

THE TURKISH SEISMIC REHABILITATION CODE

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SUMMARY

Chapter 7 of the 2006 Turkish Seismic Code entitled “Assessment and Strengthening of Existing Buildings” sets standards for assessment and rehabilitation of existing buildings. The existing material strengths are based on statistical evaluation of field data. Linear elastic and nonlinear static procedures are proposed for structural evaluation. A performance-based evaluation methodology is used under three levels of earthquake ground motion intensities with different return periods. The performance acceptance criteria are based on demand to capacity ratios at critical sections for the linear procedures, and material strains for the nonlinear procedures. Member performance limits are described for three damage levels, considering the anticipated failure mode and ductility capacity of each member. Structural performance is then calculated by accounting for the distribution of member damages over the building. Retrofit techniques are proposed also for reinforced concrete structures. They consist of detailing requirements for concentric and eccentric added shear walls, column and beam jacketing and strengthening of masonry infill walls by adding new material layers. Fundamental differences in the Turkish Code compared to Part 3 of Eurocode 8 are, less stringent requirements for linear elastic procedures, and the assembly of member performances for obtaining a global system performance level.

1. INTRODUCTION

In association with recent urban risk mitigation efforts in Turkey, a seismic rehabilitation code has been developed during 2003-2005 by an appointed committee. This code, entitled “Assessment and Strengthening of Existing Buildings” is appended to the Turkish Seismic Code as its Chapter 7 [Turkish, 2006], and has been effective legally by 2006.

Strong earthquakes during the last two decades in Turkey revealed that the existing building stocks in urban regions are significantly vulnerable to seismic hazard. A large number of buildings including schools, hospitals commercial and residential buildings built in regions of high seismicity require urgent retrofitting before the next strong earthquake in their region. Although this is a world scale problem for all earthquake prone countries, high seismicity rate in Turkey and high rate of urbanization, naturally accompanied with inadequate construction supervision which resulted in lower construction quality, are responsible for the accumulation of higher seismic risks. The necessity of a seismic rehabilitation code for Turkey is obvious, and need for an effective rehabilitation code leading to economically feasible solutions for huge vulnerable building stocks is clear.

FEMA-356 [ASCE, 2000] and Eurocode 8, Part 3 [European, 2003] are two prominent rehabilitation standards preceding the Turkish Rehabilitation Code. Both of these standards follow similar approaches in adopting performance based assessment methodologies for existing buildings. The differences in the Turkish Code compared to these two standards are less stringent requirements for linear elastic procedures, and the assembly of member performances for obtaining a global system performance level. Moreover, several retrofitting techniques are described for reinforced concrete buildings, both at the component and system level.

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2. COLLECTION OF FIELD DATA

Field data includes member dimensions, details, structural system topology and material properties. This data is collected by field measurements, field testing and by inspecting the as-built drawings, when available, and employed in determining the member capacities. The scope of data collection also includes the identification of structural system and foundation type, existing damage and past alterations. Finally, the conformance of field data with the as-built drawings has to be checked.

2.1 Knowledge Levels

The information collected from the existing buildings is classified with respect to the scope of data and the type of building system. These levels are “*limited*”, “*moderate*” and “*comprehensive*”. They are separated with respect to the presence of in-situ drawings and the extent of data collected from the building by field measurements. The description of knowledge levels are presented in the sections below for concrete buildings.

2.1.1 *Limited knowledge level*

As-built structural drawings are not present. Structural plans are determined by field studies. The location of all structural members and partition walls are marked on the story plans. Foundation system is identified by excavating inspection pits in sufficient number. The collected topological information has to be adequate for constructing the analytical building model. It is assumed that the reinforcement details in concrete members conform to the design code enforced at the year of construction. In order to confirm this assumption, reinforcement has to be inspected visually in 10% of columns and 5% of beams in each story by removing the cover concrete in two adjacent sides at appropriate locations. Then the ratios of existing reinforcement to the minimum reinforcement in the code, both longitudinal and transverse, are calculated for beams and columns separately, and they are called the “*reinforcement realization ratios*”. These ratios are applied to all members.

The material strengths that have to be employed in calculating member capacities are called the “*existing material strengths*” in the Code. A minimum of two concrete core samples is required from columns and walls at each story. The lowest concrete strength obtained from the tests is defined as the existing concrete strength. The existing steel strength is based on the nominal characteristic yield strength of the visually observed reinforcement.

