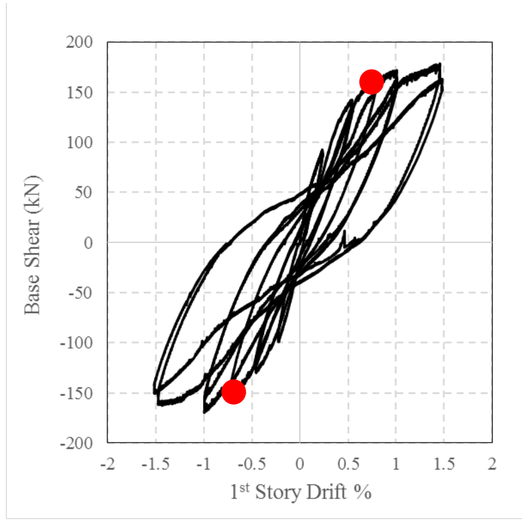
% 0.50 Drift Ratio - 2 cycles

Beams
W_{max}=1.2 mm

Columns
W_{max}=0.6 mm



Observations and Test Results: TB1

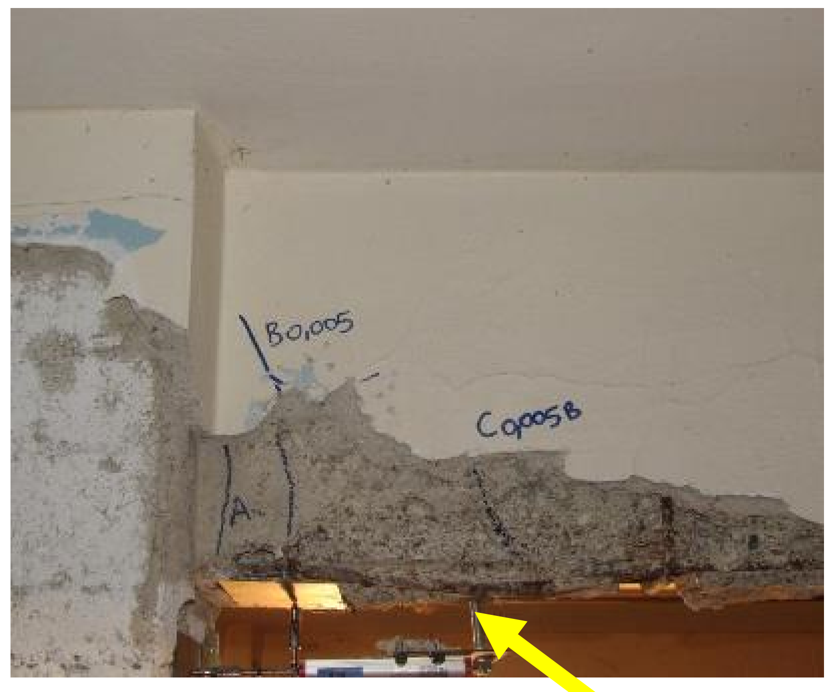


% 0.75 Drift Ratio - 1 cycle

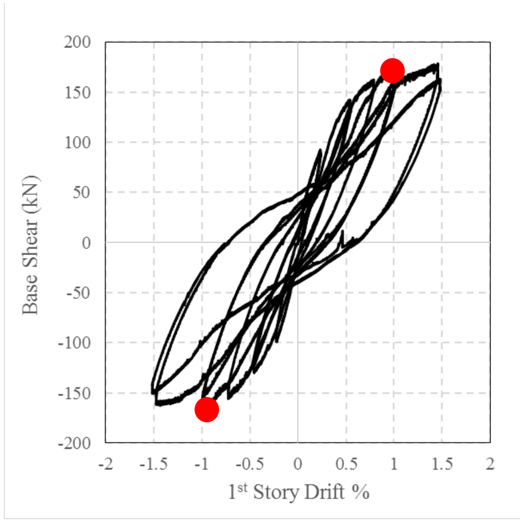
Beams
W_{max}=2.2 mm
Crushing at positive peak

Bucking of bars at negative peak (K11)

