PERFORMANCE EVALUATION OF EXISTING MEDIUM RISE REINFORCED CONCRETE BUILDINGS ACCORDING TO 2006 TURKISH SEISMIC REHABILITATION CODE

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY

BY

ZEYNEP DÜZCE

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

NOVEMBER 2006

Approval of the Graduate School of Natural and Applied Sciences

Prof. Dr. Canan Özgen
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Prof. Dr. Erdal Çokca
Head of Department

This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

Prof. Dr. Haluk Sucuoğlu
Supervisor

Examining Committee Members

Prof. Dr. Güney Özcebe	(METU, CE)	
Prof Dr. Haluk Sucuoğlu	(METU, CE)	
Assoc. Prof. Dr. Ahmet Yakut	(METU, CE)	
Asst. Prof. Dr. Altuğ Erberik	(METU, CE)	
M.S Joseph Kubin	(PROTA Yazılım Bilişim ve Müh. Hizm.Ltd)	

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last name: Zeynep Düzce

Signature

ABSTRACT

PERFORMANCE EVALUATION OF EXISTING MEDIUM RISE REINFORCED CONCRETE BUILDINGS ACCORDING TO 2006 TURKISH SEISMIC REHABILITATION CODE

Düzce, Zeynep
M.Sc., Department of Civil Engineering
Supervisor: Prof. Dr. Haluk Sucuoğlu

November 2006, 147 pages

Linear elastic and nonlinear analysis procedures of 2006 Turkish Seismic Rehabilitation Code are applied to medium rise reinforced concrete buildings. In this study, four storey residential buildings are designed according to the 1998 and 1975 Turkish Seismic Design Codes, and the analysis procedures are verified on these case studies. In addition to these buildings, the analysis procedures are tested on an existing school building before and after retrofitting.

The assessment procedures employed in the 2006 Turkish Seismic Rehabilitation Code are based on linear elastic analysis (equivalent lateral load method, mode superposition method); non-linear analysis (pushover analysis with equivalent lateral load method and mode superposition method) and non-linear time history analysis. In this study, linear elastic analysis with equivalent lateral loads and non-linear static analysis (pushover analysis) with equivalent lateral loads are investigated comparatively.

SAP2000 software is used for pushover analysis; however the plastic rotation values obtained from SAP2000 are not used directly but defined according to the code procedures. Post-elastic rotations at yielding sections are transferred to Excel and the corresponding strains are calculated from these rotations by Excel Macro. These strains are compared with strain limits described in the 2006 Turkish Seismic Rehabilitation Code to obtain the member performances.

In the linear elastic procedure, structural analysis is performed also by SAP2000 to obtain the demand values, whereas the capacity values are calculated by another Excel Macro. With these demand and capacity values, corresponding demand to capacity ratios are calculated and compared with demand to capacity ratio limits described in 2006 Turkish Seismic Rehabilitation Code to obtain the member performances.

Global performances of the buildings are estimated from the member performances and from the inter-storey drifts for both two methods. The results are compared to each other, and critically evaluated.

Keywords: 2006 Turkish Seismic Rehabilitation Code, linear elastic procedure, nonlinear static procedure, member performances, global performance of buildings.

ÖZ

ORTA YÜKSEKLİKTEKİ MEVCUT BETONARME BİNALARIN 2006 TÜRK DEPREM YÖNETMELİĞİNE GÖRE PERFORMANS DEĞERLENDİRMESİ

Düzce, Zeynep
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi: Prof. Dr. Haluk Sucuoğlu

Kasım 2006, 147 sayfa

Bu tezde, 2006 Türk Deprem Yönetmeliği'nin doğrusal ve doğrusal olmayan analiz metotları orta yükseklikteki betonarme binalara uygulanmıştır. Bu analizler için dört katlı bir konut binası 1998 ve 1975 Türk Deprem Yönetmeliklerine göre tasarlanmış ve analiz metotları bu binalar üzerinde karşılaştırılmıştır. Bu binalara ilave olarak, analiz metotları mevcut bir okul binasının güçlendirilmemiş ve güçlendirilmiş hali üzerinde de değerlendirilmiştir.

2006 Türk Deprem Yönetmeliği'nde yer alan analiz metotları doğrusal elastik analiz (eşdeğer deprem yüklemesi, mod birleştirme yöntemi); doğrusal elastik olmayan analiz (eşdeğer deprem yüklemesi ve mod birleştirme yöntemi ile artımsal itme analizi) ve zaman tanım alanında doğrusal olmayan hesap yöntemleridir. Bu çalışmada eşdeğer deprem yüklemesiyle doğrusal elastik analiz ve eşdeğer deprem yüklemesiyle doğrusal elastik olmayan analiz (artımsal itme analizi) incelenmiştir.

Artımsal itme analizleri için SAP2000 programı kullanılmıştır, ancak programdan elde edilen plastik dönme değerleri doğrudan kullanılmamış, yönetmeliğe göre tanımlanarak programa girilmiştir. Analizden sonra plastik dönme sonuçları Excel’e aktarılmış ve plastik dönmelerden birim şekil değiştirmeler hesaplanmıştır. Bu birim şekil değiştirmeler 2006 Türk Deprem Yönetmeliği’ndeki birim şekil değiştirmelerle kıyaslanarak eleman performansları elde edilmiştir. Bina genel performansı eleman performanslarından ve deplasman performanslarından elde edilir.

Elastik analiz yönteminde, yapısal analiz yine SAP2000 programıyla yapılmıştır ve etkiler elde edilmiştir. Kapasiteler ise başka bir Excel programı ile hesaplanmıştır. Bu etki ve kapasitelerden etki-kapasite oranları hesaplanarak 2006 Türk Deprem Yönetmeliği’ndeki sınır değerleri ile kıyaslanmıştır ve eleman performansları elde edilmiştir.

Bina genel performansı, her iki yöntem için eleman performanslarından ve kat ötelenmelerinden elde edilir. Elde edilen sonuçlar birbirleriyle kıyaslanarak incelenmiştir.

Anahtar Kelimeler: 2006 Türk Deprem Yönetmeliği, doğrusal elastik analiz, artımsal itme analizi, eleman performansları, binaların genel performansı.

To my family...

ACKNOWLEDGEMENTS

I would like to express great appreciation to my supervisor Prof. Dr. Haluk Sucuoğlu for his support, advice and patience during this study.

Special thanks go to Celaletdin Uygar for his guidance, help, support and especially for being everything in my life.

Also, I would like to thank M.Selim Günay, Can Değirmenci and Ali Şengöz for their support and advises.

Finally, my parents deserve the greatest thanks for their unlimited understanding and love.

TABLE OF CONTENTS

PLAGIARISM iii

ABSTRACT iv

ÖZ…… vi

ACKNOWLEDGEMENTS ix

TABLE OF CONTENTS x

LIST OF TABLES xiii

LIST OF FIGURES xv

CHAPTERS

1. INTRODUCTION 1

1.1 Aim of the Study 1

1.2 Review of Past Studies 2

2. DESCRIPTION OF ASSESSMENT METHODS FOR EXISTING BUILDINGS IN 2006 TURKISH SEISMIC REHABILITATION CODE 17

2.1 Linear Elastic Analysis 17

2.1.1 Modeling for Linear Elastic Analysis 17

2.1.2 Calculating the Member Capacities 18

2.1.3 Performance Assessment of Members 22

2.2 Nonlinear Static (Pushover) Analysis 27

2.2.1 Modeling for Pushover Analysis by Using SAP2000 30

2.2.2 Calculating the Capacity Curve Of the Structure 36

2.2.3 Calculating Displacement Demand of the Structure 37

2.2.4 Acceptability of Members in Nonlinear Procedure 42

2.3 Estimation of Building Performances 44

3. CASE STUDY I : ASSESSMENT OF A RESIDENTIAL BUILDING DESIGNED TO 1998 TURKISH EARTHQUAKE CODE……………………………………47

3.1 Linear Elastic Procedure 50

3.1.1 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})............... 50

3.1.2 Global Performance of the Building 58

3.2 Non-Linear Analysis..59

3.2.1 Comparison of Column and Beam Section Strains with the Section Strain Limits 59

3.2.2 Global Performance of the Building 65

4. CASE STUDY II : ASSESSMENT OF A RESIDENTIAL BUILDING DESIGNED TO 1975 TURKISH EARTHQUAKE CODE… 67

4.1 Assessment of the Building by Linear Elastic Procedure......................70

4.1.1 Calculation of the Equivalent Lateral Load Distribution 70

4.1.2 Distribution of the Base Shear Force to the Stories.. 71

4.1.3 Analysis of the Building Under Vertical Loading (G+nQ) and the Lateral Loading (E).. 72

4.1.4 Calculation of Beam End Moment Capacities (M_K)... 73

4.1.5 Calculation of the Column Axial Loads 74

4.1.6 Calculation of the Column End Moment Capacities.. 79

4.1.7 Shear Check for Beams and Columns... 79

4.1.8 Shear Check for Joints 82

4.1.9 Calculation of “Demand / Capacity Ratios (r)” and “Limit values (r_{Limit})” of Beam and Column Sections 84

4.1.10 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})............. 86

4.1.11 Global Performance of the Building 92

4.2 Assessment of the Building by Non-Linear Procedure 94

4.2.1 Calculation of Moment Curvature Relations of Beam Ends............................... 95

4.2.2 Calculation of Moment-Curvature Relations for Column Ends 97

4.2.3 Capacity Curve in X and Y Directions 98

4.2.4 Calculation of Performance Point in X and Y Directions................................... 98

4.2.5 Shear Check for Beams and Columns... 99

4.2.6 Shear Check for Joints 100

4.2.7 Calculation of Strains at Member Sections 101

4.2.8 Comparison of Column and Beam Section Strains with the Section Strain Limits 103
4.2.9 Global Performance of the Building 109

5. CASE STUDY III : ASSESSMENT OF A SCHOOL BUILDING BEFORE AND AFTER RETROFITTING 111

5.1 Properties of Existing School Building Before Retrofitting 111

5.1.1 Linear Elastic Analysis of Existing School Building Before Retrofitting 115
5.1.2 Global Performance of the Building 115

5.2 Properties of Existing School Building After Retrofitting 115

5.2.1 Linear Elastic Analysis of Existing School Building After Retrofitting 118
5.2.2 Global Performance of the Building 118
5.2.3 Non-linear Elastic Analysis of Existing School Building After Retrofitting 119
5.2.4 Global Performance of the Building 120

6. DISCUSSION OF RESULTS 121

7. CONCLUSIONS 139

REFERENCES 142

APPENDIX

STRESS-STRAIN DIAGRAMS USED IN CALCULATING MOMENT-CURVATURE RELATIONS 144

LIST OF TABLES

Table 2.1 r_{Limit} Values for Reinforced Concrete Columns 24
Table 2.2 r_{Limit} Values for Reinforced Concrete Beams 26
Table 2.3 r_{Limit} Values for Reinforced Concrete Shearwalls 26
Table 2.4 Storey Displacement Limits for Different Performance Levels 46
Table 3.1 Storey Masses, Mass Center Coordinates and Mass Moment of Inertias 47
Table 3.2 Project and Code Parameters of the Building 49
Table 3.3 Percentage of Unacceptable Beams and Columns 58
Table 3.4 Interstorey Drifts 59
Table 3.5 Percentage of Unacceptable Beams and Columns 65
Table 3.6 Interstorey Drifts 66
Table 4.1 Storey masses, mass center coordinates and mass moment of inertias 67
Table 4.2 Project and Code Parameters 69
Table 4.3 Distribution of the Base Shear Force to the Floors 71
Table 4.4 Beam End Moment Capacities of K104 73
Table 4.5 Axial Load Calculation for the Columns of Frame B 78
Table 4.6 Shear Check for the Joints (the joints at the top of the columns) of Frame B 83
Table 4.7 Percentage of Unacceptable Beams and Columns 93
Table 4.8 Interstorey Drifts 94
Table 4.9 Shear Check for the Joints (the joints at the top of the columns) of Frame B 101
Table 4.10 Percentage of Unacceptable Beams and Columns 109
Table 4.11 Interstorey Drifts 109
Table 5.1 Storey Masses, Mass Center Coordinates and Mass Moment of Inertias 114
Table 5.2 Project and Code Parameters of the Building 114
Table 5.3 Percentage of Unacceptable Beams and Columns 115
Table 5.4 Storey Masses, Mass Center Coordinates and Mass Moment of Inertias 117

Table 5.5 Project and Code Parameters of the Building 118
Table 5.6 Percentage of Unacceptable Beams and Columns 119
Table 5.7 Interstorey Drifts 119
Table 5.8 Percentage of Unacceptable Beams and Columns 120
Table 5.9 Interstorey Drifts 120
Table 7.1.a Comparison of Member Acceptabilities in Different Code Procedures for Residential 1998 139
Table 7.1.b Comparison of Member Acceptabilities in Different Code Procedures for Residential 1975 139
Table 7.1.c Comparison of Member Acceptabilities in Different Code Procedures for School Retrofitted 140