2.1.2 *Moderate knowledge level*

Essentially the same as the limited knowledge level, however reinforcement is inspected in 20% of columns and 10% of beams in each story. Moreover, a minimum of three concrete core samples are taken from the columns and walls, where the minimum total number is nine. (Mean – one standard deviation) of the test strength values is defined as the existing concrete strength.

2.1.3 *Comprehensive knowledge level*

As-built structural drawings have to be available. They are verified by field measurements, and the differences are corrected. Other field measurements related to building geometry, reinforcement details and material properties are similar to those defined for the moderate knowledge level. However existing steel strength is verified by testing three steel specimens taken from the building. Stress-strain behaviour of reinforcing steel is also determined by testing these specimens.

2.2 Knowledge Factors

Knowledge factors are applied to the calculated member capacities. They are 0.75 for the limited, 0.90 for the moderate, and 1.0 for the comprehensive knowledge levels, respectively.

3. DAMAGE LIMITS AND DAMAGE STATES IN STRUCTURAL MEMBERS

Structural members are classified as “ductile” and “brittle” with respect to their mode of failure in determining the damage limits. For ductile members, three damage limits are defined at the cross section level. These are

minimum damage limit (MN), safety limit (SF) and collapse limit (CL) as indicated on Figure 1. The corresponding damage states are also given on the same figure. MN defines the onset of significant post-elastic behaviour at a critical cross section. Brittle members are not permitted to exceed this limit. A member damage state is determined by its critical cross section with the most severe damage state.

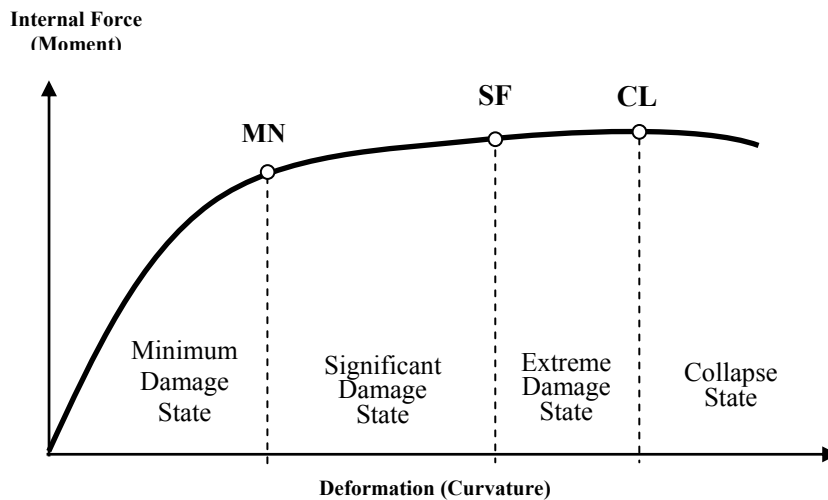


Figure 1: Damage limits and damage states in a ductile member

4. SEISMIC ANALYSIS PROCEDURES

The objective of seismic analysis is to determine the seismic performance of an existing building. Linear elastic or nonlinear (pushover) procedures can be employed for analysis where the seismic intensity is defined by linear elastic response spectra representing three different intensity levels, with respective exceeding probabilities of 50, 10 and 2 % in 50 years.

4.1 Linear Elastic Procedures

Equivalent static lateral force analysis and modal response spectrum analysis can be employed for performance assessment. Equivalent static lateral force analysis is limited to 8 story buildings with total height not exceeding 25 m, and not possessing torsion irregularity. For buildings with more than two stories, 85% of the total mass is considered in calculating the base shear force. Modal response spectrum analysis can be applied to all buildings without any restrictions. The signs of internal member forces and capacities under an earthquake excitation direction are taken as the signs consistent with the dominant mode shape in this direction.

4.1.1 Performance assessment of reinforced concrete members

Damage limits are expressed in terms of the *demand/capacity ratios (member r factors)* for ductile members at their critical cross sections. Ductile concrete frame members are controlled by the flexural failure mode where shear capacity exceeds the shear force developed when the member reaches its flexural capacity. A shear wall is accepted as ductile if it has confined end regions, $H/l > 2$ and horizontal reinforcement ratio exceeds 0.0025. The demand/capacity ratio for beams, columns and shear walls is the ratio of earthquake moment to the residual capacity moment at the critical section, where the residual capacity moment is the difference between the flexural capacity and the dead load moment. The demand/capacity ratio of reinforced (strengthened) masonry infill walls is the ratio of shear force under earthquake loads to the shear capacity.