Columns
W_{max}=1.6 mm



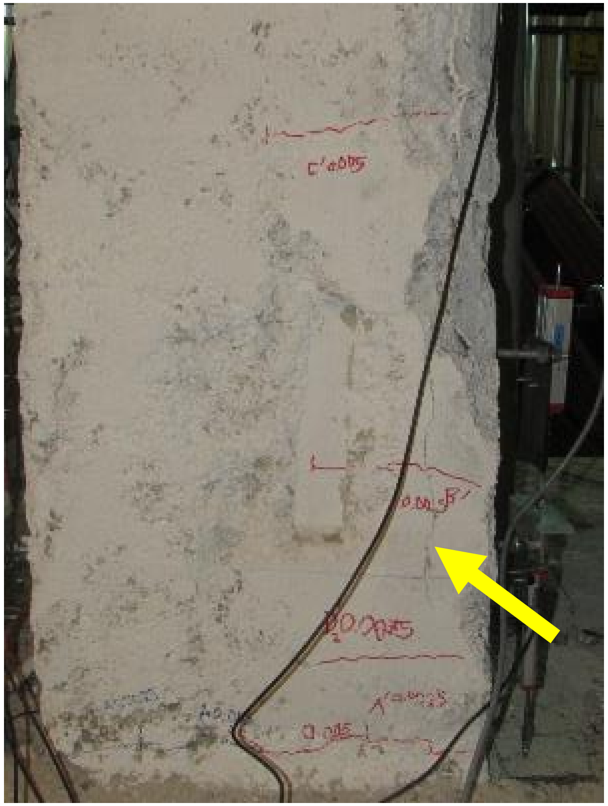
Observations and Test Results: TB1



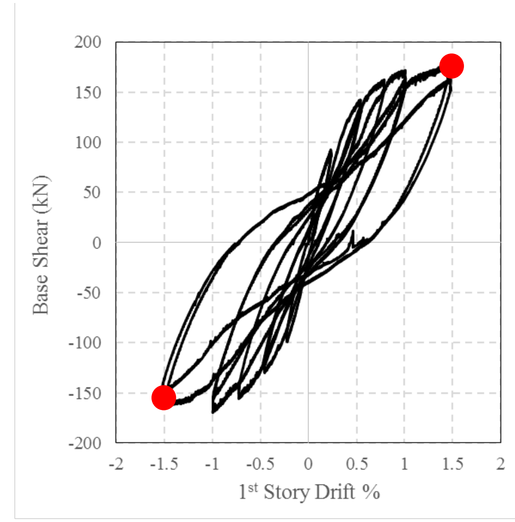
% 1.00 Drift Ratio - 2 cycles

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

Columns
W_{max}=1.8 mm
Vertical Cracks



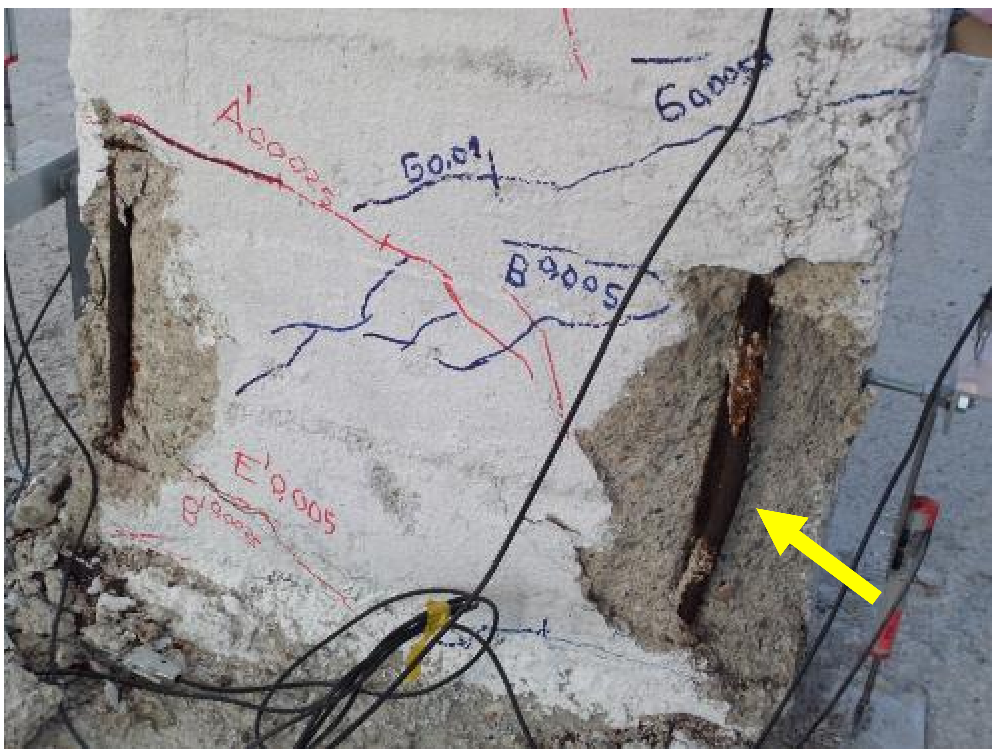
Observations and Test Results: TB1



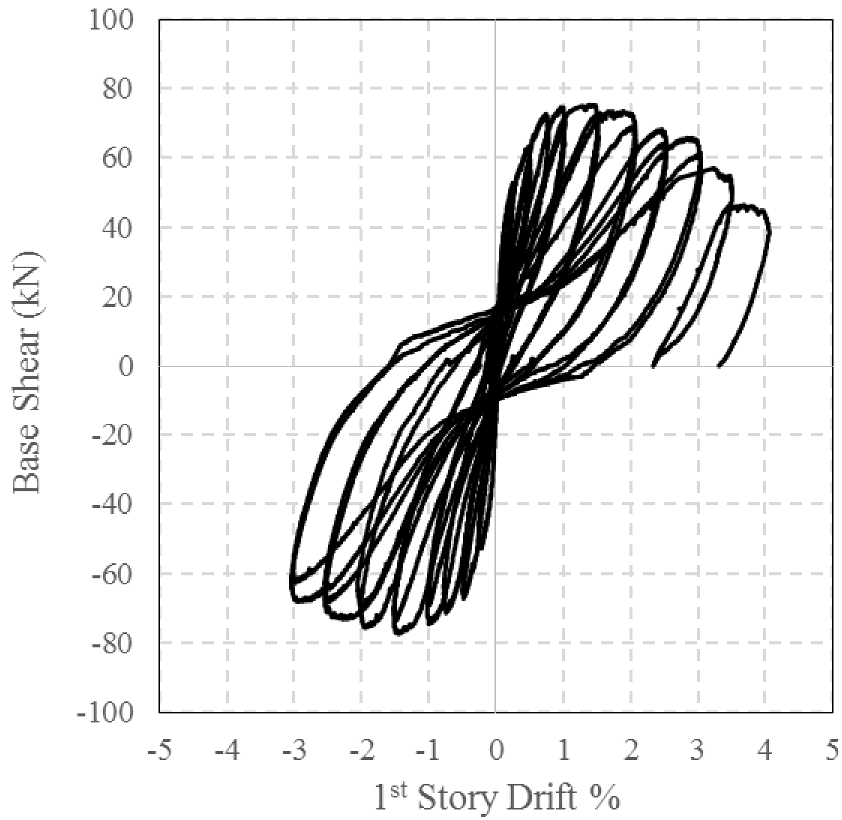
% 1.50 Drift Ratio - 2 cycles

Beams
W_{max}=9 mm

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

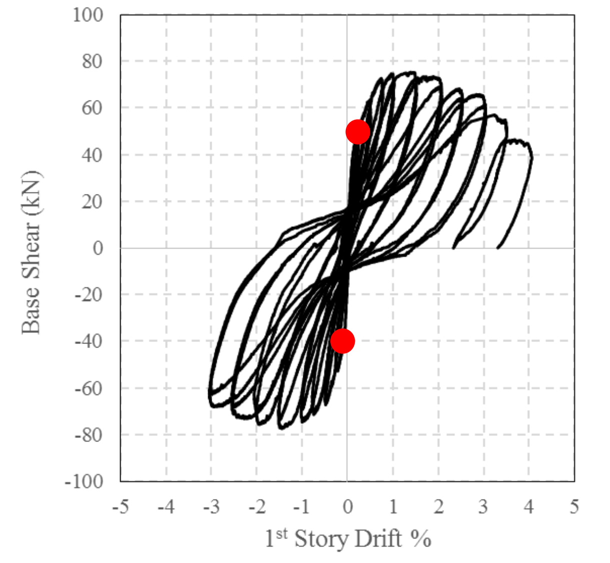


Observations and Test Results: TB2



Observations and Test Results: TB2

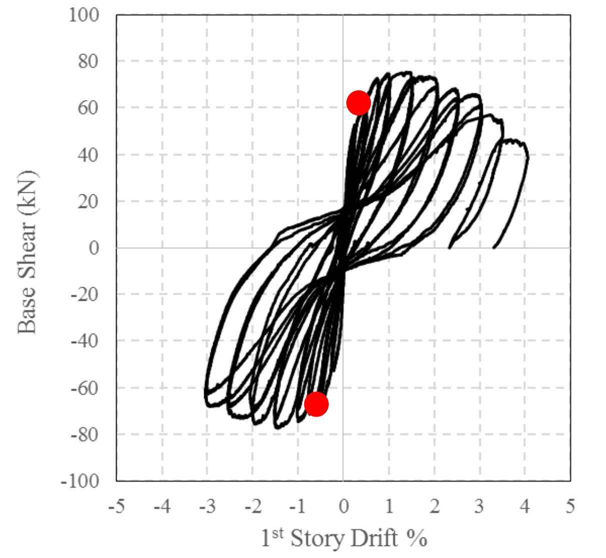
% 0.25 Drift Ratio – 1cycle



Beams	Columns