LIST OF FIGURES

Figure 1.1 Summary of Force-Based Assessment Procedure [2] 7

Figure 1.2 Summary of Displacement-Based Assessment Procedure for Frames [2] 8

Figure 1.3 Converting Capacity Curve into Capacity Spectrum 10

Figure 1.4 Performance Point Calculation 10

Figure 1.5 Idealized Force – Displacement Curve 14

Figure 2.1 Calculating Beam End Shear Capacities Due to Earthquake Loads 18

Figure 2.2 Calculating Column Axial Loads Due to Earthquake Loads 19

Figure 2.3 Moment Capacities of Beams and Columns at a Joint 20

Figure 2.4 Equal Displacement Rule 23

Figure 2.5 Moment – Curvature Relations for Changing N/A_cf_c Values 24

Figure 2.6 Interaction Diagram for α = 0º 31

Figure 2.7 Interaction Diagram for α = 45º 31

Figure 2.8 Interaction Diagram for α = 90º 31

Figure 2.9 Bi-linearization of Moment – Curvature Curves 34

Figure 2.10 Capacity and Demand Curve for $T \geq T_B$ 39

Figure 2.11 Calculating a_{y1}^{o} and a_{y1} 41

Figure 2.12 Target Performance Levels 44

Figure 3.1 Typical Structural Plan View 48

Figure 3.2 Column Dimensions 48

Figure 3.3 Three Dimensional Model 49

Figure 3.4 Frame B 49

Figure 3.5 r / r_{Limit} for 1st Storey Columns 50

Figure 3.6 r / r_{Limit} for 2nd Storey Columns 51

Figure 3.7 r / r_{Limit} for 3rd Storey Columns 51

Figure 3.8 r / r_{Limit} for 4th Storey Columns 52

Figure 3.9 r / r_{Limit} for 1st Storey Beams 52

Figure 3.10 r / r_{Limit} for 2nd Storey Beams 53

Figure 3.11 r / r_{Limit} for 3rd Storey Beams 53

Figure 3.12 r / r_{Limit} for 4^{th} Storey Beams 54
Figure 3.13 r / r_{Limit} for 1^{st} Storey Columns 54
Figure 3.14 r / r_{Limit} for 2^{nd} Storey Columns 55
Figure 3.15 r / r_{Limit} for 3^{rd} Storey Columns 55
Figure 3.16 r / r_{Limit} for 4^{th} Storey Columns 56
Figure 3.17 r / r_{Limit} for 1^{st} Storey Beams 56
Figure 3.18 r / r_{Limit} for 2^{nd} Storey Beams 57
Figure 3.19 r / r_{Limit} for 3^{rd} Storey Beams 57
Figure 3.20 r / r_{Limit} for 4^{th} Storey Beams 58
Figure 3.21 ε / $ε_{Limit}$ for 1^{st} Storey Columns 60
Figure 3.22 ε / $ε_{Limit}$ for 2^{nd} Storey Columns 60
Figure 3.23 ε / $ε_{Limit}$ for 3^{rd} Storey Columns 61
Figure 3.24 ε / $ε_{Limit}$ for 1^{st} Storey Beams 61
Figure 3.25 ε / $ε_{Limit}$ for 2^{nd} Storey Beams 62
Figure 3.26 ε / $ε_{Limit}$ for 3^{rd} Storey Beams 62
Figure 3.27 ε / $ε_{Limit}$ for 1^{st} Storey Columns 63
Figure 3.28 ε / $ε_{Limit}$ for 2^{nd} Storey Columns 63
Figure 3.29 ε / $ε_{Limit}$ for 1^{st} Storey Beams 64
Figure 3.30 ε / $ε_{Limit}$ for 2^{nd} Storey Beams 64
Figure 3.31 ε / $ε_{Limit}$ for 3^{rd} Storey Beams 65
Figure 4.1 Typical Storey Structural Plan View 68
Figure 4.2 Column Dimensions 68
Figure 4.3 Three Dimensional Model 69
Figure 4.4 Frame B 69
Figure 4.5 Natural vibration periods, mode shapes and effective mass ratios in X and Y directions 70
Figure 4.6 Linear Elastic Earthquake Spectrum 71
Figure 4.7 Moment Diagram of Frame B (M_D) under Vertical Loading 72
Figure 4.8 Moment Diagram of Frame B (M_E) under Lateral Loading 72
Figure 4.9 Torsional Irregularity Check 73
Figure 4.10 Calculating the Residual Moment Capacities $ΔM_E$ 75
Figure 4.11 The Shear Forces Transmitted from the Beams 77
Figure 4.12(a) The Interaction Diagram of 1S7 79

Figure 4.12(b) The Moment Capacities of Column 1S7 79
Figure 4.13 Calculation of M_{Kt} 80
Figure 4.14 Calculation of Shear Forces Transmitted from Beams........................ 82
Figure 4.15 Joint Section.. 83
Figure 4.16 r / r_{Limit} Values for 1st Storey Columns 86
Figure 4.17 r / r_{Limit} Values for 2nd Storey Columns 87
Figure 4.18 r / r_{Limit} Values for 3rd Storey Columns 87
Figure 4.19 r / r_{Limit} Values for 1st Storey Beams 88
Figure 4.20 r / r_{Limit} Values for 2nd Storey Beams 88
Figure 4.21 r / r_{Limit} Values for 3rd Storey Beams 89
Figure 4.22 r / r_{Limit} Values for 1st Storey Columns 89
Figure 4.23 r / r_{Limit} Values for 2nd Storey Columns 90
Figure 4.24 r / r_{Limit} Values for 3rd Storey Columns 90
Figure 4.25 r / r_{Limit} Values for 1st Storey Beams 91
Figure 4.26 r / r_{Limit} Values for 2nd Storey Beams 91
Figure 4.27 r / r_{Limit} Values for 3rd Storey Beams 92
Figure 4.28 Natural Vibration Periods, Mode Shapes and Effective Mass Ratios in X and Y Directions.. 95
Figure 4.29 Moment-Curvature Relation for K104.. 95
Figure 4.30 Negative Moment-Plastic Curvature and Moment-Plastic Rotation Relations 96
Figure 4.31 Positive Moment-Plastic Curvature and Moment-Plastic Rotation Relations 96
Figure 4.32 Interaction Diagrams, Moment-Plastic Curvature Relations, Moment-Plastic Rotation Relations in X and Y Directions 97
Figure 4.33 Capacity Curves in X and Y directions 98
Figure 4.34 Capacity-Demand Curve in X Direction.. 99
Figure 4.35 Capacity-Demand Curve in Y Direction.. 99
Figure 4.36 Bottom Section of Column 1S7 102
Figure 4.37 ε / ε_{Limit} Values for 1st Storey Columns 103
Figure 4.38 ε / ε_{Limit} Values for 2nd Storey Columns 104
Figure 4.39 ε / ε_{Limit} Values for 1st Storey Beams 104
Figure 4.40 ε / ε_{Limit} Values for 2nd Storey Beams 105

Figure 4.41 $\varepsilon / \varepsilon_{Limit}$ Values for 3rd Storey Beams....105
Figure 4.42 $\varepsilon / \varepsilon_{Limit}$ Values for 1st Storey Columns....106
Figure 4.43 $\varepsilon / \varepsilon_{Limit}$ Values for 2nd Storey Columns....106
Figure 4.44 $\varepsilon / \varepsilon_{Limit}$ Values for 3rd Storey Columns....107
Figure 4.45 $\varepsilon / \varepsilon_{Limit}$ Values for 1st Storey Beams....107
Figure 4.46 $\varepsilon / \varepsilon_{Limit}$ Values for 2nd Storey Beams....108
Figure 4.47 $\varepsilon / \varepsilon_{Limit}$ Values for 3rd Storey Beams....108
Figure 5.1 Front View of the School Building....112
Figure 5.2 Typical Structural Plan View....112
Figure 5.3 Three Dimensional Model....113
Figure 5.4 Frame C, Rigid End Zones, Example Column and Beam....113
Figure 5.5 Typical Structural Plan View....116
Figure 5.6 Three Dimensional Model....116
Figure 5.7 Frame C, Rigid End zones, Example Column and Beam....117
Figure 6.1 First Storey Columns of RESIDENTIAL 1998 in X Direction....122
Figure 6.2 First Storey Columns of RESIDENTIAL 1998 in Y Direction....122
Figure 6.3 First Storey Beams of RESIDENTIAL 1998 in X Direction....123
Figure 6.4 Second Storey Beams of RESIDENTIAL 1998 in X Direction....124
Figure 6.5 Third Storey Beams of RESIDENTIAL 1998 in X Direction....124
Figure 6.6 First Storey Beams of RESIDENTIAL 1998 in Y Direction....124
Figure 6.7 Second Storey Beams of RESIDENTIAL 1998 in Y Direction....125
Figure 6.8 Third Storey Beams of RESIDENTIAL 1998 in Y Direction....125
Figure 6.9 First Storey Columns of RESIDENTIAL 1975 in X Direction....126
Figure 6.10 First Storey Columns of RESIDENTIAL 1975 in Y Direction....127
Figure 6.11 First Storey Beams of RESIDENTIAL 1975 in X Direction....128
Figure 6.12 Second Storey Beams of RESIDENTIAL 1975 in X Direction....129
Figure 6.13 First Storey Beams of RESIDENTIAL 1975 in Y Direction....129
Figure 6.14 Second Storey Beams of RESIDENTIAL 1975 in Y Direction....130
Figure 6.15 First Storey Columns of SCHOOL RETROFITTED in X Direction....131
Figure 6.16 First Storey Columns of SCHOOL RETROFITTED in Y Direction....131
Figure 6.17 First Storey Shearwalls of SCHOOL RETROFITTED in X Direction.132
Figure 6.18 First Storey Shearwalls of SCHOOL RETROFITTED in Y Direction.132
Figure 6.19 First Storey Beams of SCHOOL RETROFITTED in X Direction....134

Figure 6.20 Second Storey Beams of SCHOOL RETROFITTED in X Direction .. 135
Figure 6.21 Third Storey Beams of SCHOOL RETROFITTED in X Direction 135
Figure 6.22 Fourth Storey Beams of SCHOOL RETROFITTED in X Direction 136
Figure 6.23 First Storey Beams of SCHOOL RETROFITTED in Y Direction 136
Figure 6.24 Second Storey Beams of SCHOOL RETROFITTED in Y Direction .. 137
Figure 6.25 Third Storey Beams of SCHOOL RETROFITTED in Y Direction 137
Figure 6.26 Fourth Storey Beams of SCHOOL RETROFITTED in Y Direction 138
Figure A.1 Stress – Strain Model for Concrete 146
Figure A.2 Stress – Strain Model for Steel 147

CHAPTER I
INTRODUCTION

1.1 Aim of the Study

The basic aim of this thesis is to evaluate the assessment methods presented in the 2006 Turkish Earthquake Code, Chapter 7: “Assessment and Retrofit of Existing Buildings”. The analysis procedures in the code are classified as linear elastic and non-linear, however the elastic analysis procedure is complemented with the capacity analysis. The main purpose of this linear analysis procedure is to reach reliable results by combining linear analyses with capacity principles in order to determine seismic performance of buildings.

Nonlinear analysis methods are more rigorous in evaluating the seismic performance of buildings than the linear procedures since they allow redistribution of internal actions in the post-elastic range. On the other hand, linear elastic analysis methods are preferred in practice since less effort is required to obtain the results. In addition, the calculation procedures of linear elastic analysis are straight forward and more practical. The accuracy of linear elastic procedure can be improved by using the capacity principles. The efficiency of this method can be tested by comparing the results of nonlinear procedure by the results of linear elastic analysis.

The main objective of this thesis is to assess the seismic performances of the selected buildings by the linear elastic procedure and the nonlinear procedure, then carry out a critical comparative evaluation based on the obtained results.

To manage this objective, the following steps are implemented for each selected building :

The first step is to conduct non-linear static analysis(pushover) for each building. This pushover analysis is performed by a 3-D model in SAP2000 [1]. Instead of default values for hinge properties in SAP2000, the moment-curvature diagrams are obtained by an Excel Macro as well as the interaction diagrams for each member of the building, and interaction diagrams are converted into 3-D diagrams by the procedure proposed by Parme et al. (1966). Using the results obtained from analysis, the performance level of the building is estimated by comparing strain values corresponding to plastic rotations with the limit values of strain values given in the proposed earthquake code.

The second step is to conduct linear elastic analysis, combined with the capacity analysis in order to determine the column and beam capacities of each building under seismic effects. Then the demand to capacity ratios are determined in order to decide on the member performances. From this analysis, the performance level of the building is determined by comparing the related demand to capacity ratios with the limit values proposed in the 2006 Turkish Earthquake Code.

The results obtained by the procedures discussed in these two steps are compared in order to verify the procedures and the performance limit states proposed by the 2006 Turkish Earthquake Code.

1.2 Review of Past Studies

Linear elastic (force-based) and nonlinear (displacement-based) procedures for seismic assessment take place in several seismic codes and guidelines. These are summarized below for different codes.

New Zealand Assessment Guidelines

Both force-based and displacement-based assessment methods for concrete buildings are included in this guideline. It is stated that [2] :

"Force-based methods

The assessment procedure is based on determining the probable strength and ductility of the critical mechanism of post-elastic deformation of the lateral force-resisting elements.

Once the available lateral load strength and displacement ductility of the structure has been established with reference to the NBS (New Building Standard) *response spectra for earthquake forces for various levels of structural ductility factor then enables the designer to assess the likely seismic performance of the structure in relation to that of a new building. Such comparisons will need to take account of any modifications to NBS requirements necessary to address existing buildings (as given in the Guidelines).*

The key steps of a force-based seismic assessment procedure (Park 1996) can be summarized as follows:

Step F1: Estimate the probable flexural and shear strengths of the critical sections of the members and joints assuming that no degradation of strength occurs due to cyclic lateral loading in the post-elastic range.

Step F2: *Determine the post-elastic mechanism of deformation of the structure that is likely to occur during seismic loading and the probable lateral seismic force capacity of the structure, V.*

Step F3: *Estimate the basic seismic hazard coefficient $C_h(T,\mu)$ corresponding to the ideal lateral force capacity of the structure, V, found in Step F2 from:*

$$C_h(T,\mu) = \frac{V}{W_t S_p R Z} \qquad 1.1$$

where: W_t = *seismic weight of the structure*

S_p = *structural performance factor*

R = *risk factor for the structure*

Z = *zone factor*

Step F4: *Estimate the fundamental period of vibration of the structure, T. Then using the appropriate seismic hazard acceleration spectra of NZS*(New Zealand Standard) *4203:1992 determine the required structural ductility factor μ for the estimated $C_h(T,\mu)$ and T.*

Step F5: *Evaluate whether the identified plastic hinge regions have the available ductility to match the required overall structural ductility factor μ. The element will require retrofitting if the rotation capacity of the plastic hinges is inadequate.*

Step F6: *Estimate the degradation in the shear and bond strength of members and joints during cyclic deformations at the imposed curvature ductility factor in the plastic hinge regions. Check whether any degradation in shear and bond strength will cause failure of the* *members or joints. If it does not, then the assessment apart from Step F7 is complete. If it does, the structure will require strengthening.*

Step F7: *Estimate the interstorey drift and decide whether it is acceptable in terms of the requirements of NZS 4203.*

The sequencing of and interaction between these steps is shown in the flowchart form in Figure 1.1

Displacement-based methods

Displacement-based methods place a direct emphasis on establishing the ultimate displacement capacity of lateral force resisting elements. Displacement-based assessment utilizes displacement spectra which can more readily represent the characteristics of actual earthquakes.

The development of procedures encompassing this approach represents a relatively recent development. In 1995 Priestley developed an outline of the key steps for such a procedure for reinforced concrete buildings. He has taken this work further, with appropriate simplifications, to produce the following general procedure which is considered more suitable for use in a design office context.

The key steps of a displacement-based seismic assessment procedure can be summarized as follows:

Step D1: *Calculate the probable flexural strengths of the critical sections of the members.*

Step D2: *Determine the post-elastic deformation mechanism, and hence lateral force capacity of the structure.*

Step D3: Calculate member plastic rotation capacities from moment curvature analyses.

Step D4: Calculate shear strengths of members (and joints where applicable) in order to determine whether shear failure will occur before the limits to flexural plastic rotation capacity are reached. The available plastic rotation capacity is reduced if necessary to the value pertaining at shear failure. The storey plastic drift capacity is estimated from the plastic rotation capacities.

Step D5: The overall structure displacement capacity, Δ_{sc}*, and ductility capacity,* μ_{sc}*, are found from the mechanism determined in Step D2 and the critical storey drift.*

Step D6: Calculate the effective stiffness at maximum displacement, and the corresponding effective period of vibration. Determine the equivalent viscous damping of the structure.