The calculated member r factors for beams, columns, shear walls and reinforced infill walls are compared with the r limits given in Tables 1-4 to determine the member damage states in accordance with Figure 1. Reinforced masonry infill walls should also satisfy the interstorey drift limits given in Table 4. Member r factors are analogous to section ductility ratios when equal displacement rule prevails.

Table 1: Demand/capacity ratios for reinforced concrete beams (*r*)

Ductile Beams			Damage Limit		
$\frac{\rho - \rho'}{\rho_b}$	Confinement	$\frac{V}{b_w d f_{ct}}$ ⁽¹⁾	MN	SF	CL
≤ 0.0	Conforming	≤ 0.65	3	7	10
≤ 0.0	Conforming	≥ 1.30	2.5	5	8
≥ 0.5	Conforming	≤ 0.65	3	5	7
≥ 0.5	Conforming	≥ 1.30	2.5	4	5
≤ 0.0	Non conforming	≤ 0.65	2.5	4	6
≤ 0.0	Non conforming	≥ 1.30	2	3	5
≥ 0.5	Non conforming	≤ 0.65	2.5	4	6
≥ 0.5	Non conforming	≥ 1.30	1.5	2.5	4
Brittle Beams			1	1	1

(1) Units are N and mm.

Table 2: Demand/capacity ratios for reinforced concrete columns (*r*)

Ductile Columns			Damage Limit		
$\frac{N}{A_c f_c}$	Confinement	$\frac{V}{b_w d f_{ct}}$ ⁽¹⁾	MN	SF	CL
≤ 0.1	Conforming	≤ 0.65	3	6	8
≤ 0.1	Conforming	≥ 1.30	2.5	5	6
≥ 0.4	Conforming	≤ 0.65	2	4	6
≥ 0.4	Conforming	≥ 1.30	2	3	5
≤ 0.1	Non conforming	≤ 0.65	2	3.5	5
≤ 0.1	Non conforming	≥ 1.30	1.5	2.5	3.5
≥ 0.4	Non conforming	≤ 0.65	1.5	2	3
≥ 0.4	Non conforming	≥ 1.30	1	1.5	2
Brittle Columns			1	1	1

(1) Units are N and mm.

Table 3: Demand/capacity ratios for reinforced concrete shear walls (*r*)

Ductile Walls	Damage Limit		
Confinement	MN	SF	CL
onforming	3	6	8
Non conforming	2	4	6
Brittle Walls	1	1	1

Table 4: Demand/capacity ratios and interstory drift ratios for reinforced masonry infill walls (*r*)

(ℓ_{wall} / h_{wall}) range 0.5 - 2.0	Damage Limit		
	MN	SF	CL
Demand/capacity ratio (<i>r</i>)	1	2	-
Interstory Drift Ratio	0.0015	0.0035	-

In reinforced concrete beam-column joints, joint shear capacity should be larger than the shear at the horizontal cross section transmitted from the column above and the longitudinal reinforcement of beams spanning into the joint when they reach their yield capacity. Otherwise all jointing members are accepted as brittle. All capacities are calculated by employing the existing material strengths.

Axial forces in reinforced concrete columns and shear walls are calculated in accordance with the flexural capacities of connecting beams and the axial capacities of connecting struts that represent infill walls. Axial force transmitted from a beam into a column axis is limited with the shear at the connecting beam end, which corresponds to the shear force when the beam reaches its flexural capacity. This limit analysis is required since earthquake loads are not reduced by the system response reduction (R) factors in linear elastic analysis.

4.2 Nonlinear Procedures

Incremental equivalent static lateral force analysis (single mode pushover) and incremental modal response spectrum analysis [Aydinoğlu, 2003] or multi-mode pushover analysis [Chopra, 2002] can be employed for performance assessment. Incremental equivalent static lateral force analysis is limited to 8 story buildings with total height not exceeding 25 m, and not possessing torsion irregularity.

Nonlinear flexural behaviour in frame members are confined to plastic hinges, where the plastic hinge length L_p is assumed as half of the section depth ($L_p = h/2$). Pre-yield linear behaviour of concrete sections is represented by cracked sections, which is $0.40EI_0$ for beams and varies between $(0.40-0.80)EI_0$ with the axial stress for columns. Strain hardening in the plastic range may be ignored, provided that the plastic deformation vector remains normal to the yield surface. Diagonal braces that represent reinforced masonry infill walls are modelled as elasto-plastic axial tension-compression members.