No damage	W_{max}=0.1 mm

Observations and Test Results: TB2

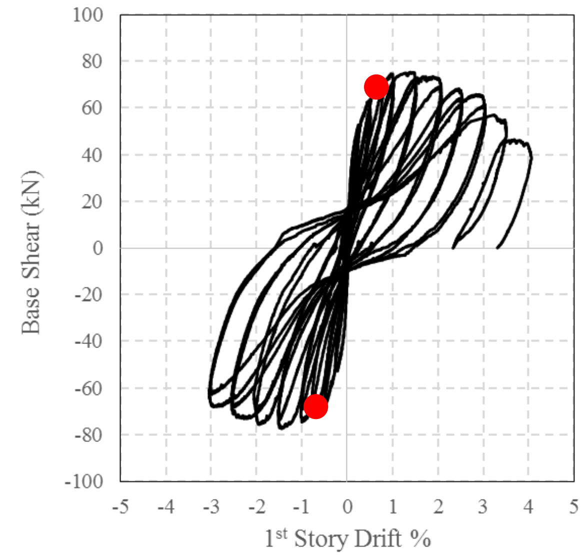
% 0.50 Drift Ratio – 2cycle



Beams	Columns
No damage	W_{max}=0.3 mm

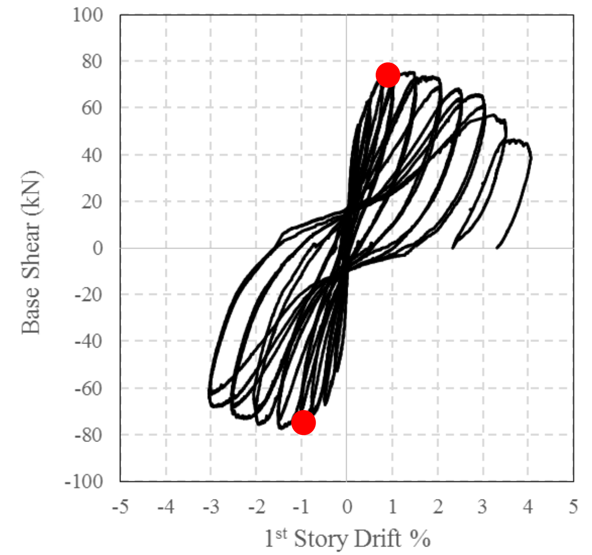
Observations and Test Results: TB2

% 0.75 Drift Ratio – 1cycle



Beams	Columns
No damage	W_{max}=1.4 mm

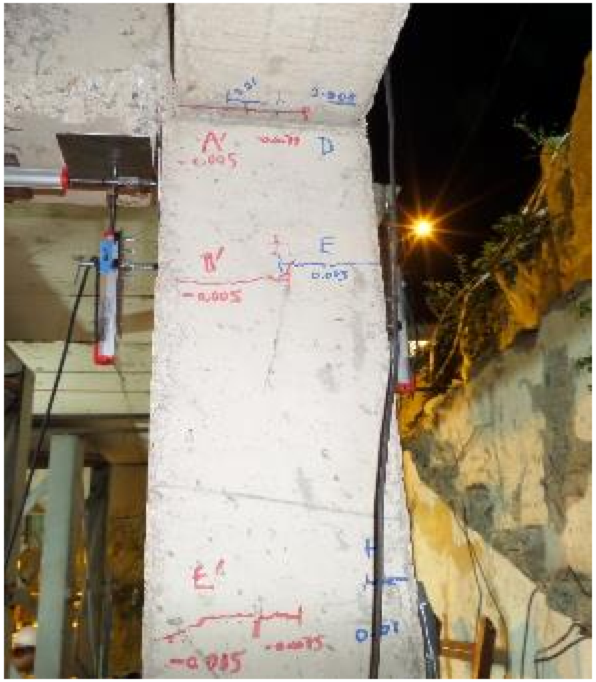
Observations and Test Results: TB2



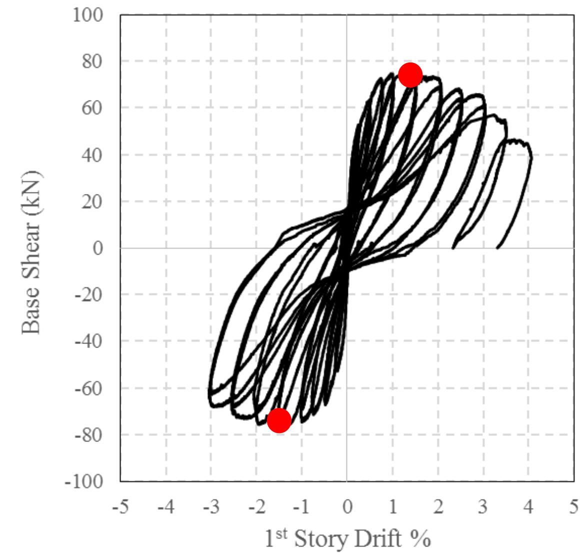
% 1.00 Drift Ratio – 2cycle

Beams
No damage

Columns
W_{max}=1.7 mm



Observations and Test Results: TB2



% 1.50 Drift Ratio – 2cycle

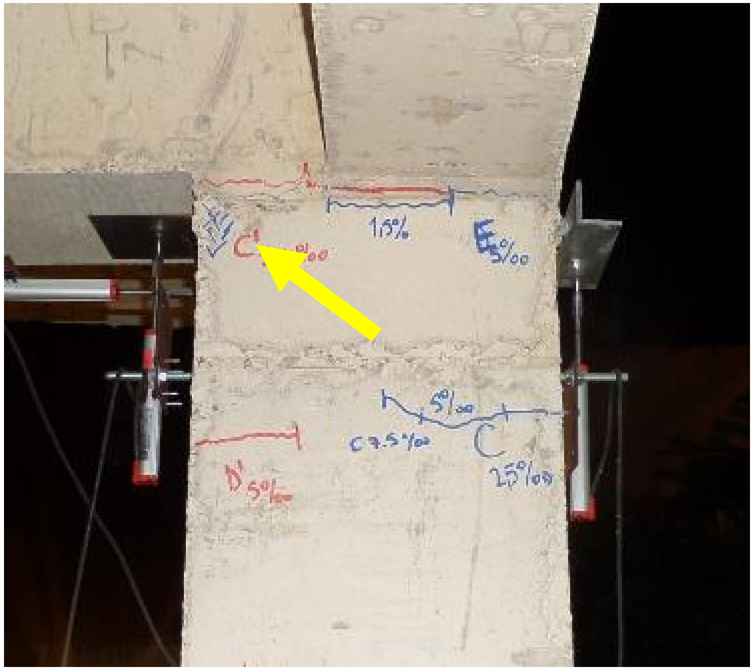
Beams

No damage

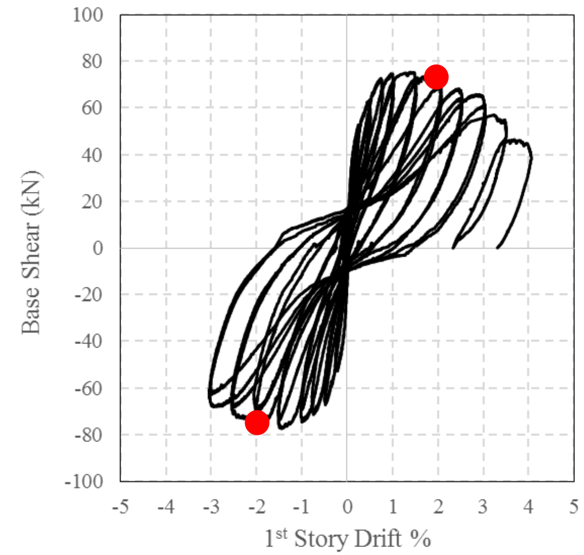
Columns

W_{max}=3.5 mm