Step D7: Estimate the structure displacement demand, Δ_{sd}*, from the code displacement spectra.*

Step D8: Compare the displacement capacity, Δ_{sc}*, against the demand,* Δ_{sd}*, and establish compliance or otherwise.*

The sequencing of and interaction between these steps is shown in flowchart form in Figure 1.2" (New Zealand Society for Earthquake Engineering, 2003) [2]

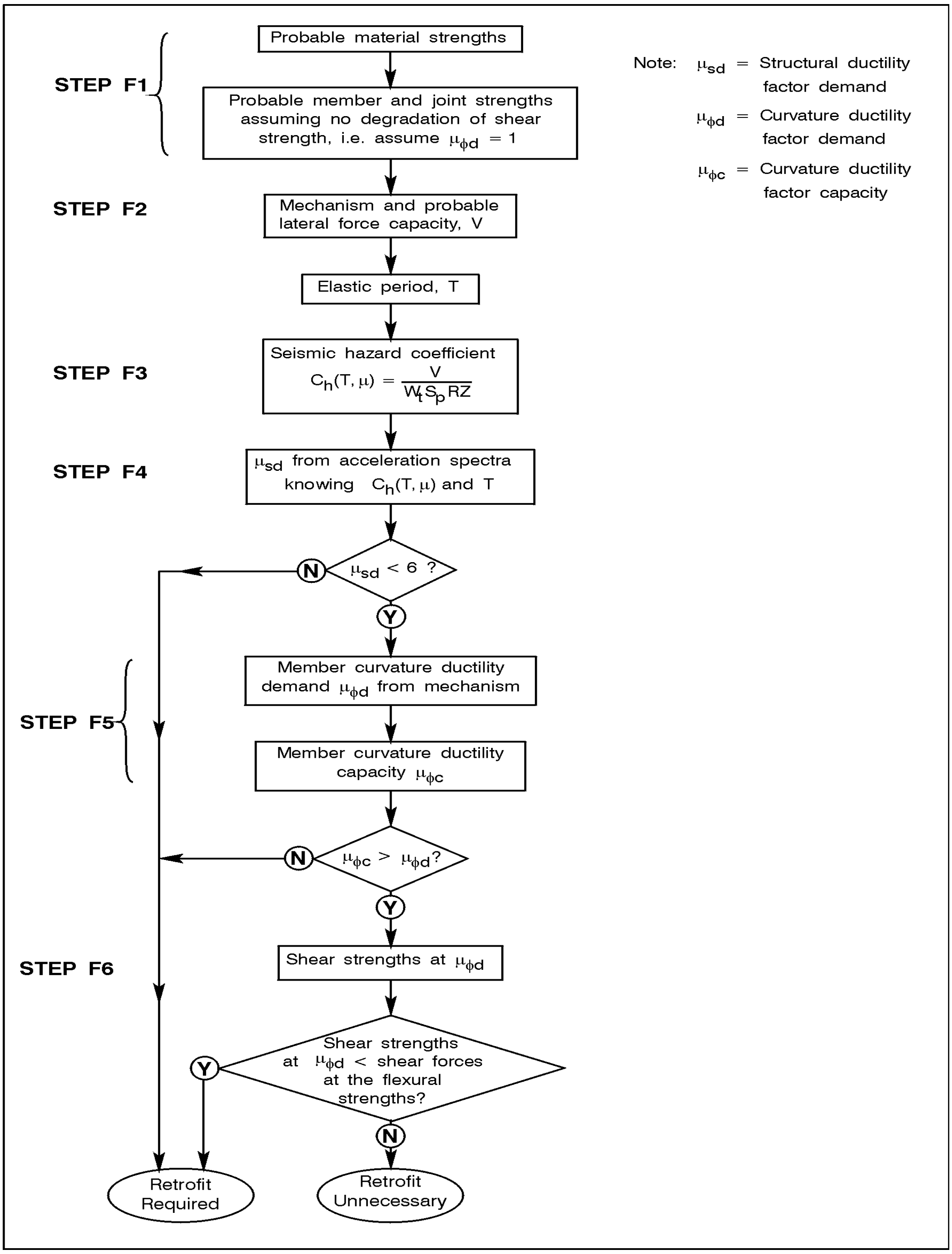


Figure 1.1 Summary of Force-Based Assessment Procedure [2]

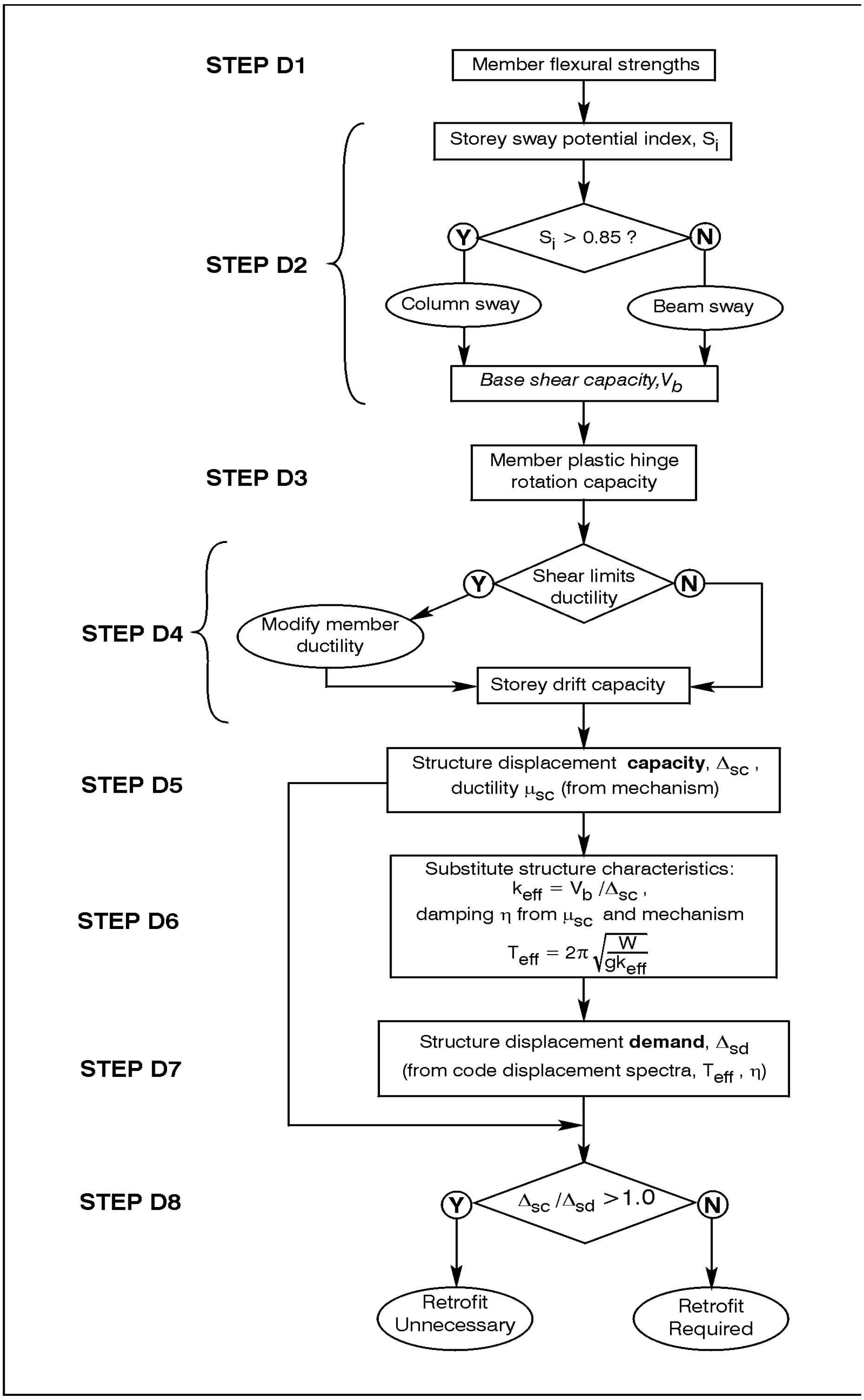


Figure 1.2 Summary of Displacement-Based Assessment Procedure for Frames [2]

ATC-40 Seismic Evaluation and Retrofit of Concrete Buildings (1996) [3]:

Non-linear static procedures are employed in this document and the Capacity Spectrum Method is highlighted, since linear static procedures can not predict failure mechanism after first yield.

Non-linear static procedures are composed of the following steps :

- Members are classified as brittle and ductile. If shear demand of any member exceed its shear capacity, member is defined as brittle, otherwise it is defined as ductile.

- For ductile members plastic rotation capacities are determined from moment – curvature relations. Plastic rotation capacities are calculated by multiplying the plastic curvature capacity with plastic hinge length.

- For ductile members, plastic rotation demands are calculated by the following steps:

1. Pushover curve (capacity curve) for the selected building is obtained.

2. Capacity curve in force-displacement format is converted to ADRS (Acceleration Displacement Response Spectrum) format(Figure 1.3)

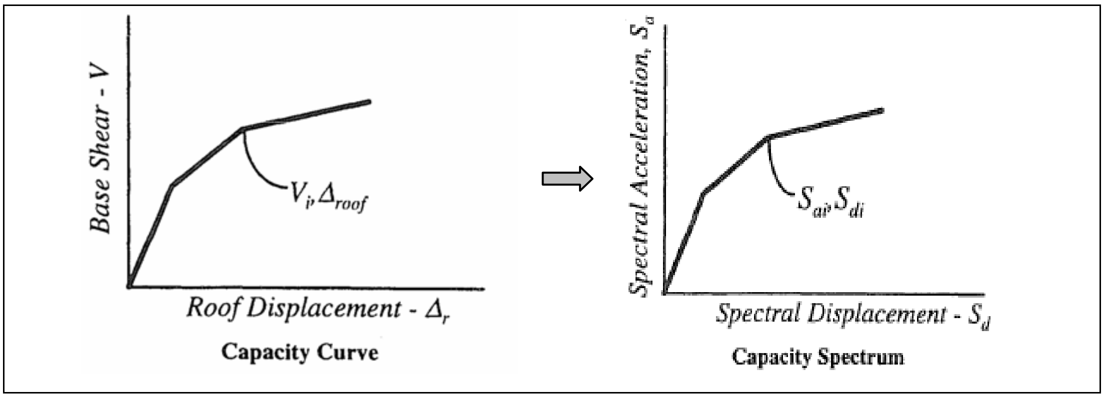


Figure 1.3 Converting Capacity Curve into Capacity Spectrum

3. Elastic response spectrum is obtained and converted to ADRS format as a demand curve.
4. The demand and capacity curve is plotted together to estimate spectral displacement demand at their intersection. (Figure 1.4)

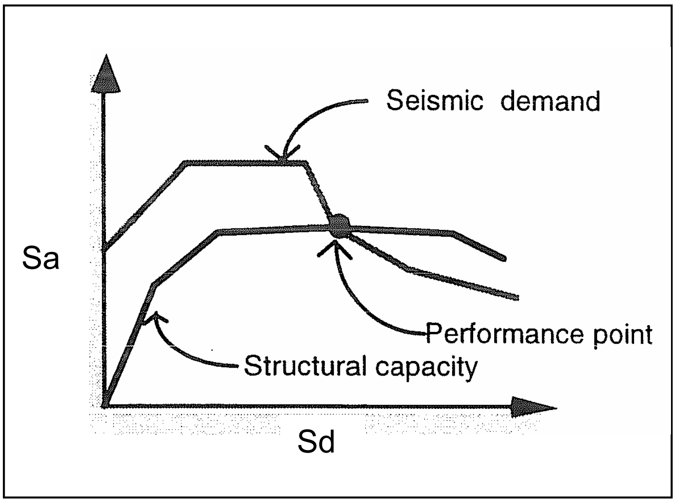


Figure 1.4 Performance Point Calculation

5. Displacement demand is calculated from this spectral displacement demand and called as performance point.
6. The building is pushed up to calculated performance point, and plastic rotation demands are obtained

- The calculated plastic rotation demands are compared with the plastic rotation limit values defined in ATC-40 to determine the acceptance of the member.

FEMA Seismic Rehabilitation Prestandard (1997, 2000) [4]:

The analysis methods employed in FEMA are Linear Static Procedure, Linear Dynamic Procedure, Nonlinear Static Procedure and Nonlinear Dynamic Procedure. Elements and components are classified as primary or secondary according to their contribution to overall stiffness, strength and deformation capacity. In addition, the elements are classified as ductile or brittle according to their mode of failure. If the failure mode is shear, the element is defined as brittle, and if the failure mode is flexure, the element is defined as ductile.

Linear Static Procedure is composed of the following steps:

-The moment capacities and shear capacities of the members are calculated (Q_{CE}) for ductile elements and (Q_{CL}) for brittle elements.

-Elastic demands of the members are calculated by the following steps:

1. Fundamental period of the building is calculated for each direction under consideration.

2. The pseudo lateral load for the direction under consideration is determined elastically and distributed at every floor level.
3. Internal forces and system displacements are determined from this pseudo lateral force as elastic demands (Q_{UD}) for ductile elements and (Q_{UF}) for brittle elements.

-Q_{UD} values are compared with Q_{CE} values to determine the acceptance of the ductile members. The ductile members should satisfy the following equation :

$$m*k* Q_{CE} > Q_{UD} \tag{1.2}$$

where :

m = component or element demand modifier to account for expected ductility associated with this action at the selected Structural Performance Level.

k = knowledge factor.

-Q_{UF} values are compared with Q_{CL} values to determine the acceptance of the brittle members. The brittle members should satisfy the following equation:

$$k * Q_{CL} > Q_{UF} \tag{1.3}$$

where:

k = knowledge factor.

Linear Dynamic Procedure is composed of the following steps:

-The moment capacities and shear capacities of the members are calculated.

-Elastic demands of the members are calculated by the following steps:

1. Modal spectral analysis is applied by linear elastic response spectra, which is not modified for non-linear response.
2. Peak modal responses of the modes summing up to 90% of participating mass of the building are calculated by means of spectral modal analysis. After that, these modal peak responses are combined by SRSS (square root sum of squares) rule or CQC (complete quadratic combination) rule.

-Component demands should satisfy the acceptance criteria specified at the selected Structural Performance Level.

Non-Linear Static Procedure is composed of the following steps:

-For ductile members plastic rotation capacities are calculated.

- For ductile members, plastic rotation demands are calculated by the following steps:

1. The force-displacement relation (capacity curve) for control node is obtained by single mode pushover analysis.
2. From this capacity curve, effective lateral stiffness and effective yield strength is calculated by balancing the areas below and above part of the capacity curve.

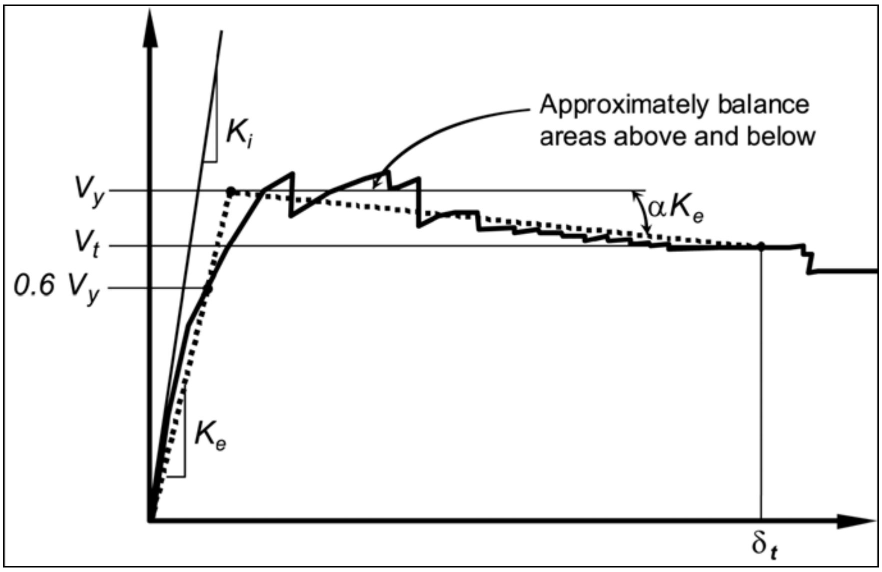


Figure 1.5 Idealized Force – Displacement Curve

3. Effective fundamental period is calculated by using effective lateral stiffness and elastic stiffness (Figure 1.5):

$$T_e = T_i * \sqrt{\frac{K_i}{K_e}} \tag{1.4}$$

where:

T_i = elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis.