4.2.1 Incremental equivalent static lateral force (single mode pushover) analysis

The objective is to carry out nonlinear static analysis under incrementally increasing lateral forces distributed in accordance with the dominant mode shape in the earthquake excitation direction. Lateral forces are increased until the earthquake displacement demand is reached. Internal member forces and plastic deformations are calculated at the demand level. A capacity diagram is obtained from the incremental analysis which is expressed in the “base shear force - roof displacement” plane. Then the coordinates of this plane is transformed into “modal response acceleration versus modal response displacement” as shown in Figure 2 below.

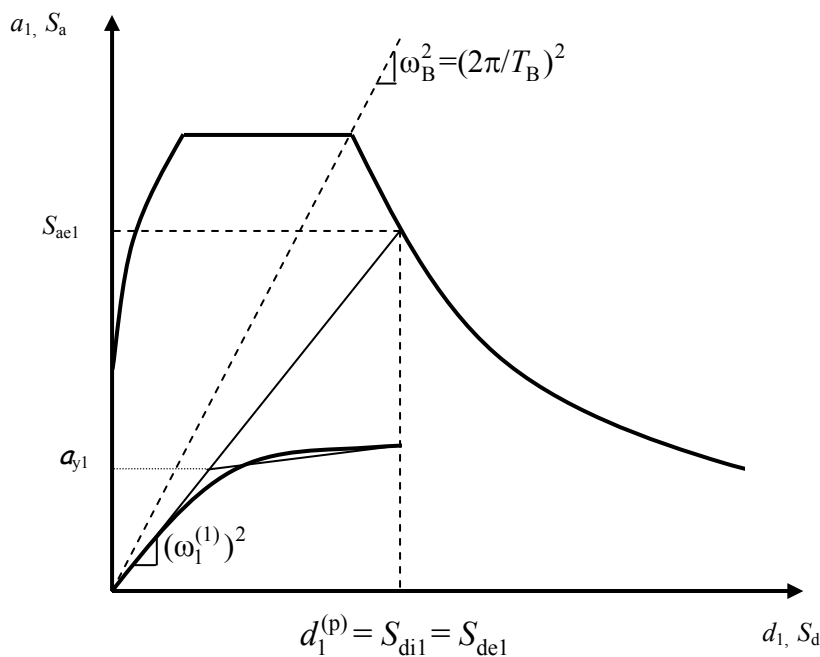


Figure 2: Capacity diagram and the displacement demand in the modal acceleration-displacement plane

The modal displacement demand d_1 is equal to the inelastic displacement demand S_{di1} , which is in turn equal to the modal linear elastic displacement demand S_{de1} when $(\omega_1^{(1)})^2 \leq \omega_B^2$ as shown in Figure 2. When $(\omega_1^{(1)})^2 > \omega_B^2$,

$$S_{di1} = C_{R1} S_{de1} \quad (1)$$

where

$$C_{R1} = \frac{1 + (R_{y1} - 1) T_B / T_1^{(1)}}{R_{y1}} \geq 1 \quad (2)$$

and

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \quad (3)$$

Building displacements, internal deformations and forces can be calculated at the modal displacement demand d_1 by appropriate transformations using the first mode properties. The plastic rotations obtained at the member plastic hinge locations are then used for calculating the plastic curvature demands at these critical sections.

$$\phi_p = \frac{\theta_p}{L_p} \quad (4)$$

4.2.2 Performance assessment of reinforced concrete members

Concrete compressive strains and steel tensile strain demands at the plastic regions are calculated from the moment-curvature diagrams at the plastic curvature level in Eq. (4). Moment-curvature diagrams of the critical sections are obtained by using appropriate stress-strain rules for concrete and steel. Finally, the calculated strain demands are compared with the damage limits given below to determine the member damage states in view of Figure 1.