First Concrete Crushing



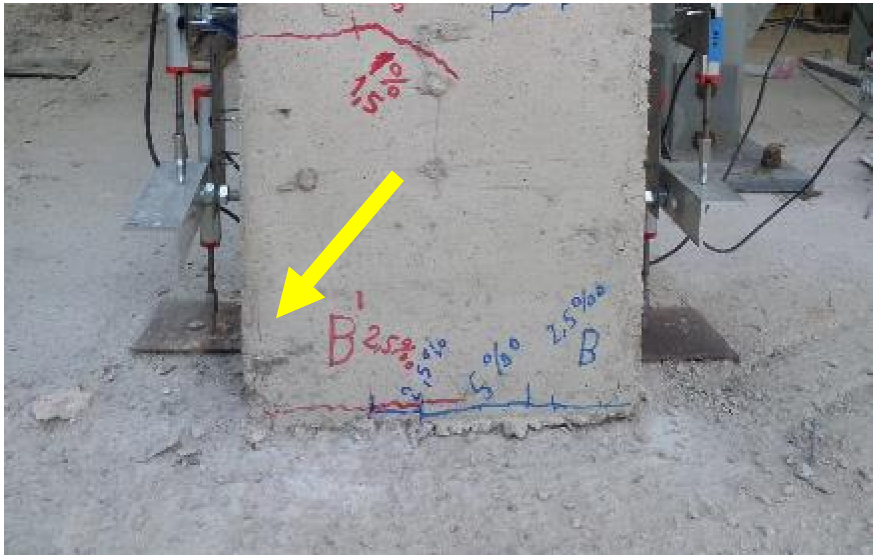
Observations and Test Results: TB2



% 2.00 Drift Ratio – 2cycle

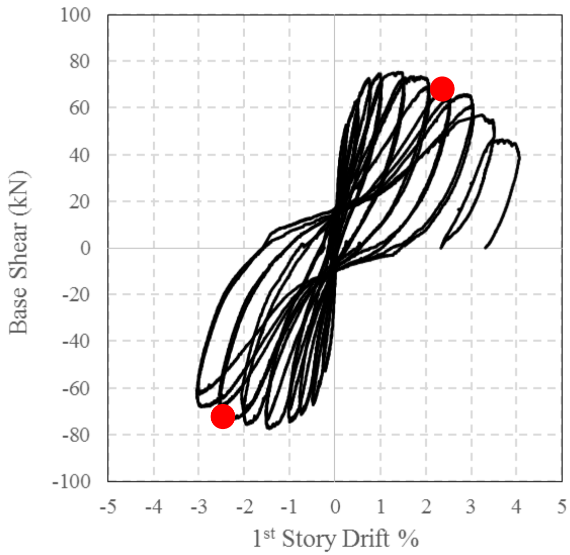
Beams
No damage

Columns
W_{max}=4.5 mm
Concrete Crushing
Vertical Cracks



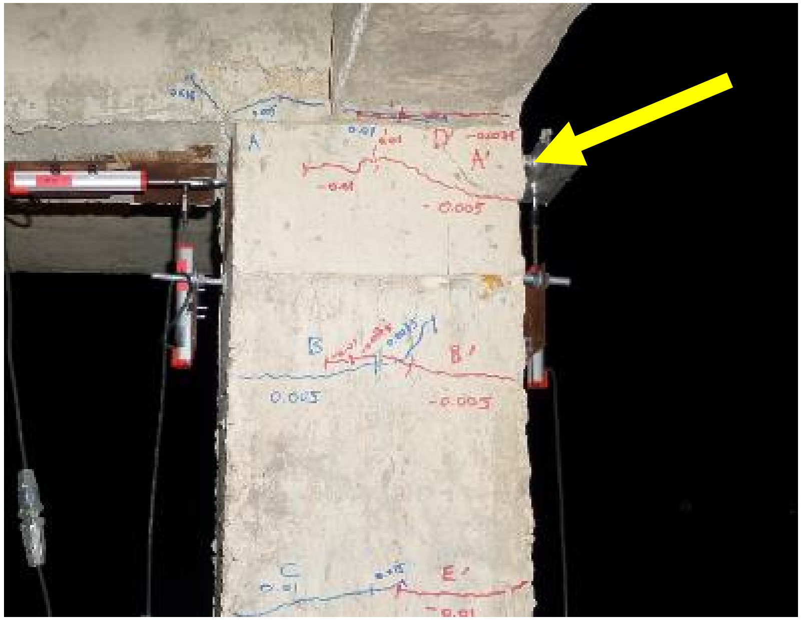
Observations and Test Results: TB2

% 2.50 Drift Ratio – 2cycle



Beams
No damage

Columns
W_{max}=7 mm
Beginning of Concrete Cover Spalling

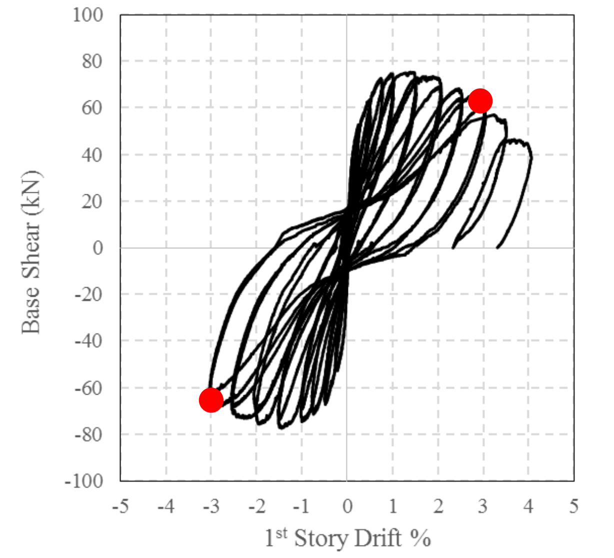


Observations and Test Results: TB2

% 3.00 Drift Ratio – 2cycle

Beams
No damage

Columns
W_{max}=8 mm
Concrete Cover Spalling

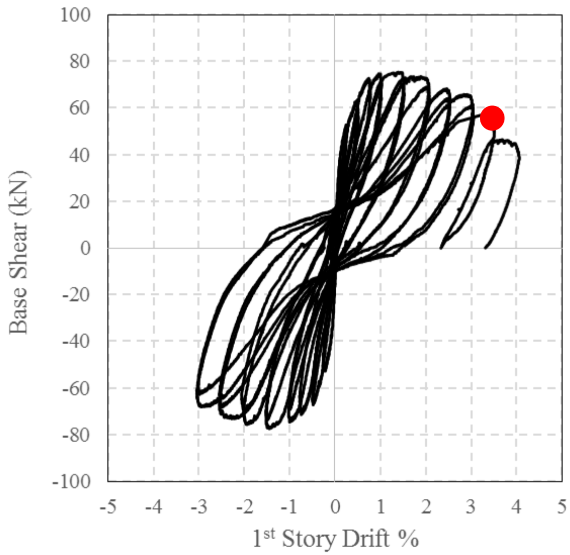


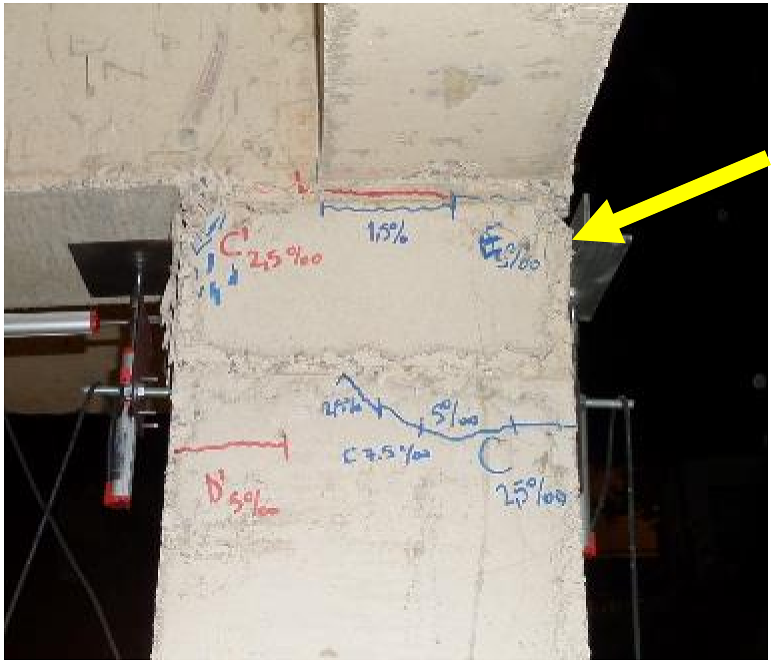
Observations and Test Results: TB2