K_i = elastic lateral stiffness of the building in the direction under consideration.

K_e = effective lateral stiffness of the building in the direction under consideration

4. By this effective fundamental period and some modification factors, the target displacement is calculated. This method is called the Coefficient Method.
5. The building is pushed up to the calculated target displacement, and plastic rotations demands are obtained.

-The calculated plastic rotation demands are compared with the plastic rotation limit values defined in FEMA to determine the acceptance of members.

In Non-Linear Dynamic Procedure, instead of calculating an approximate target displacement, the demand forces and displacements are estimated by dynamic analysis using ground motion time histories.

Eurocode 8, Part 3, Strengthening and Repair of Buildings (2001) [5] :

The analysis methods employed in Eurocode 8 are :

- Equivalent lateral force analysis (linear)
- multi-modal response spectrum analysis (linear)
- non-linear static analysis
- non-linear time history analysis

Linear Methods of Analysis (static or dynamic):

- Capacities are changed according to corresponding limit states.
- for ductile elements, the demands are those obtained from analysis
- for brittle elements, the demands are those obtained from analysis or the value obtained by equilibrium conditions, considering the strength of ductile components.

Non-Linear Methods of Analysis (static or dynamic):

- Capacities are changed according to corresponding limit state.
- for both ductile and brittle elements, the demands are those obtained from analysis.

There are three type of limit states; Near Collapse Limit State, Significant Damage Limit State and Damage Limitation Limit State.

For Near Collapse Limit State, capacities are based on approximately defined ultimate deformations for ductile elements and on ultimate strengths for brittle elements.

For Significant Damage Limit State, capacities are based on damage related deformations for ductile elements and on conservatively estimated capacities for brittle ones.

For Damage Limitation Limit State, capacities are based on yield strengths for all structural elements.

CHAPTER II

DESCRIPTION OF ASSESSMENT METHODS FOR EXISTING BUILDINGS IN 2006 TURKISH SEISMIC REHABILITATION CODE

The assessment procedures studied in this thesis are linear elastic analysis with equivalent lateral load distribution and non-linear analysis (pushover analysis) with equivalent lateral load distribution. These assessment procedures are summarized below.

2.1 Linear Elastic Analysis

Linear elastic analysis is one of the two analysis procedures included in the 2006 Turkish Seismic Rehabilitation Code. The prerequisites for this procedure are such that:

The selected buildings should not be taller than 25m, have at most 8 stories and torsional irregularity coefficient is smaller than 1.4

2.1.1 Modeling for Linear Elastic Analysis

- Three dimensional models are prepared and analyzed by SAP2000. However, the procedure is applied in two orthogonal directions separately.

- For demand calculation, R is taken as 1, and building importance factor I is also taken as 1. By these values, structural base shear force is calculated and distributed to the floors. The earthquake demands in members are calculated under this lateral force distribution.

- The demands caused by the vertical loading (1G + nQ) are also calculated by linear elastic analysis.

2.1.2 Calculating the Member Capacities

1. According to the loading directions, the capacities of beam ends (M_k) are calculated. Existing strength values of concrete and strength obtained from site inspection are used for calculation of capacities. For positive loading, the bottom reinforcement capacity at the left end and top reinforcement capacity at the right end is utilized. For negative loading direction vice versa is true.

2. Under vertical loading (1G + nQ) , the end moments of beams (M_D) are calculated and these moments are subtracted vectorially from capacity moments of the related ends (M_k). By using the resulting residual moments (ΔM_k) the shear forces (V_E) are calculated as follows (Figure 2.1) :

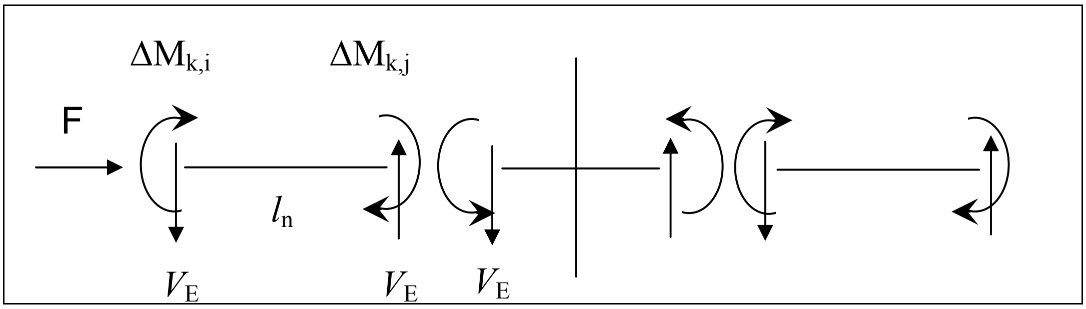


Figure 2.1 Calculating Beam End Shear Capacities Due to Earthquake Loads

$$V_E = \left(\Delta M_{k,i} + \Delta M_{k,j}\right) / l_n \tag{2.1}$$

$$\Delta M_{k,i} = M_{k,i} - M_{D,i}$$
$$\Delta M_{k,j} = M_{k,j} - M_{D,j} \qquad (2.2)$$

where:

i refers to the *i* end of the beam and *j* refers to the *j* end of the beam in Equation (2.1) and Equation (2.2).

l_n is the clear length of the beam.

If the shear forces calculated from earthquake loading are not greater than V_E values, then these shear forces are taken as V_E

3. Beam end shear capacities (V_E) calculated in Step 2 are transferred to the columns. All (V_E) values transferred to a column are added vectorially in order to calculate the axial forces due to earthquake loading (N_E) (Figure 2.2)

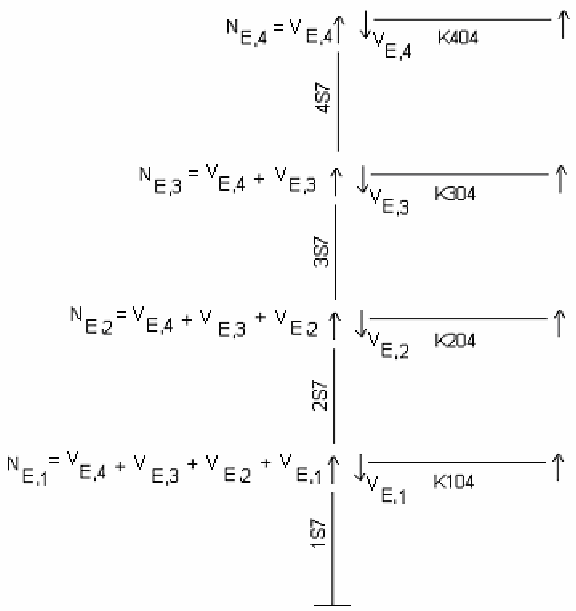


Figure 2.2 Calculating Column Axial Loads Due to Earthquake Loads

4. Under vertical loading (1G + nQ), the axial loads of columns (N_D) are calculated, then they are combined by the axial forces calculated in Step 3.

$$N_r = N_D + N_E \tag{2.3}$$

From these N_r values, the moment capacities (M_k) of column ends are estimated from the interaction diagrams.

The column to beam capacity ratios (*CBCR*) are calculated from the M_k values of beam ends and column ends as follows (Figure 2.3):

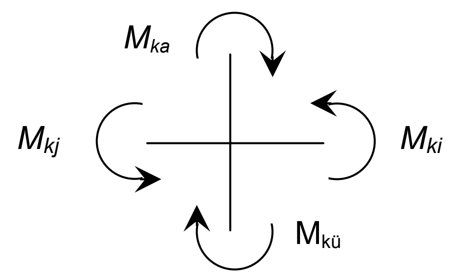


Figure 2.3 Moment Capacities of Beams and Columns at a Joint

M_{ka} = bottom moment capacity of the column above the joint
$M_{kü}$ = top moment capacity of the column below the joint
M_{kj}= right beam end capacity of the beam at left hand side of the joint
M_{ki}= left beam end capacity of the beam at right hand side of the joint

$$CBCR = \frac{\textit{Total column moment capacities at joint } (M_{ka} + M_{kü})}{\textit{Total beam moment capacities at joint } (M_{kj} + M_{ki})} \tag{2.4}$$

5. If *CBCR* > 1 the beams connected to the joint may yield, and there is no need to modify moment capacities of beam ends, the column and beam capacities estimated at the 4th step are valid.

6. If *CBCR* < 1 the columns connected to the joint may yield, hence the beam end moment capacities should be modified.

M_{kj} and M_{ki} are multiplied by the *CBCR* values in order to correct the beam end capacities.

Step 3 and step 4 are repeated with the modified beam end capacities.

7. The corresponding moment capacities of the columns are calculated by using the final N_r values.

However, these moment capacities are valid for ductile elements. In other words, if the failure mode of element is ductile, the moment capacities are used, but if the failure mode is brittle, shear capacities are valid.

Shear capacities (V_r) for beam end sections and column mid sections are calculated according to TS 500. If V_E is greater than V_r, then the element is defined as brittle, otherwise ductile.

8. The demands are calculated by linear elastic analysis. The demand calculation is performed by SAP2000. For ductile members, section moments are taken as demand. For brittle members, section shear forces are taken as demand.

For ductile beams, the beam residual capacities are $\Delta M_{k,i}$ and $\Delta M_{k,j}$ from the 2nd step of the procedure.

For ductile columns, the column capacities are M_k values calculated at the 4th step (if CBCR values are less than 1, column capacities are to be M_k values calculated at 7th step after repeating).

For brittle beam, column and shear walls, the shear capacities are calculated according to TS-500.

2.1.3 Performance Assessment of Members

The assessment includes the following procedure:

By dividing the earthquake demands (moments) with the corresponding residual capacities, the demand to capacity ratios (r) are calculated.

The limit r values (r_{Limit}) are obtained from the tables of the 2006 Turkish Earthquake Code for the target performance level.

For columns, this r_{Limit} values (Table 2.1) are related to the axial force, shear force and confinement of member ends.

In r-factor Tables, r values are referred to the ductility factors derived from the equal displacement rule in Figure 2.4

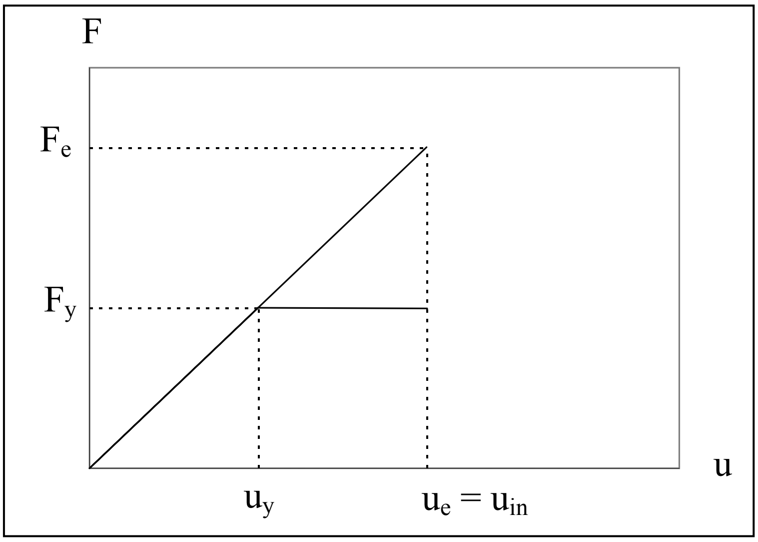


Figure 2.4 Equal Displacement Rule

$$\frac{F_e}{F_y} = \frac{u_{in}}{u_y} = r \tag{2.5}$$

In Table 2.1, N/A_cf_c values are related with the moment curvature relations accordingly with the ultimate curvature capacity. (Figure 2.5) [7] As N/A_cf_c increases, ultimate curvature capacity decreases so that the ultimate rotation capacity decreases. For this reason, the r_{Limit} values are decreasing for increasing N/A_cf_c values.

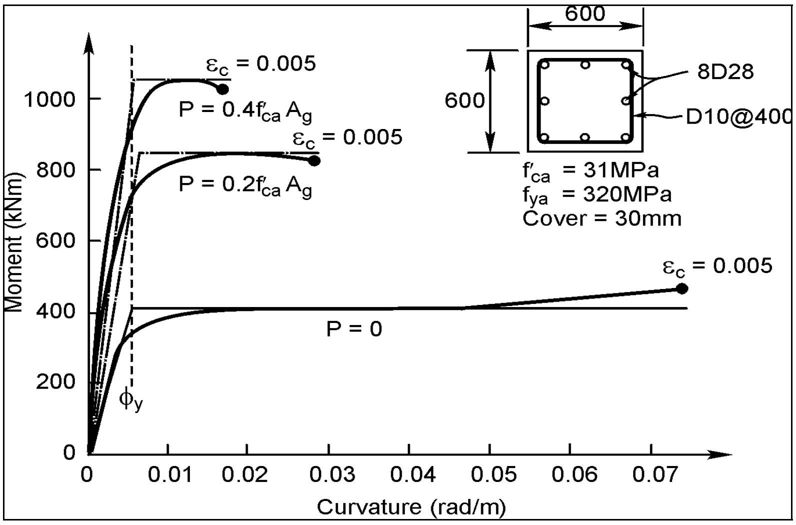


Figure 2.5 Moment – Curvature Relations for Changing N/A_cf_c Values

Table 2.1 r_{Limit} Values for Reinforced Concrete Columns

Ductile Columns			**Performance Level**		
$\frac{N}{A_c f_c}$	**Confinement**	$\frac{V}{b_w d\, f_{ct}}$	**IO**	**LS**	**CP**
≤ 0.1	YES	≤ 0.65	3	6	8
≤ 0.1	YES	≥ 1.30	2.5	5	6
≥ 0.4	YES	≤ 0.65	2	4	6
≥ 0.4	YES	≥ 1.30	2	3	5
≤ 0.1	NO	≤ 0.65	2	3.5	5
≤ 0.1	NO	≥ 1.30	1.5	2.5	3.5
≥ 0.4	NO	≤ 0.65	1.5	2	3
≥ 0.4	NO	≥ 1.30	1	1.5	2
Brittle Columns			1	1	1

For beams, r_{Limit} limit values (Table 2.2) are related to the shear force, reinforcement ratio and confinement of member ends:

The reinforcement ratio in beams is defined as:

$$\frac{\rho - \rho'}{\rho_b} \tag{2.6}$$

where:

ρ is the tension reinforcement ratio.