Concrete and steel strain limits at the fibres of a cross section for minimum damage limit (MN)

$$(\varepsilon_{cu})_{MN} = 0.004 ; (\varepsilon_s)_{MN} = 0.010 \quad (5)$$

Concrete and steel strain limits at the fibres of a cross section for safety limit (SF)

$$(\varepsilon_{cg})_{SF} = 0.004 + 0.0095 (\rho_s / \rho_{sm}) \leq 0.0135 ; (\varepsilon_s)_{SF} = 0.040 \quad (6)$$

Concrete and steel strain limits at the fibres of a cross section for collapse limit (CL)

$$(\varepsilon_{cg})_{CL} = 0.004 + 0.013 (\rho_s / \rho_{sm}) \leq 0.018 ; (\varepsilon_s)_{CL} = 0.060 \quad (7)$$

In Equations (5-7), ε_{cu} is the concrete strain at the outer fibre, ε_{cg} is the concrete strain at the outer fibre of the confined core, ε_s is the steel strain and (ρ_s / ρ_{sm}) is the ratio of existing confinement reinforcement at the section to the confinement required by the Code. The interstorey drift ratios given in Table 4 for reinforced masonry infill walls are also valid for nonlinear procedures.

5. BUILDING EARTHQUAKE PERFORMANCE AND RETROFIT DECISIONS

Building earthquake performance level is determined after determining the member damage states, as explained above. The rules for determining building performance are given below for each performance level.

5.1 Immediate Occupancy

In any story, in the direction of the applied earthquake loads, not more than 10% of beams are in the significant damage state whereas all other structural members are in the minimum damage state. Retrofitting is not required.

5.2 Life Safety

In any story, in the direction of the applied earthquake loads, not more than 20% of beams and some columns are in the extreme damage state whereas all other structural members are in the minimum or significant damage states. However shear carried by those columns in the extreme damage state should be less than 20% of the story shear at each story. Retrofitting of the building may be required depending on the number and distribution of members in the extreme damage state.

5.3 Collapse Prevention

In any story, in the direction of the applied earthquake loads, not more than 20% of beams and some columns are in the collapse state whereas all other structural members are in the minimum, significant or extreme damage states. However shear carried by those columns in the collapse state should be less than 20% of the story shear at each story. Furthermore, such columns should not lead to a stability loss. Occupancy of the building should not be permitted. Decision on retrofitting or demolishing of the building depends on the feasibility of retrofitting.

5.4 Collapse

If the building fails to satisfy any of the above performance levels, it is accepted as in the collapse state. Occupancy of the building should not be permitted. The building should be retrofitted, however its retrofit may not be economically feasible.

5.5 Limitation of Interstory Drifts

In any story, in the direction of the applied earthquake loads, interstory drift ratios should satisfy the limits given in Table 5 for each performance level. $\delta_{b, \max}$ is the maximum relative drift in vertical members in the i 'th story.

Table 5: Interstory drift limits

Interstory Drift Ratio	Performance Level		
	Immediate Occupancy	Life Safety	Collapse Prevention
$\delta_{b, \max}/h_i$	0.008	0.02	0.03

6. TARGET PERFORMANCE LEVELS FOR BUILDINGS

The reference design spectrum in the Code has 10% probability of exceeding in 50 years. Based on Turkish strong motion data, it is estimated that the spectral ordinates for 50% probability of exceeding in 50 years are half of the reference spectrum whereas the ordinates for 2% probability of exceeding in 50 years are 1.5 times that of the reference spectrum. Accordingly, the target performance levels of retrofitted buildings are summarized in Table 6.

Table 6: Target performance levels for buildings under different earthquake intensities

Type of Building Use	Probability of Exceeding		
	50% in 50 years	10% in 50 years	2% in 50 years
Emergency facilities (hospitals, etc.)	-	Immed. Occup.	Life Safety
Buildings with long duration occupancy (schools, etc.)	Immed. Occup.	-	Life Safety
Theatres, concert halls, sport arenas, congress centres	-	Life Safety	Collapse Prvn.
Buildings containing toxic materials	-	Immed. Occup.	Collapse Prvn.
Others (residential, commercial, etc.)	-	Life Safety	-

7. RETROFITTING OF CONCRETE BUILDINGS

The scope of building retrofitting is correction of seismic weaknesses, adding new members to increase earthquake safety, improvement of the seismic behaviour of existing members, and reduction of mass. The design details of new members should conform to the Code requirements for new buildings. Member and system retrofitting techniques given in this section represent frequently used implementations. Those implementations that are not covered here are acceptable as long as they conform to the approaches, requirements and criteria stated herein.

7.1 Jacketing of Columns

Jacketing is applied to increase the ductility, shear and axial strength, to reduce the shear and compressive stresses, and correction of lap splice deficiencies in columns. Jacketing cannot increase the flexural capacity of columns.