% 3.50 Drift Ratio – Monotonic

Beams
No damage

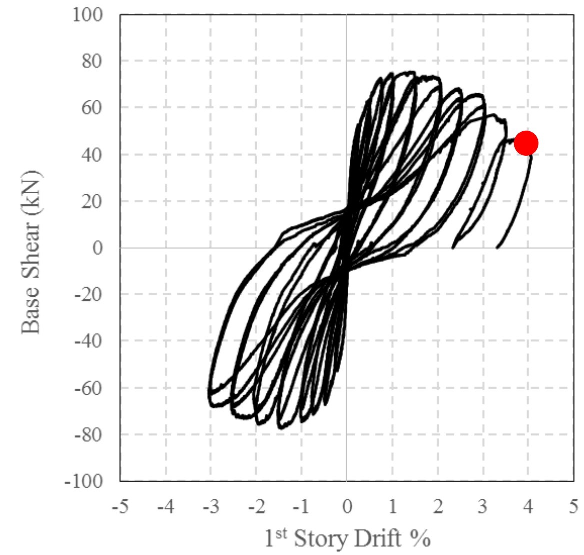
Columns
W_{max}=10 mm
Concrete Cover Spalling





Observations and Test Results: TB2

% 4.00 Drift Ratio – Monotonic



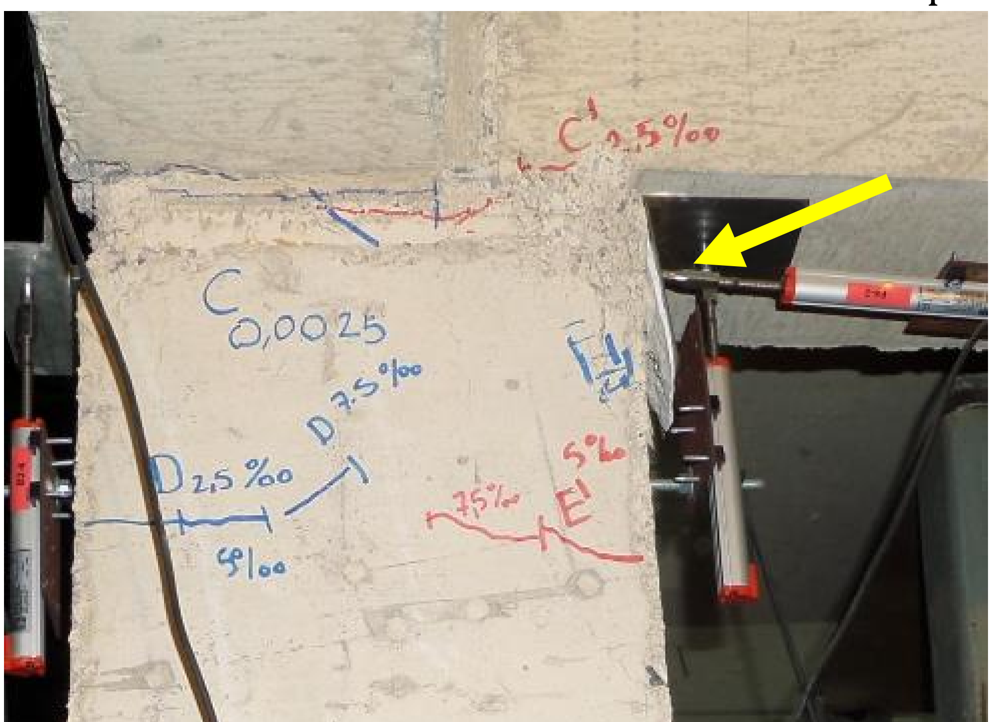
Beams

No damage

Columns

W_{max}=13 mm

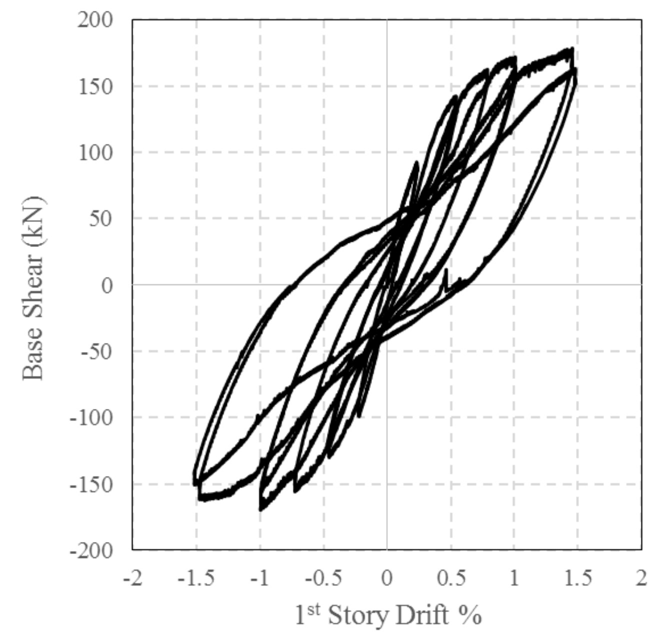
Concrete Cover Spalling



Crack Widths: TB1

Columns

Drift Ratio (%)	Crack Width at Peak (mm)	Residual Crack Width (mm)
0.25	0.1	0
0.50	0.5	0.1
0.75	2.0	0.6
1.50	3.0	1.5



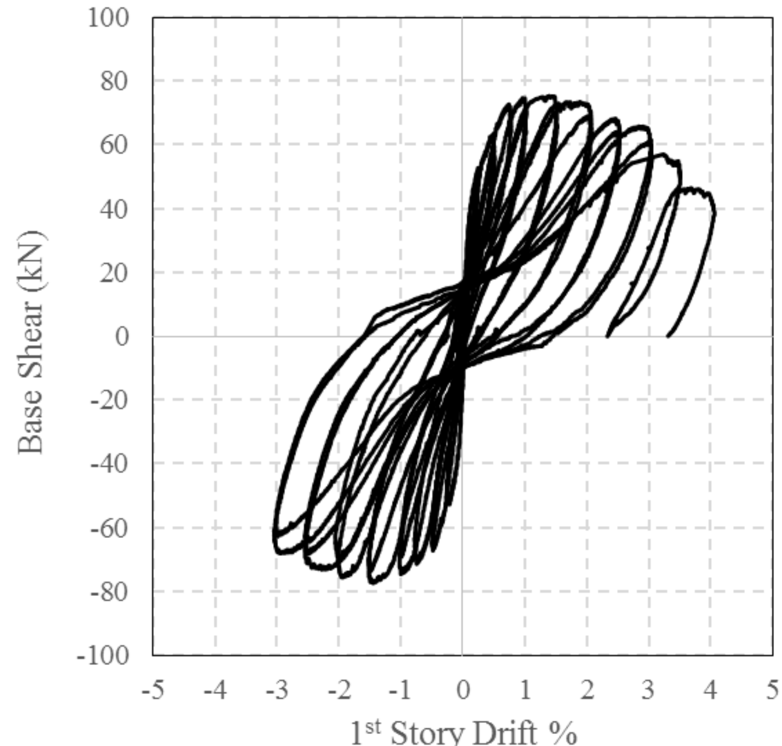
Beams

Drift Ratio (%)	Crack Width at Peak (mm)	Residual Crack Width (mm)
0.25	0.2	0.1
0.50	2.2	0.6
0.75	1.8	1.2
1.00	6.0	5.0
1.50	9.0	6.0