ρ' is the compression reinforcement ratio.

ρ_b is the balance reinforcement ratio.

For beam sections, if $\rho - \rho' < \rho_b$ then the section will be under-reinforced. Therefore, $\dfrac{\rho - \rho'}{\rho_b}$ is a factor related with ductility of the sections. For increasing $\dfrac{\rho - \rho'}{\rho_b}$ values ductility decreases and r_{Limit} decreases.

For both beam and column sections, as shear demand increases, the section tends to behave as brittle. Hence, for increasing $\dfrac{V}{b_w d f_{ct}}$ values ductility decreases and r_{Limit} decreases

Table 2.2 r_{Limit} Values for Reinforced Concrete Beams

Ductile Beams			**Performance Level**		
$\frac{\rho - \rho'}{\rho_b}$	**Confinement**	$\frac{V}{b_w d\, f_{ct}}$	**IO**	**LS**	**CP**
≤ 0.0	YES	≤ 0.65	3	7	10
≤ 0.0	YES	≥ 1.30	2.5	5	8
≥ 0.5	YES	≤ 0.65	3	5	7
≥ 0.5	YES	≥ 1.30	2.5	4	5
≤ 0.0	NO	≤ 0.65	2.5	4	6
≤ 0.0	NO	≥ 1.30	2	3	5
≥ 0.5	NO	≤ 0.65	2.5	4	6
≥ 0.5	NO	≥ 1.30	1.5	2.5	4
Brittle Beams			1	1	1

Table 2.3 r_{Limit} Values for Reinforced Concrete Shearwalls

Ductile Shearwalls	**Performance Level**		
Confinement	**IO**	**LS**	**CP**
YES	3	6	8
NO	2	4	6
Brittle Shearwalls	1	1	1

The r values are divided by r_{Limit} values in order to calculate the r/ r_{Limit} values.

Acceptability of members are related with the r/ r_{Limit} ratios.

If $(r / r_{Limit}) < 1$ the corresponding end of member is acceptable for the target performance level.

If $(r / r_{Limit}) > 1$ the corresponding end of member is not acceptable for the target performance level.

If both ends of a member is acceptable then the member will be acceptable. However if one of the ends is not acceptable, member will be not acceptable for the target performance level.

2.2 Nonlinear Static (Pushover) Analysis

The single mode (effective) nonlinear static analysis procedure can be applied to the buildings not taller than 25m, have at most 8 stories and torsional irregularity coefficient is smaller than 1.4

According to the probability of exceedance, the target performance level is selected and the building is controlled for this performance level.

After selecting the performance level, the following steps are carried out:

1. 3-D analytical model of the building is prepared.

2. For column and shear wall plastic sections, moment-plastic rotation relations and interaction diagrams are defined. For beam plastic sections, moment-plastic rotation relations are defined. These plastic sections are assigned at the clear span ends for beams and columns, and at the bottom of the shear walls.

3. By using uncracked section stiffness values, a static analysis under vertical loading (1G + nQ) is performed.

4. Cracked section stiffnesses are calculated by the axial load values calculated in Step 3.

5. In buildings for which the slabs are modeled as rigid diaphragms, two orthogonal translational masses and moment of inertia values are assigned at the mass center of each floor. By these masses and cracked stiffness values, the fundamental periods and the fundamental mode shapes are calculated.

6. Equivalent lateral load distribution is calculated by multiplying the masses and modal amplitudes for each floor level for the direction under consideration, then assigned to the mass center of that floor.

7. The static analysis under vertical loading is repeated. Then a pushover analysis with equivalent lateral load distribution defined in Step 6 is performed. As a result of this analysis, the base shear capacity – roof displacement curve is obtained (See Section 2.2.2).

8. Base shear capacity – roof displacement curve is converted into spectral displacement – spectral acceleration curve and called as modal capacity curve.

9. Spectral displacement demand is calculated by using the modal capacity curve and elastic demand spectrum.

10. The target displacement is calculated from spectral displacement demand, as explained in the Section 2.2.3.

11. At the target displacement, member shear forces are calculated. These shear forces are compared with the shear capacities obtained from TS500. If the member shear force at critical section is greater than shear capacity, then the failure of this member is brittle. This member can not satisfy “Collapse Prevention Limit State”. Otherwise this member is ductile.

12. For ductile members, plastic rotation demands at the target displacement are obtained. From these plastic rotations, plastic curvatures are calculated. The yield curvatures are obtained from moment-curvature relations. These yield and plastic curvatures are added in order to obtain total curvatures.

13. The concrete and steel strain values corresponding to the total curvatures calculated in Step12 are obtained from moment-curvature relations. The concrete and steel strain limit values are calculated according to the code. If concrete or steel strain does not satisfy the limit, this member end does not satisfy the selected limit state. If any member end does not satisfy the limit, this member does not satisfy the performance limit.

14. According to the number of unacceptable beams at each storey and storey shear contribution of unacceptable columns, the acceptance of each storey is determined. If any storey is unacceptable, then the building does not satisfy the selected performance level.

15. Finally, inter-storey drifts are calculated at each storey and compared with the limits defined in the code. If these inter-storey drifts are greater than the limit values at any storey, then the building does not satisfy the selected performance level.

2.2.1 Modeling for Pushover Analysis by Using SAP2000

The summary of pushover analysis is given above, however some points should be highlighted in detail. These points are explained in the following paragraphs:

1. Three dimensional building models are prepared and analyzed by SAP2000. In this study, the moment – curvature relations as well as the interaction diagrams are obtained separately for beam and column sections and then inserted into SAP2000 as hinge property data.

Although SAP2000 can not update the moment – rotation relationships with changing axial forces, it can update the yield and ultimate moment capacities by using interaction diagrams. Axial force–moment capacity curves corresponding to bending about two major axis of each column section are obtained. From these major curves, the other axial force - moment capacity curves (Figure 2.6 - 2.8) required to define interaction surfaces are obtained by the following equation proposed by Parme et al (1966):

$$\left(\frac{\text{Mux}}{\text{Muxo}}\right)^{\left(\frac{\log(0.5)}{\log(\beta)}\right)} + \left(\frac{\text{Muy}}{\text{Muyo}}\right)^{\left(\frac{\log(0.5)}{\log(\beta)}\right)} = 1 \tag{2.7}$$

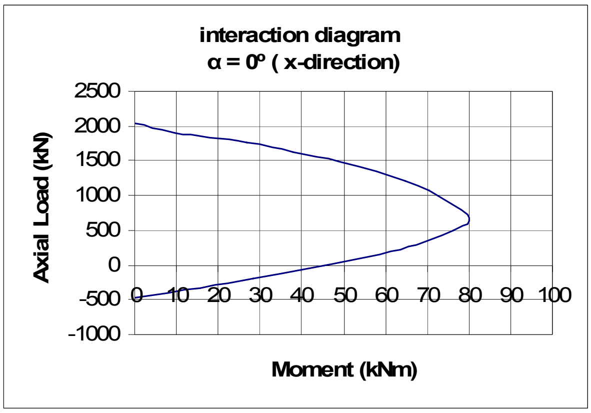


Figure 2.6 Interaction Diagram for α = 0º

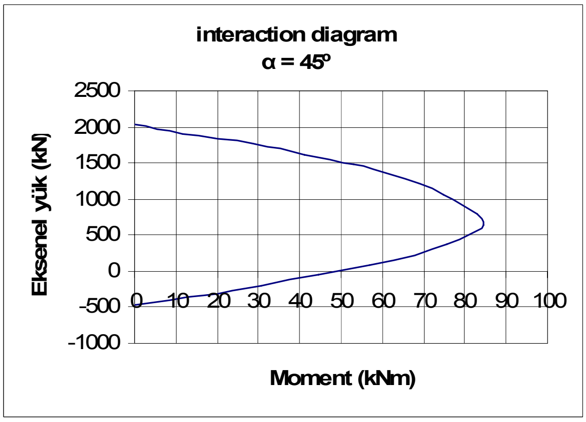


Figure 2.7 Interaction Diagram for α = 45º

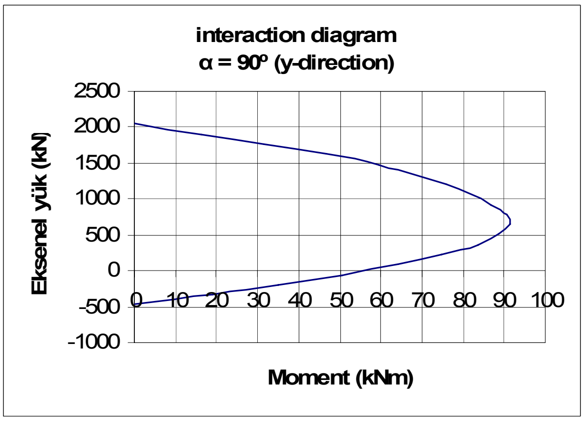


Figure 2.8 Interaction Diagram for α = 90º

where:

M_{uxo} = uniaxial flexural strength about x-axis

M_{uyo} = uniaxial flexural strength about y-axis

M_{ux} = component of biaxial flexural strength on the x-axis at required inclination

M_{uy} = component of biaxial flexural strength on the y-axis at required inclination

β = parameter dictating the shape of interaction surface

β = 0.7 for ground, first and 2nd floor columns

β = 0.6 for 3rd and 4th floor columns

2. There are three types of hinge properties in SAP2000. They are default hinge properties, user-defined hinge properties and generated hinge properties. Only default hinge properties and user-defined hinge properties can be assigned to frame elements.

Default hinge properties can not be modified because the default properties are section dependent. The default properties can not be fully defined by the program until the section that they apply to is identified. Thus to see the effect of the default properties, the default property should be assigned to a frame element, and then the resulting generated hinge property should be viewed. The built-in default hinge properties are typically based on FEMA-273 and/or ATC-40 criteria.

User-defined hinge properties can either be based on default properties or they can be fully user-defined. When user-defined properties are based on default properties, the hinge properties can not be modified because, again, the default properties are section dependent. When user-defined properties are not based on default properties, then the properties can be modified.

User defined hinge properties, which are not based on default properties are utilized in this study since the interaction diagrams as well as the moment - rotation relationships are defined for each member end separately from moment – curvature relations. For calculating moment – curvature relations and interaction diagrams, concrete and steel strengths are obtained from in –situ testing. Material safety factors are not applied to the in-situ strengths.

For beams, M3 hinge property is used which includes only moment – rotation relations whereas for columns PMM hinge property is used which includes both moment – rotation relations and interaction diagrams.

3. Moment – curvature relations are obtained from the concrete and steel stress – strain diagrams proposed by Mander Model presented in Appendix [6] and then bi-linearized to estimate $Ø_y$ (yield curvature), $Ø_u$ (ultimate curvature), M_y (yield moment capacity) and M_u (ultimate moment capacity).

For bi-linearizing moment–curvature curves, a method described by Priestley is employed [8].

Thus, the yield curvature is given by:

$$\phi_y = \phi_{y'} * (M_n / M_y) \tag{2.8}$$

where:

M_y: the moment at the point that the section first attains the reinforcement tensile yield strain ($\varepsilon_y = f_y / E_s$) or the concrete extreme compression fibre attains a strain of 0.002, whichever occur first.

$\phi_{y'}$: the curvature at the point that the section first attains the reinforcement tensile yield strain ($\varepsilon_y = f_y / E_s$) or the concrete extreme compression fibre attains a strain of 0.002, whichever occur first.

M_n: the moment at the point that the reinforcement tension strain reaches 0.015 or the concrete extreme compression fibre strain reaches 0.004, whichever occur first.

The point (ϕ_y, M_n) is the yield point of moment – curvature curve and (ϕ_u, M_u) is the ultimate point of moment – curvature curve, where ϕ_u is ultimate curvature and M_u is ultimate moment and both of them are calculated from moment – curvature analysis. (Figure 2.9)

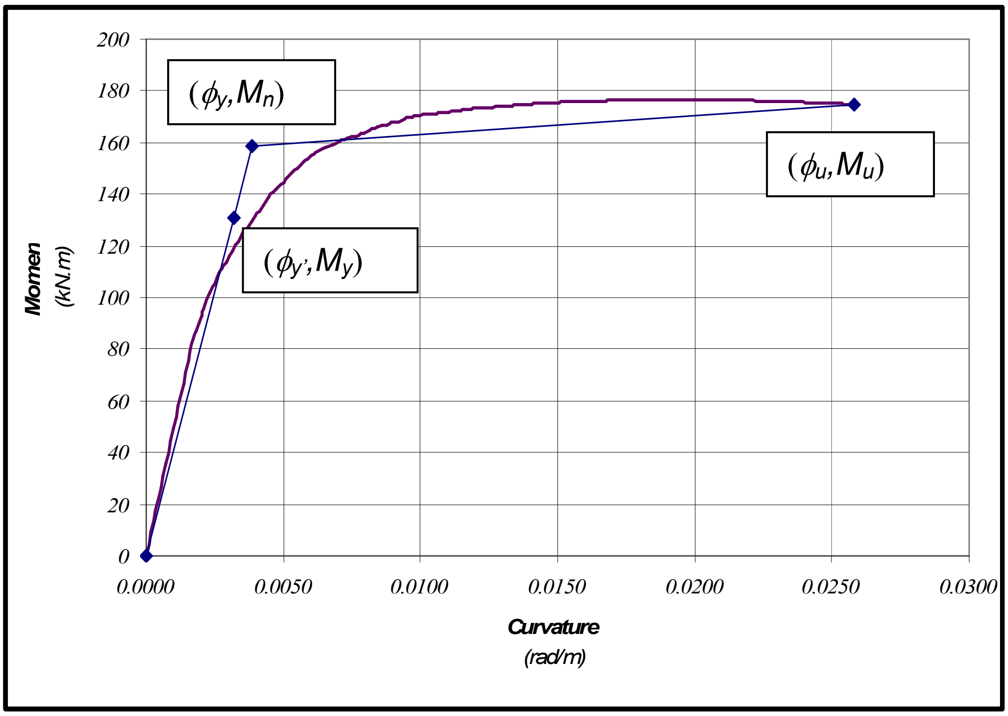


Figure 2.9 Bi-linearization of Moment – Curvature Curves

4. The moment – curvature relations are converted into moment – rotation relations by the following equations:

$$\theta_y = (\phi_y * L_n) / 6 \tag{2.9}$$

$$\theta_u = (\phi_u - \phi_y) * L_p + \theta_y \tag{2.10}$$

where:

ϕ_y = yield curvature of moment -curvature relation

ϕ_u = ultimate curvature of moment -curvature relation

L_p = plastic hinge length of section

θ_y= yield rotation

θ_u = ultimate rotation

L_n = clear length of the member

In this study, plastic hinge length is taken as half of the cross-section depth (L_p = h / 2)

5. Stiffnesses of concrete members are defined by considering cracked concrete sections. The cracked section stiffnesses of beams are taken as 40% of uncracked section stiffnesses and cracked section stiffnesses of columns and shearwalls are calculated according to their axial load level:

if $N_d/(A_c*f_{cm}) \leq 0.1$, then 0.40 of uncracked stiffness is taken as cracked stiffness,

if $N_d/(A_c*f_{cm}) \geq 0.4$, then 0.80 of uncracked stiffness is taken as cracked stiffness,

For the values of $N_d/(A_c*f_{cm})$ between 0.1 and 0.4, interpolation is done to calculate cracked stiffness.