Concrete jacketing can be applied with a minimum cover thickness of 100 mm. Jacket starts 100 mm above the bottom slab, and ends at a 100 mm distance from the top slab or girder. Gross cross section area of the jacketed column and new concrete strength is considered in calculating shear and axial capacities, however the calculated capacities are reduced by 0.9.

Steel jacketing is applied by longitudinal angle profiles at the four corners of prismatic columns. They are connected tightly to the column faces with transverse steel plates welded at four sides, spaced uniformly over the column height. Longitudinal corner profiles should be installed adequately to transfer axial forces between adjacent slabs. Special anchorage requirements are given in the Code for the reduction of lap splice deficiencies by steel plates fixed to the facing column sides.

Fibre reinforced polymer (FRP) wrapping may be used to confine the deficient columns to increase their ductility, shear and axial force capacities and to increase the strength of deficient splices, as shown in Figure 3. Equations are provided to calculate the increased capacities.

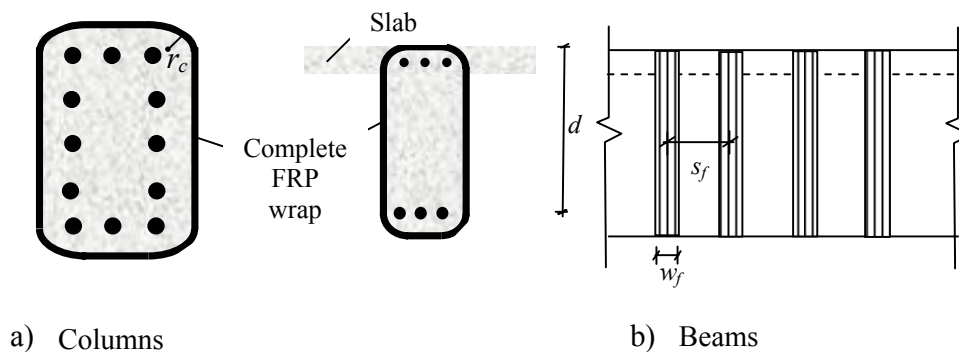


Figure 3: FRP wrapping in columns and beams

FRP wrapping may also be applied to beams in order to increase their shear strength and ductility at critical (potential plastic hinge) regions, as shown in Figure 3.

7.2 Increasing the Flexural Capacity of Columns

Column cross section should be increased in order to increase its flexural capacity, which in turn increases its shear and axial force capacities. The difference between this type of retrofitting and concrete jacketing is the continuity requirement of new longitudinal reinforcement through the slabs between adjacent floors, and its confinement. Flexural capacity of a column may also be increased by increasing its cross section asymmetrically on one side. However the core regions of the existing column and the new section have to be integrated in this case by proper confinement.

7.3 Strengthening of Existing Unreinforced Masonry Infill Walls

The existing unreinforced masonry infill walls in concrete buildings with three stories or less above the basement can be strengthened by applying reinforcing material layers on their faces to increase their stiffness and shear strength. Such infill walls should be continuous vertically along all stories. The reinforcing material layers can

be composed of a mortar layer reinforced with steel mesh, diagonally plastered layers of FRP, or thin precast concrete plates connected and plastered to form a strong layer on the wall face. All methods are subjected to special restrictions. They are represented in the analytical building models by diagonal struts which are effective in tension and compression. All modelling parameters of the struts are defined in the Code, and detailing requirements for implementation are given. The details of strengthening with wire mesh are shown in Figure 4. The experimental lateral load-drift relationship for a test specimen strengthened with the details given in Figure 4 are presented in Figure 5, and compared with the stiffness and strength values predicted by the Code equations (Acun and Sucuoğlu, 2006). The reinforcement ratio of the strengthened wall specimen was 0.05 percent.