Crack Widths: TB2

Columns

Drift Ratio (%)	Crack Width at Peak (mm)	Residual Crack Width (mm)
0.25	0.1	0
0.50	0.3	0.1
0.75	1.4	0.3
1.00	1.7	0.4
1.50	3.5	0.6
2.00	4.5	2.0
2.50	7.0	3.0
3.00	8.0	5.0
3.50	10.0	6.0
4.00	13.0	8.0



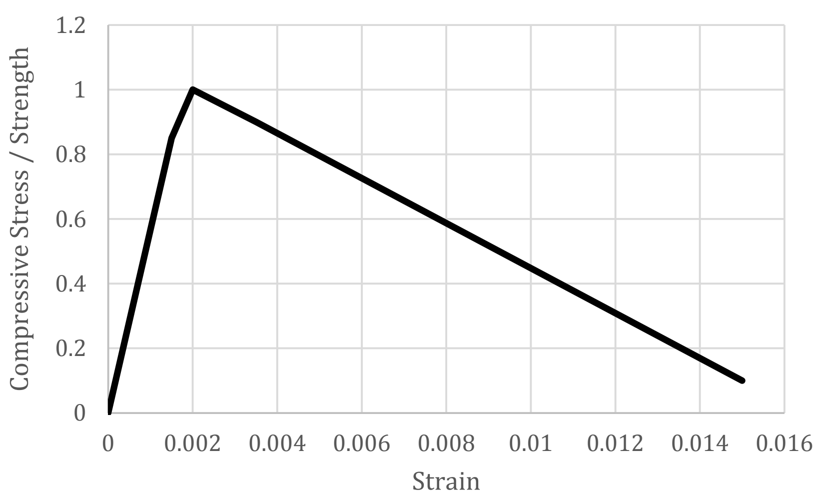
Concrete crushing initiated

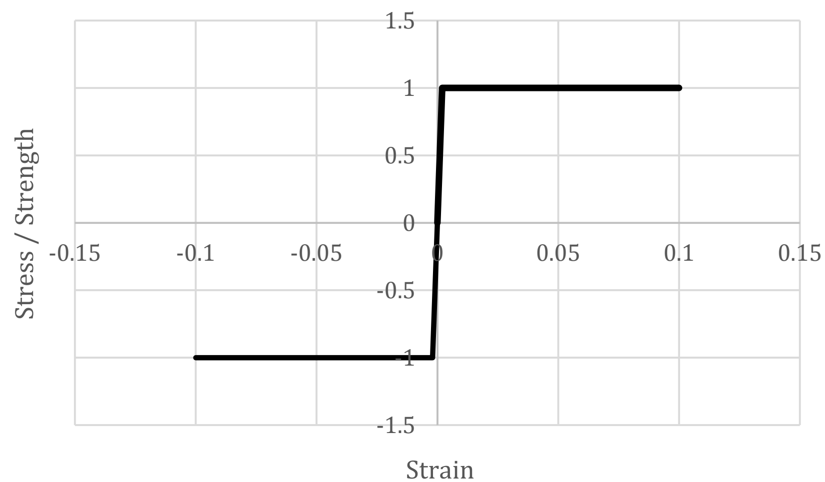
Cover spalling initiated

Predictions and Comparisons

To compare test results with analytical results
Pushover analyses were performed for TB1 and TB2.

During the analyses;
Nonlinear behavior of columns were modelled with fiber hinges considering the following material behavior

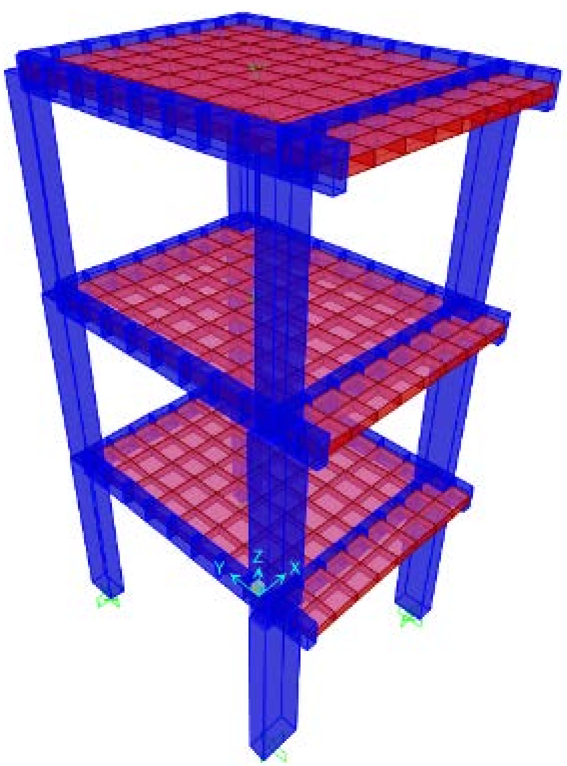




Nonlinear behavior of beams were modelled with moment-plastic rotation hinges.

Predictions and Comparisons

Analyses were made with SAP2000.