N_d is the axial load value under gravity loading, A_c is the concrete section area, f_{cm} is the concrete compressive strength.

6. For each orthogonal direction, two separate 3-D model is constructed in SAP2000 since moment – rotation relations are different in each direction. If the pushover analysis is performed in x- direction, the moment curvature (M–ϕ) curves of members for that direction are defined separately and inserted into SAP2000. If the pushover analysis is performed in y- direction, the moment curvature (M–ϕ) curves of members for that direction are defined separately and inserted into SAP2000.

2.2.2 Calculating the Capacity Curve Of the Structure

Pushover analysis is an analysis method used to determine the nonlinear static response of a building under incremental lateral forces.

In short, pushover analysis algorithm can be summarized as follows:

Gravity forces on the structure are applied initially. The modal lateral forces at the joints in the loading direction are increased until one end of one member reaches its moment capacity. The structure behaves in the linear elastic range until this point.

When one end of a member reaches its moment capacity, first yield occurs. Then the structure is pushed incrementally, a plastic hinge is placed at this end such that the stiffness of that end is reduced.

The modal lateral load is increased incrementally, until the other member ends reach their capacities. The stiffnesses of these ends are reduced accordingly.

After yielding of each member end, the modal lateral load is increased incrementally.

The above procedure is repeated until a mechanism is formed in the structure.

If the magnitude of the incremental force is the control parameter, the analysis is called force-controlled pushover. However, if the lateral displacement of a control node is the control parameter, the analysis is called displacement-controlled pushover.

Before yield point, structure behaves in force-controlled manner. However, after yield, structure behaves in displacement-controlled manner and a little increase in force causes a significant increase in displacement. In this study, displacement-controlled pushover is used, and control node is selected as the mass center of top storey. In both type of pushover analysis, when one member end yields, its stiffness is modified after an increment.

The lateral force vector applied in pushover analysis has a modal load distribution. This modal lateral load is calculated for both orthogonal directions separately, by multiplying mode shape of corresponding direction and mass at each floor level, and applied to the mass center of each floor.

2.2.3 Calculating Displacement Demand of the Structure

The displacement demand of the structure is calculated by the following procedure and called as target displacement for selected performance level.

The method described in the 2006 Turkish Seismic Rehabilitation Code is applied to selected buildings for determining the target displacement.

1. Linear elastic spectral displacement S_{de} is calculated for the dominant period of corresponding direction.

$$S_{de} = S_{ae} / \omega^2 \tag{2.11}$$

S_{ae} is the elastic spectral acceleration of the corresponding period,
ω is the frequency of corresponding period.

2. If the period of corresponding direction (T) is greater than the characteristic period of acceleration spectrum (T_B), inelastic spectral displacement is assumed equal to elastic spectral displacement. (Figure 2.10)

$$S_{die} = S_{de} \tag{2.12}$$

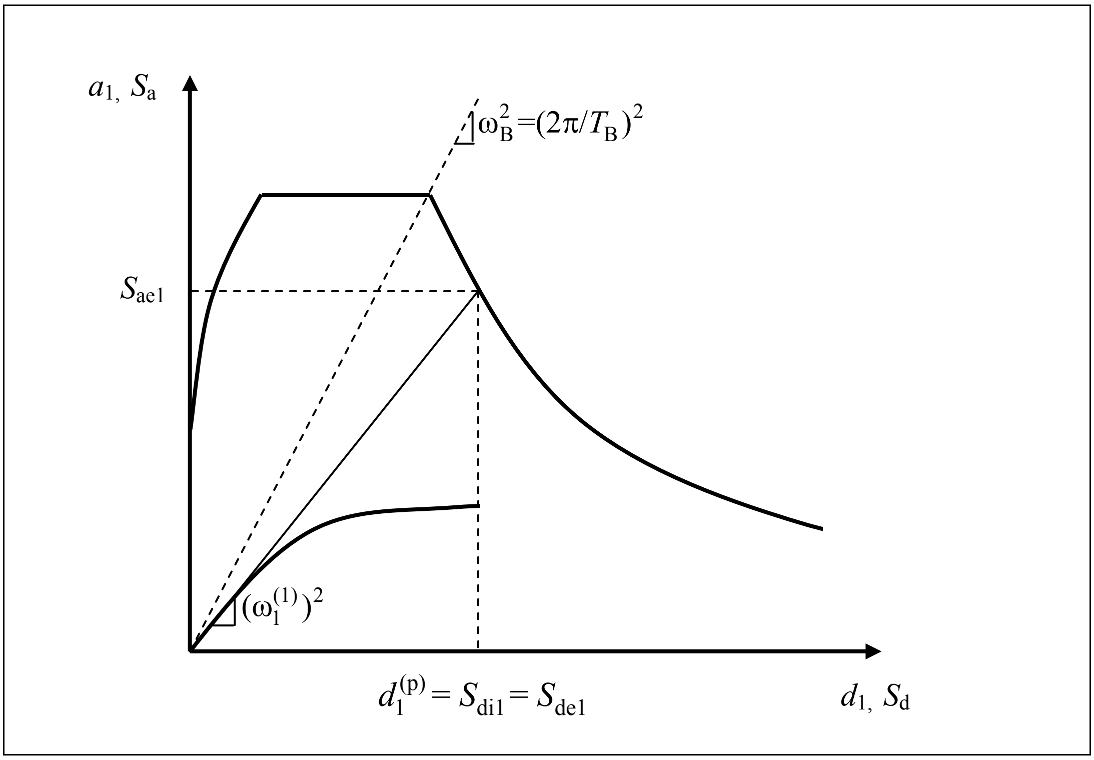


Figure 2.10 Capacity and Demand Curve for T ≥ T_B

3. Otherwise, inelastic spectral displacement is calculated by the following equation :

$$S_{die1} = C_{R1} * S_{de} \tag{2.13}$$

For first iteration, C_{R1} is taken as 1, for second iteration C_{R1} is calculated as follows:

$$C_{R1} = \frac{1 + (R_{y1} - 1)T_B/T_1}{R_{y1}} \geq 1 \tag{2.14}$$

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{2.15}$$

4. For first iteration a^{o}_{y1} is calculated as shown in Figure 2.11(a), by equal area principle, i.e. the area under the capacity curve and over the capacity curve are equal. For other iterations, a_{y1} is calculated from Figure 2.11(b) again by equal area principle, as well as R_{y1} and C_{r1} values.

5. The iterations are repeated until obtaining close results between iterations.

6. From last iteration, inelastic spectral demand is obtained (S_{die}) and converted to the inelastic displacement demand. This inelastic demand is called as target displacement.

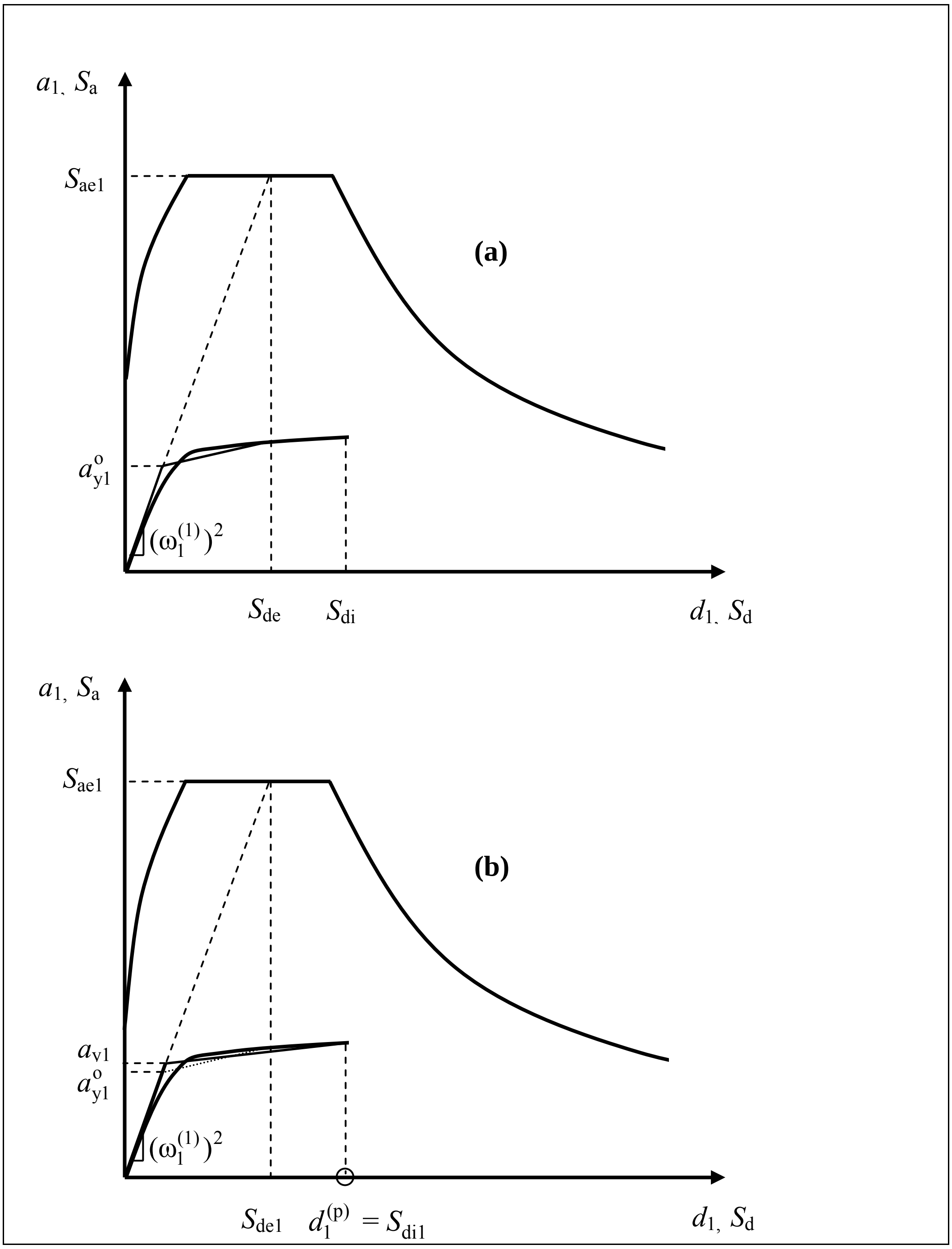


Figure 2.11 Calculating a_{y1}^{o} and a_{y1}

2.2.4 Acceptability of Members in Nonlinear Procedure

The performance assessment of members includes following steps :

1. From the pushover analysis, the plastic rotation of each end of members are obtained at the target displacement.

2. From the plastic rotations, plastic curvatures are obtained :

$$\phi_p = \frac{\theta_p}{L_p} \tag{2.16}$$

Then, the plastic curvature is added with yield curvature in order to obtain the total curvature :

$$\phi_t = \phi_p + \phi_y \tag{2.17}$$

At this total curvature, corresponding concrete and steel strains are calculated from the section moment – curvature relations, and compared to the concrete and steel strain limits associated with the target performance level.

3. The concrete and steel strain limits for target performance levels are defined as :

- For immediate occupancy performance level (MN):

$$(\varepsilon_{cu})_{MN} = 0.004 \quad ; \quad (\varepsilon_s)_{MN} = 0.010 \tag{2.18}$$

where:

ε_{cu} is the concrete outermost fiber strain

ε_s is the tensile steel strain

- For life safety performance level (GV) :

$$(\varepsilon_{cg})_{GV} = 0.004 + 0.0095(\rho_s / \rho_{sm}) \leq 0.0135 \quad ; \quad (\varepsilon_s)_{GV} = 0.040 \qquad (2.19)$$

where:

ε_{cg} is the concrete clear cover fiber strain

ε_s is the tensile steel strain

ρ_s is the existing volumetric ratio of the transverse reinforcement

ρ_{sm} is the minimum design volumetric ratio of the transverse reinforcement

- For collapse prevention performance level (GC):

$$(\varepsilon_{cg})_{GC} = 0.004 + 0.013(\rho_s / \rho_{sm}) \leq 0.018 \quad ; \quad (\varepsilon_s)_{GC} = 0.060 \qquad (2.20)$$

where:

ε_{cg} is the concrete clear cover fiber strain

ε_s is the tensile steel strain

ρ_s is the existing volumetric ratio of the transverse reinforcement

ρ_{sm} is the minimum design volumetric ratio of the transverse reinforcement

The above performance levels are indicated in Figure 2.12:

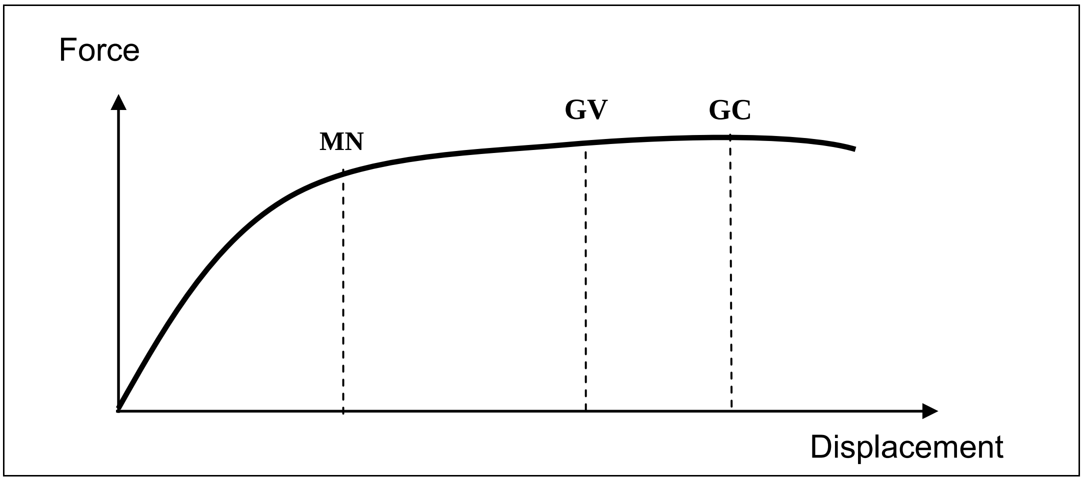


Figure 2.12 Target Performance Levels

Acceptability of members depend on the ratio of calculated strains to strain limits.

If (Strain / Strain Limit) < 1, then the corresponding end of member is acceptable for the target performance level.

If (Strain / Strain Limit) > 1, then the corresponding end of member is not acceptable for the target performance level.

If both ends of a member is acceptable, then the member is acceptable. However if one of the ends is not acceptable, member will be not acceptable for the selected target performance level.