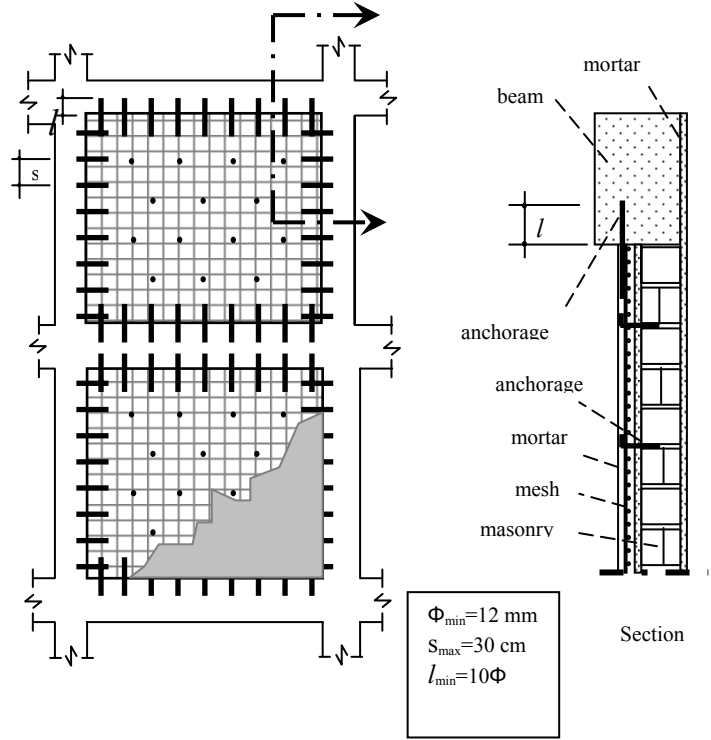


Figure 4: Unreinforced masonry infill wall strengthened with mesh-reinforced mortar layer

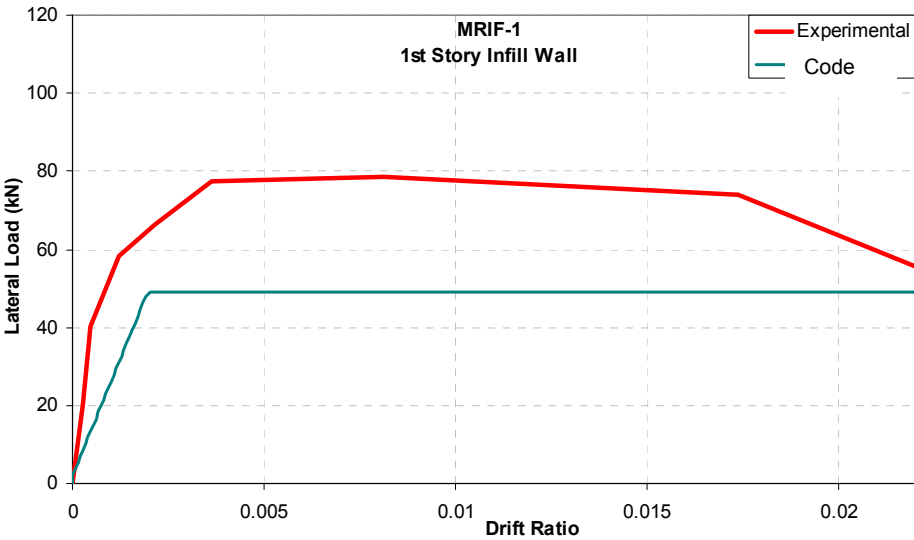


Figure 5: Comparison of Code prediction equations for a reinforced masonry infill wall specimen with the test results

7.4 Adding Concentric Infill Shear Walls

New infill walls are arranged within an existing frame axis and made continuous from foundation to their top elevation. The integrity of the existing frame and the added concrete infill wall is provided by sufficient number of anchorages detailed according to the Code. Confined end regions are formed for the integral shear wall either by using or modifying the existing peripheral columns. An adequate foundation has to be provided under the added wall. The foundation beam may be extended to include the adjacent column axes in order to take the advantage of their vertical forces for reducing uplift.

7.5 Adding Eccentric Shear Walls to Exterior Frames

Single or coupled shear walls may be connected eccentrically to the exterior frames. They are designed to satisfy the relevant code requirements for shear walls in new buildings. The integrity of the existing frame and the added concrete eccentric wall is provided by sufficient number of anchorages detailed according to the Code. An adequate foundation has to be provided under the added coupled walls. The foundation beam may be extended to include the adjacent column axes in order to take the advantage of their vertical forces for reducing uplift, or anchored to the ground for preventing uplift.

7.6 Post-Earthquake Damage Assessment

Structural safety of a building damaged after an earthquake cannot be assessed by employing the procedures stated in this Code. This statement is added to the Code because damage assessment after an earthquake has to be completed in a short time by visual surveys for tagging the buildings. The Code can be used later after the termination of the emergency phase for making retrofit decisions.

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