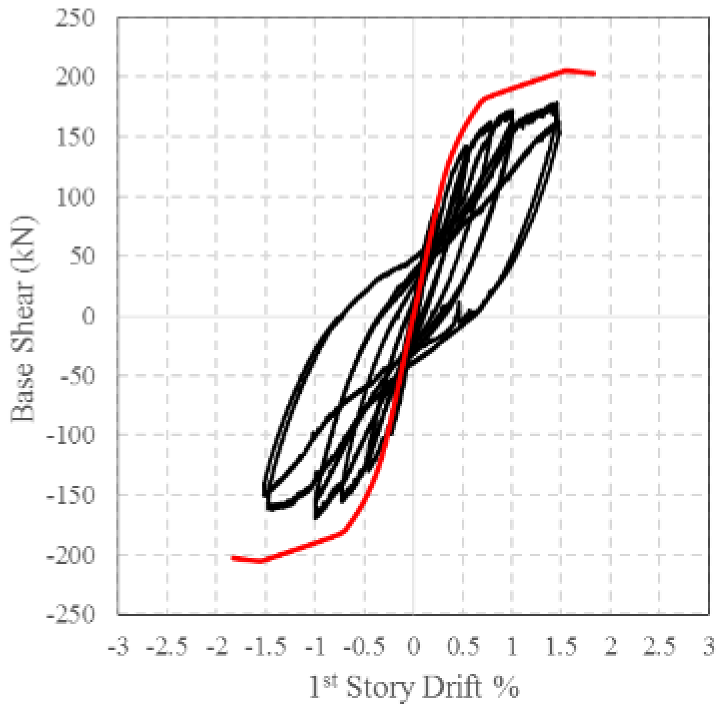


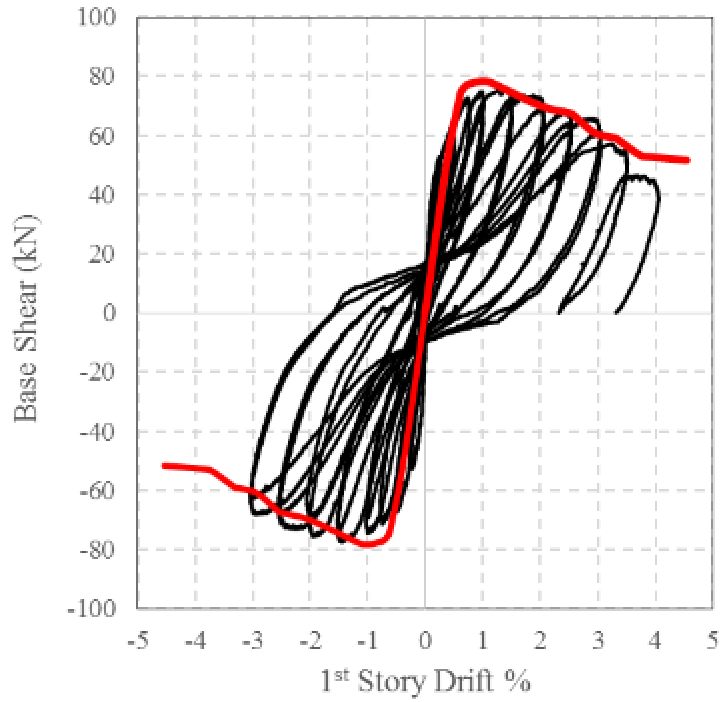
TB1

TB2

Predictions and Comparisons

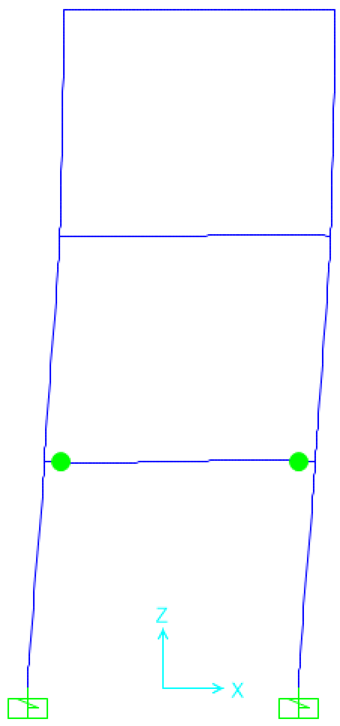
Pushover Analyses of Buildings



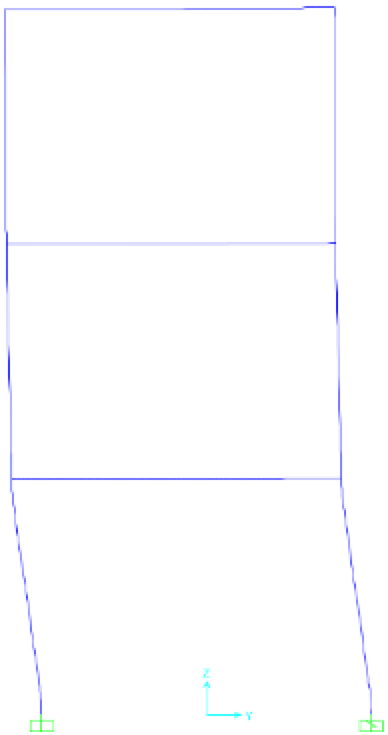


Predictions and Comparisons

Failure Mechanisms



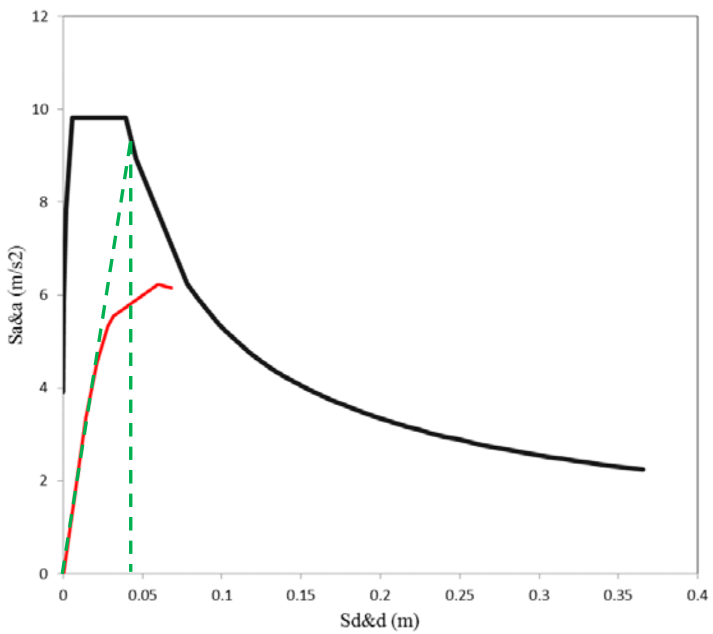
TB1- Firstly, beams reach their capacity.



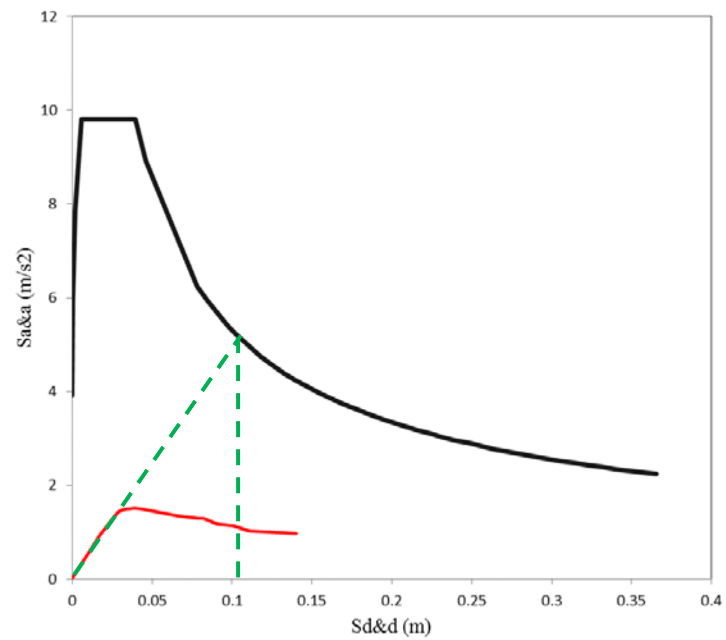
TB2- Firstly, columns reach their capacity.