2.3 Estimation of Building Performances

As mentioned above, the acceptance criteria of 2006 Turkish Earthquake Code is different for immediate occupancy performance level, life safety performance level and collapse prevention performance level. According to the acceptance criteria for these performance levels, it is also estimated whether the building should be retrofitted or not:

Immediate Occupancy Performance Level:

At each storey, in the considered direction, immediate occupancy limits for beams may be exceeded in 10% of the beams at most. However, these limits should be satisfied by all other vertical members. In this situation, the building is accepted for Immediate Occupancy Performance Level, and any retrofit is not needed.

Life Safety Performance Level:

At each storey, in the considered direction, life safety limits for beams may be exceeded in 20% of the beams at most. In addition, again at each storey, in the considered direction, the shear forces taken by the columns, which can not satisfy life safety limits, should be 20% of storey shear force at most. These limits should be satisfied by all other vertical members. In this situation, the building is accepted for Life Safety Performance Level, and retrofit is considered according to number and location of unacceptable elements.

Collapse Prevention Performance Level:

At each storey, in the considered direction, collapse prevention limits for beams may be exceeded in 20% of the beams at most. In addition, again at each storey, in the considered direction, the shear forces taken by the columns, which can not satisfy collapse prevention limits, should be 20% of storey shear force at most. These limits should be satisfied by all other vertical members. In this situation, the building is accepted for Collapse

Prevention Performance Level, and retrofit is needed. However, retrofit should be discussed whether it will be economical or not.

Collapse Performance Level:

If any building can not satisfy Collapse Prevention Performance Level, then the building will be at Collapse Performance Level, and retrofit is needed. However, retrofit should be discussed whether it will be economical or not, since it may not be economical.

The displacement limits for each performance level should also be satisfied by each storey of the building. These limits are given in Table 2.4

Table 2.4 Storey Displacement Limits for Different Performance Levels

Storey Displacement Limits	**Performance Level**		
	Immediate Occupancy	**Life Safety**	**Collapse Prevention**
$(\delta_i)_{max}/h_i$	0.008	0.02	0.03

$(\delta_i)_{max}/h_i$ is the maximum relative displacement between the adjacent stories divided by the storey height.

CHAPTER III

CASE STUDY 1

ASSESSMENT OF A RESIDENTIAL BUILDING DESIGNED TO 1998 TURKISH EARTHQUAKE CODE

In this case study, a four storey residential building designed according to 1998 Turkish Earthquake Code [10] is assessed in detail. This building is studied in order to verify the consistency of analysis and design procedures in the code.

The building consists of a moment resisting frame system. The frame system is made up of 7 axes in the short direction (X-direction) and symmetrical, whereas it is made up of 4 axes in the long direction (Y-direction) and not symmetrical. Each storey is 2.7m in height and has 287.8m² floor area. Each storey has the same structural plan. Typical storey structural plan view (Figure 3.1) and the member dimensions (Figure 3.2) are given below. Slab thicknesses are 15cm. Storey masses, mass center coordinates and mass moment of inertias are given below (Table 3.1)

Table 3.1 Storey Masses, Mass Center Coordinates and Mass Moment of Inertias

Storey	**Mass (t)**	**Mass Center Coordinates**		**Mass Moment of Inertias ($t.m^2$)**
		X (m)	**Y (m)**	
1	338.54	6.07	11.55	22947
2	338.54	6.07	11.55	22947
3	338.54	6.07	11.55	22947
4	260.78	6.00	11.50	17377

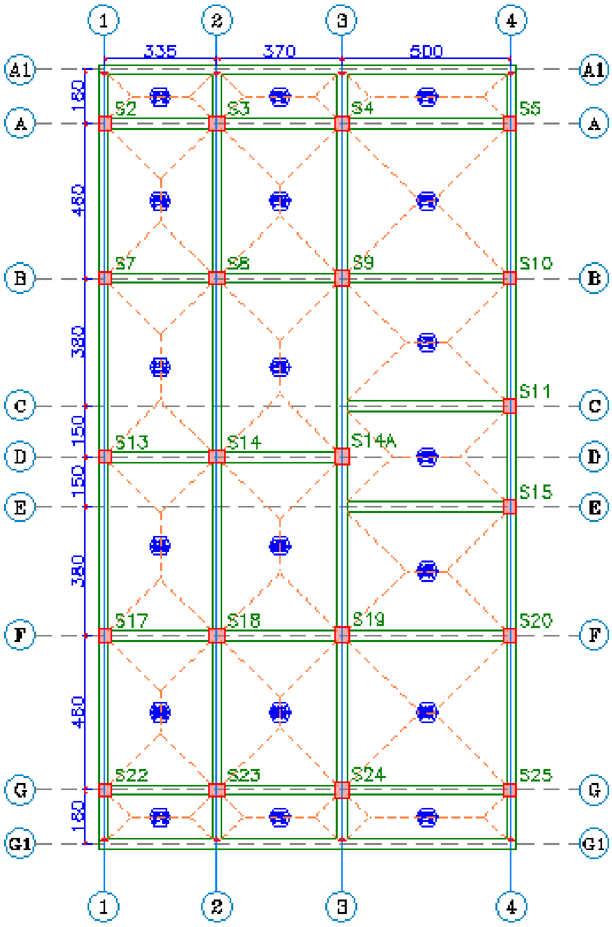


Figure 3.1 Typical Structural Plan View

Column	bx (cm)	by (cm)
S2	40	40
S3	45	40
S4	40	45
S5	40	40
S7	40	40
S8	45	40
S9	45	45
S10	40	40
S11	40	40
S13	40	40
S14	45	40
S14A	40	50
S15	40	40
S17	40	40
S18	45	40
S19	45	45
S20	40	40
S22	40	40
S23	45	40
S24	40	45
S25	40	40

Figure 3.2 Column Dimensions

Beam and column dimensions are same in each storey

All beams are 30cmx50cm

The project and code parameters of existing building are given in Table 3.2 below. In this table, knowledge level and knowledge level factor are estimated according to the existence of the project of the building.

Table 3.2 Project and Code Parameters of the Building

Project Parameters of the Existing Building	**1998 Turkish Earthquake Code Parameters**
Project of the Building (Exist / Not exist): *Exist*	Earthquake Zone: *1*
Knowledge Level: *High*	Earthquake Zone Factor: *0.4*
Knowledge Level Factor: *1*	Building Importance Factor: *1.0*
Reinforcement Existing Factor: *1*	Soil Class: *Z3*
Existing Concrete Strength (Mean – Standard Deviation): *25 MPa*	
Existing Steel Strength (Mean–Standard Deviation): *420 MPa*	
Target Performance Level: *Life Safety (At 50 years %10)*	
Live Load Participation Factor (n): *0.30*	

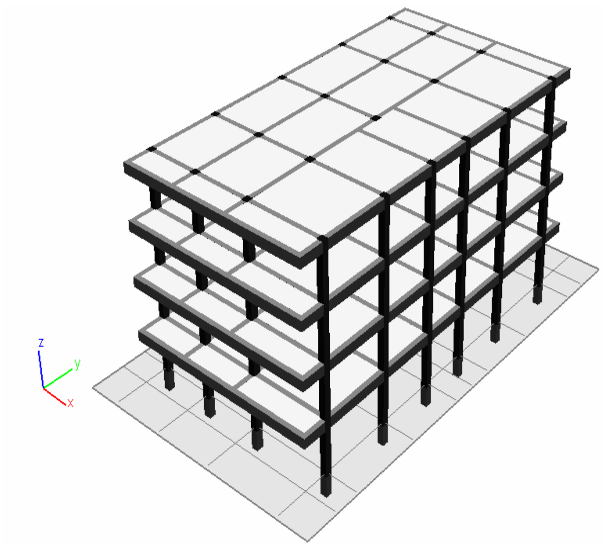


Figure 3.3 Three Dimensional Model

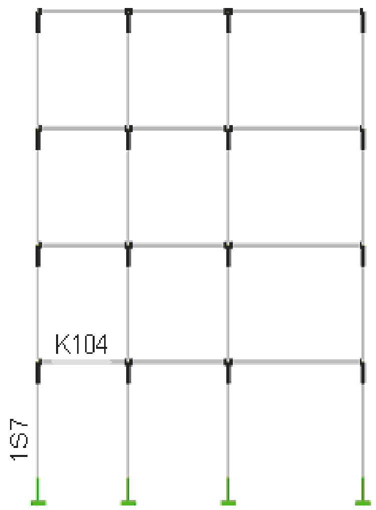


Figure 3.4 Frame B

(Rigid end zones, Example column and beam)

3.1 Linear Elastic Procedure

The building is first assessed by linear elastic procedure. This procedure will be explained for the next case study in detail. So that, for this case study, the results will be summarized.

3.1.1 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})

The ratios of r to r_{Limit} values are calculated for all member ends and presented in bar chart form for beams and columns separately in each storey.