Predictions and Comparisons

Displacement Demands of Test Buildings (Ao=0.4, Z2 Soil Class)



TB1 –
Drift Demand from 1st Story is 1.1%



TB2 –
Drift Demand from 1st Story is 3.3%

Predictions and Comparisons

Turkish Seismic Design Code 2007 Section Damage Limits

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

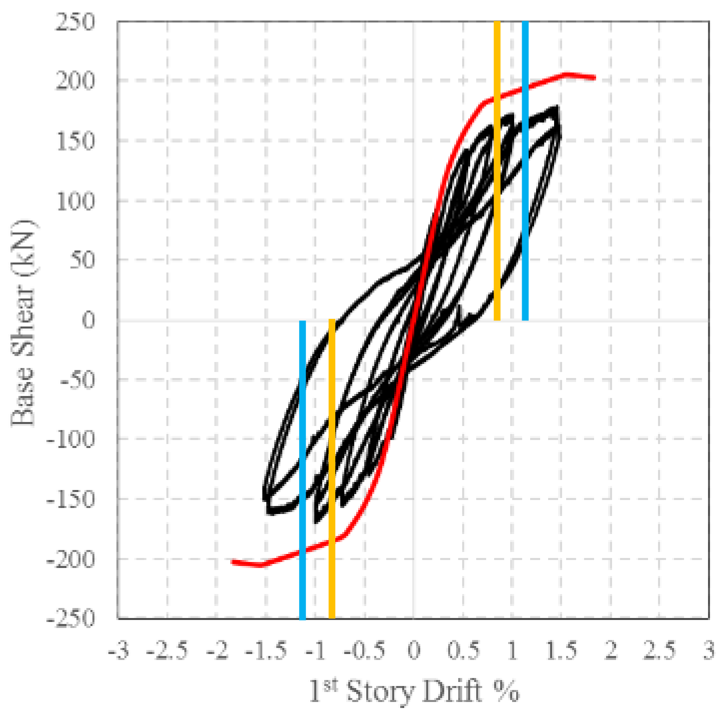
Predictions and Comparisons

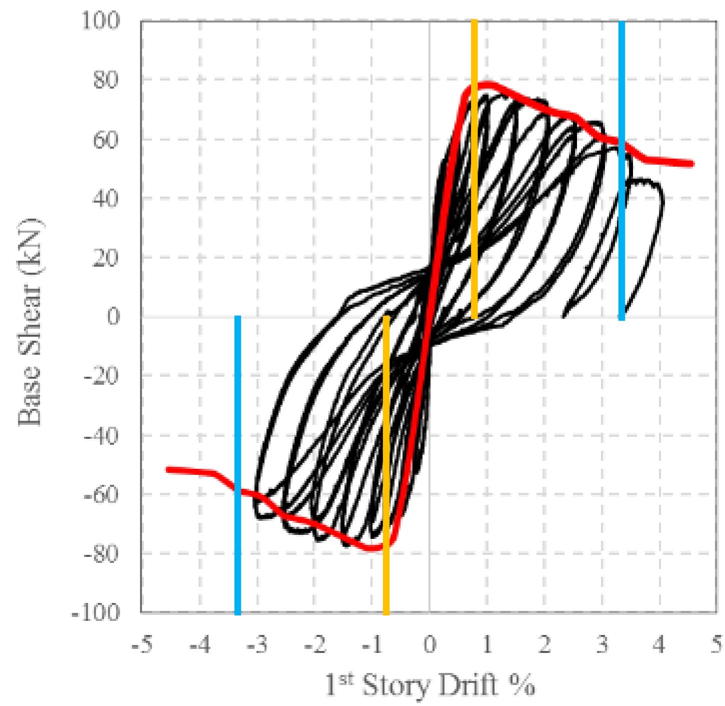
Demand of Earthquake (Ao=0.4, Z2 Soil Class)

Collapse Limit According to Turkish Seismic Code 2007

Analytical Response

Building Response





Predictions and Comparisons

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]: Plastic Rotations Angle (radians)		Residual Strength Ratio	Acceptance Criteria[a]: 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 ≤ $d/3$ 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.

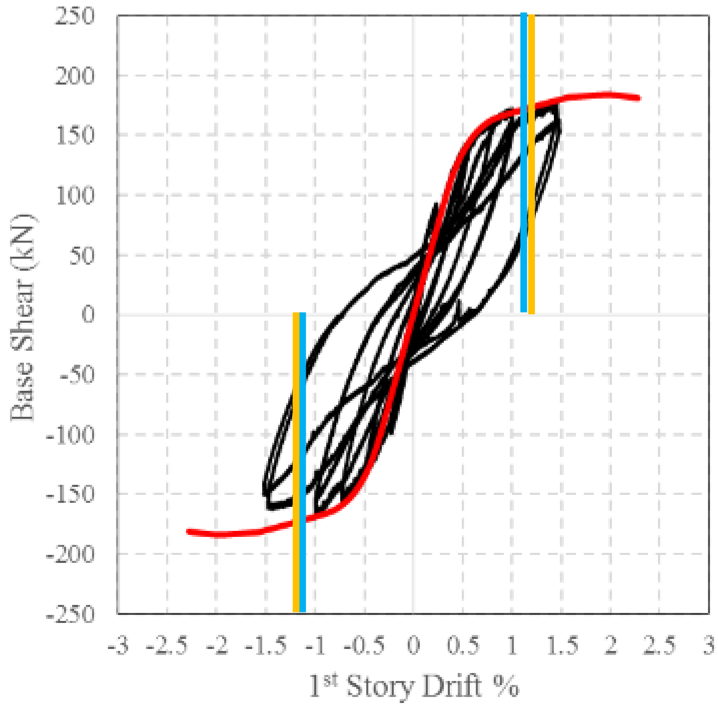
Predictions and Comparisons

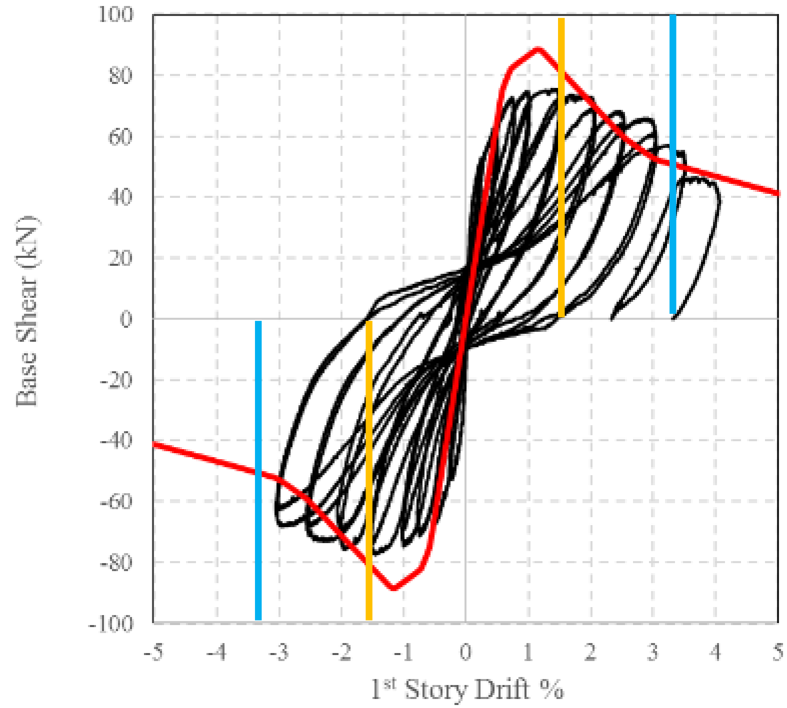
Demand of Earthquake (Ao=0.4, Z2 Soil Class)

Collapse Limit According to ASCE 41-13

Analytical Response

Building Response





Conclusions

- Procedure followed for full-scale site testing of two typical sub-standard buildings summarized.

- Damage evolution of strong column-weak beam (TB1) and weak column-strong beam (TB2) type buildings differentiated.
  - TB1: Damage first occurred at beam support regions then column lower end regions failed
  - TB2: Damage concentrated only at column end regions, rocking like behavior for columns

- Should be careful for TB2 type buildings during post-EQ damage assessment

- TSDC (2007) is more conservative then ASCE41-13 particularly in the case of members confined with improper stirrup details