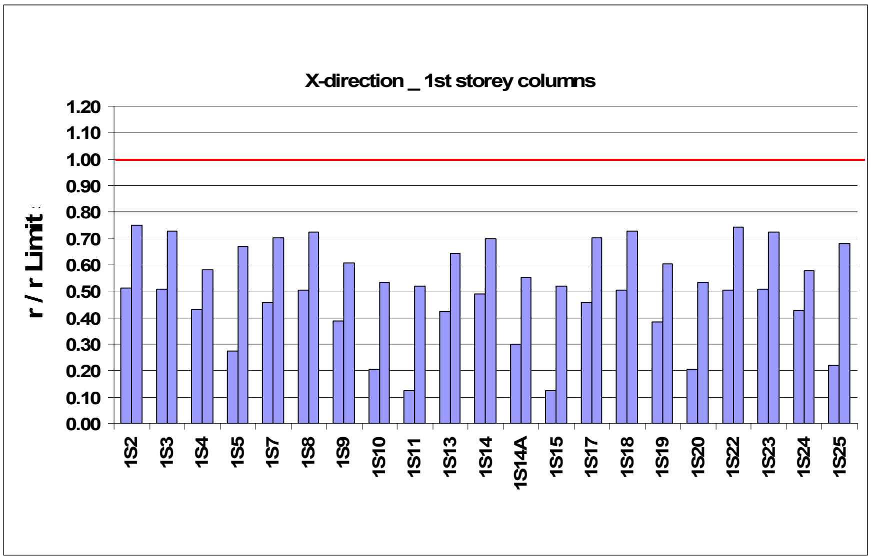


Figure 3.5 r / r_{Limit} for 1^{st} Storey Columns

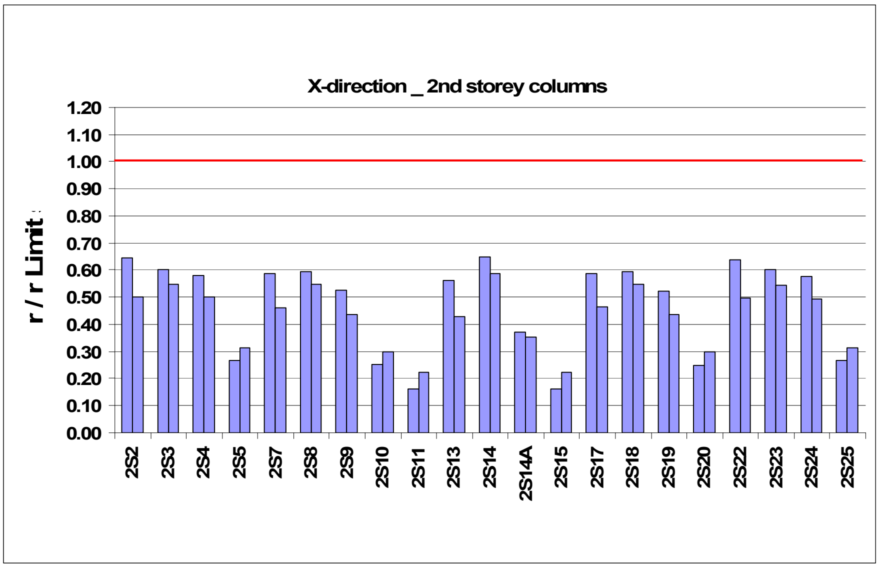


Figure 3.6 r / r_{Limit} for 2nd Storey Columns

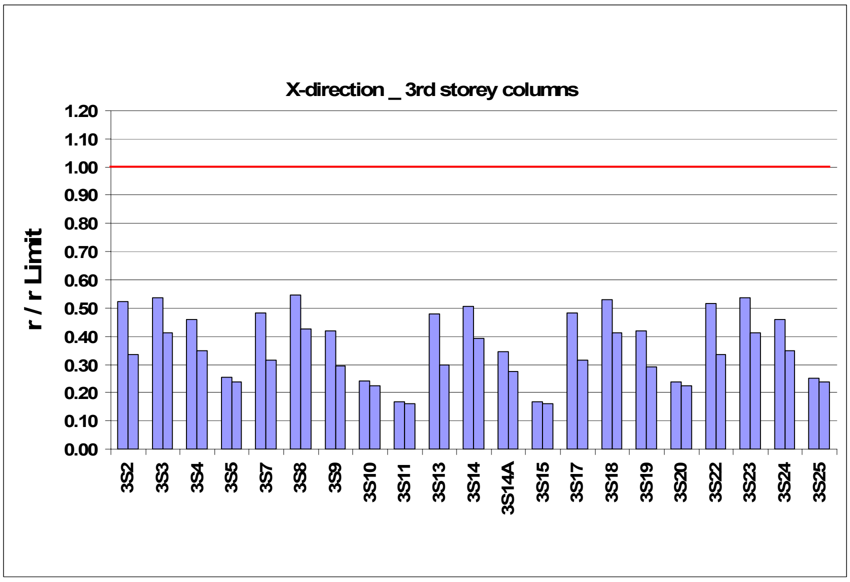


Figure 3.7 r / r_{Limit} for 3rd Storey Columns

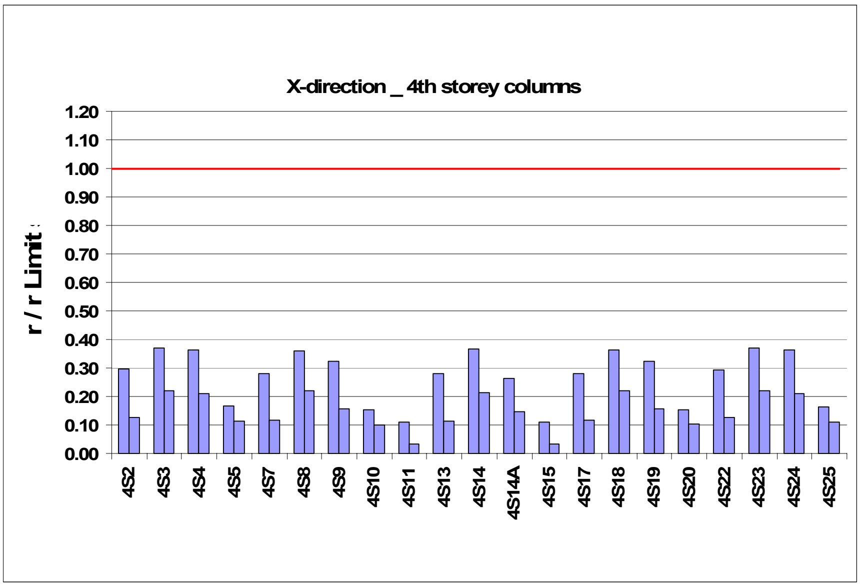


Figure 3.8 r / r_{Limit} for 4^{th} Storey Columns

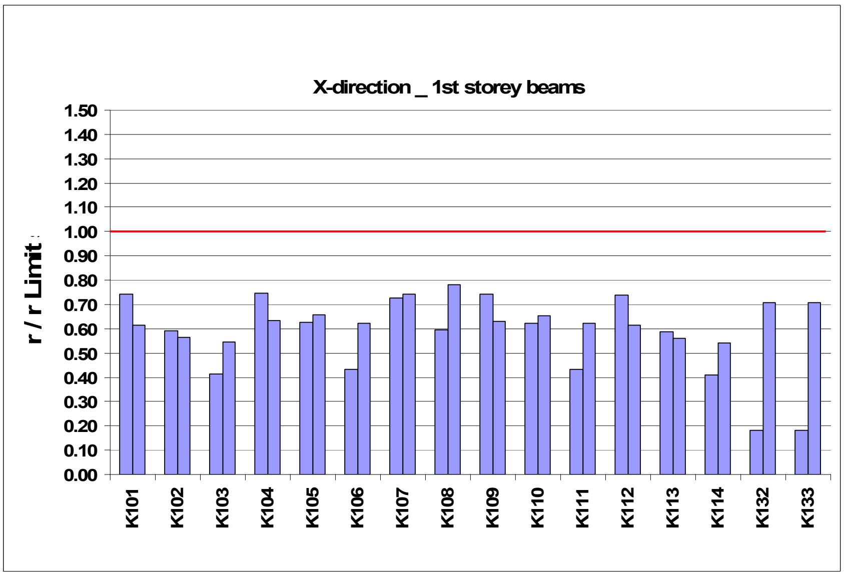


Figure 3.9 r / r_{Limit} for 1^{st} Storey Beams

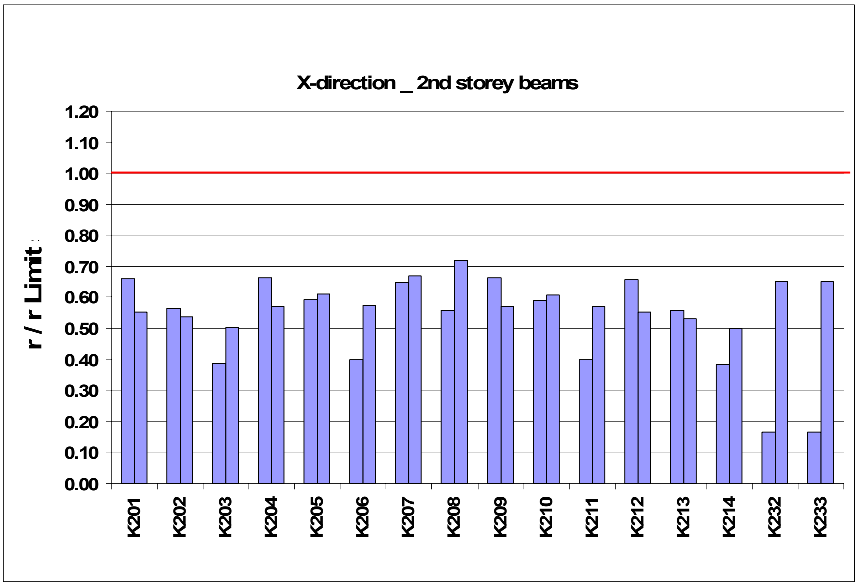


Figure 3.10 r / r_{Limit} for 2[nd] Storey Beams

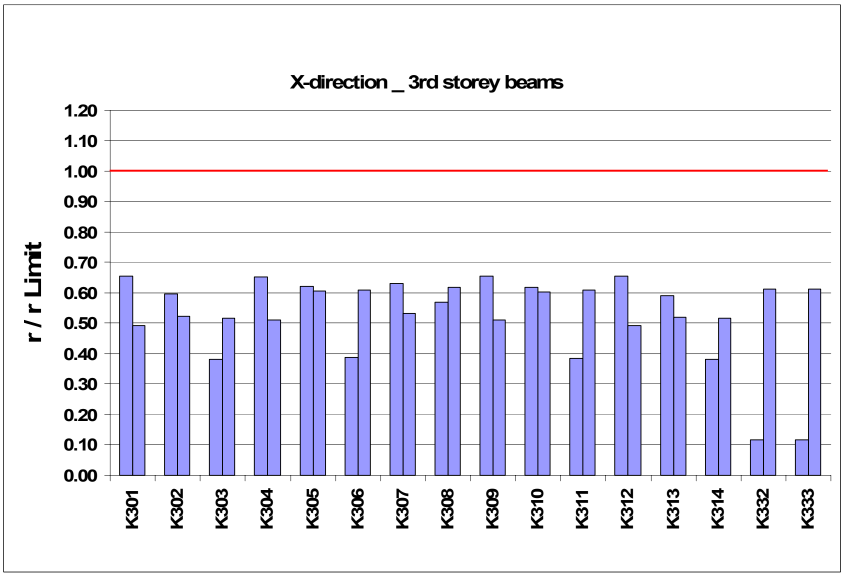


Figure 3.11 r / r_{Limit} for 3[rd] Storey Beams

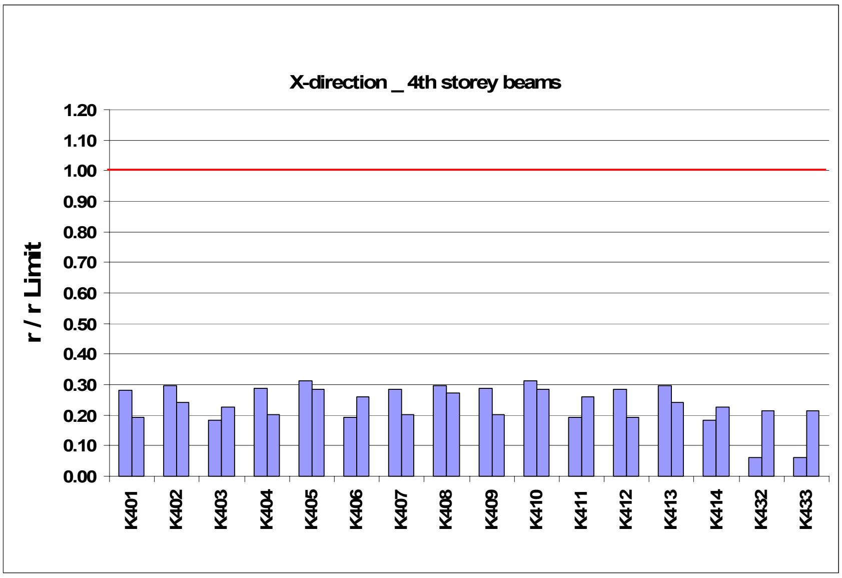


Figure 3.12 r / r_{Limit} for 4^{th} Storey Beams

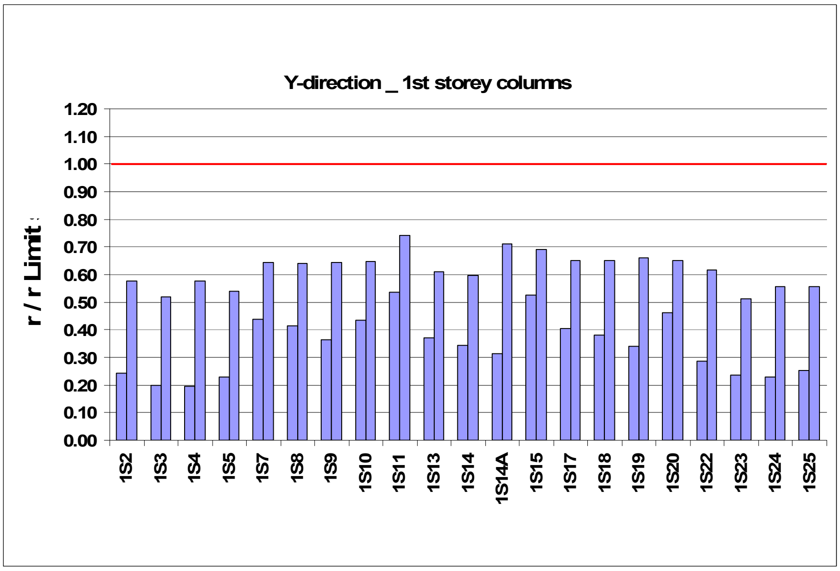


Figure 3.13 r / r_{Limit} for 1^{st} Storey Columns

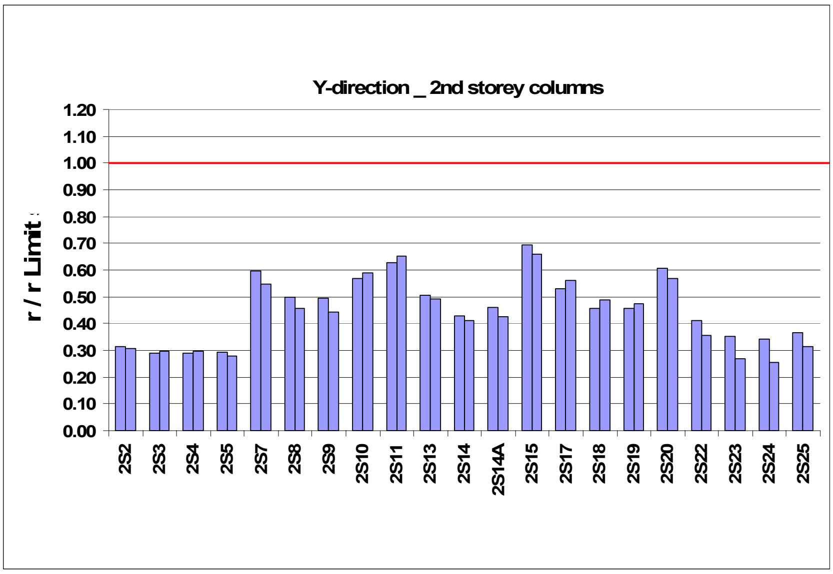


Figure 3.14 r / r_{Limit} for 2nd Storey Columns

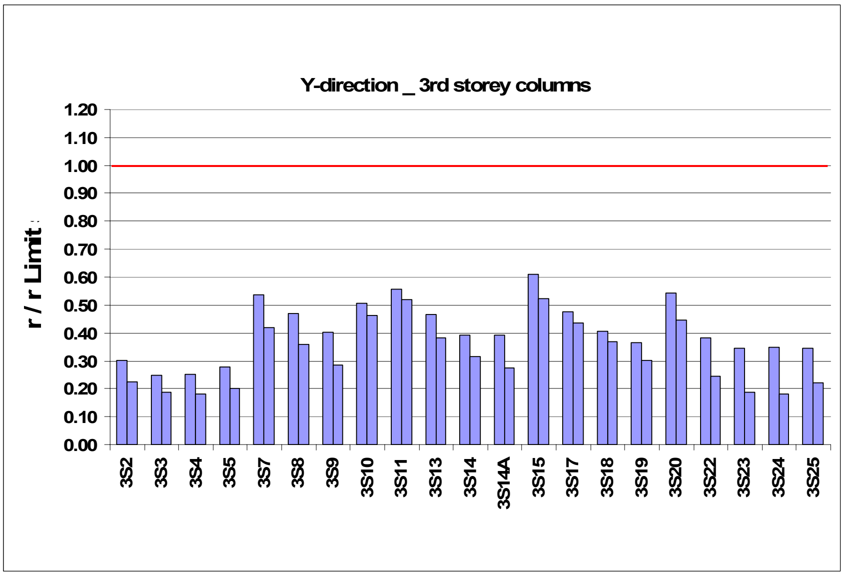


Figure 3.15 r / r_{Limit} for 3rd Storey Columns

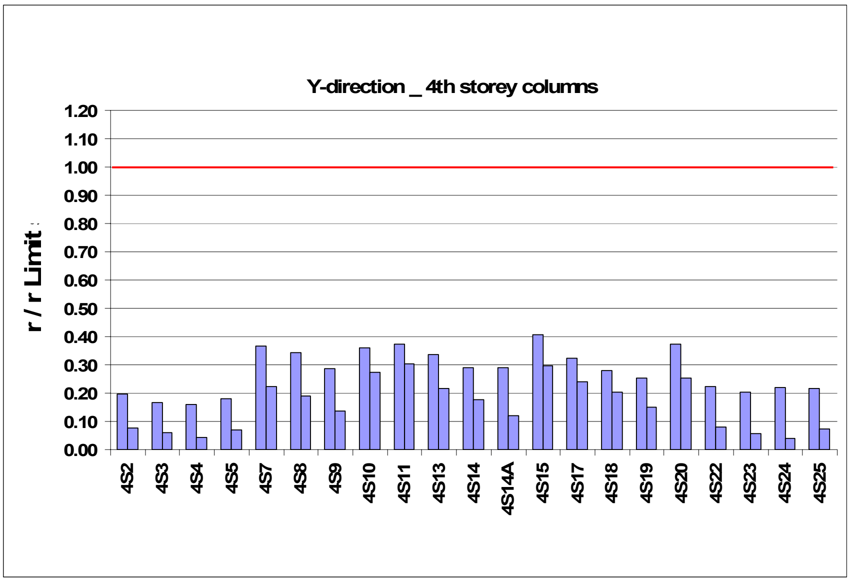


Figure 3.16 r / r_{Limit} for 4^{th} Storey Columns

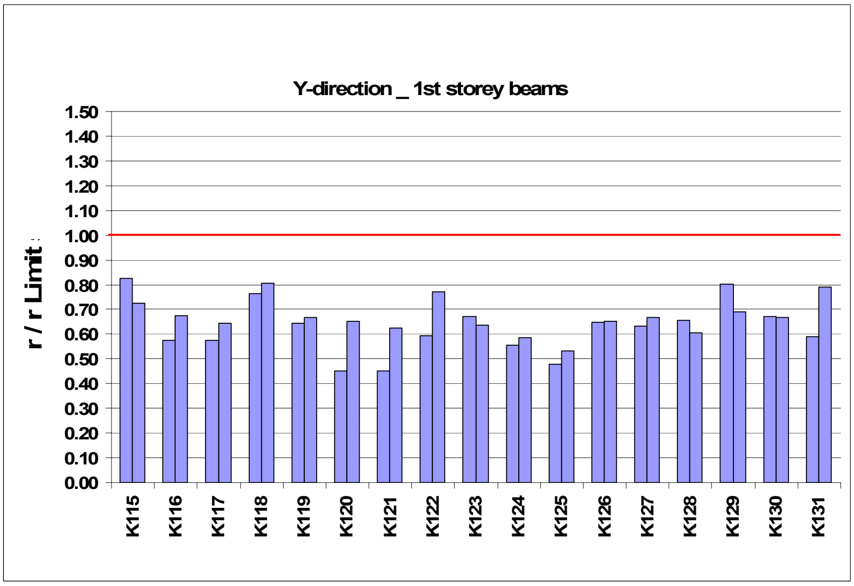


Figure 3.17 r / r_{Limit} for 1^{st} Storey Beams

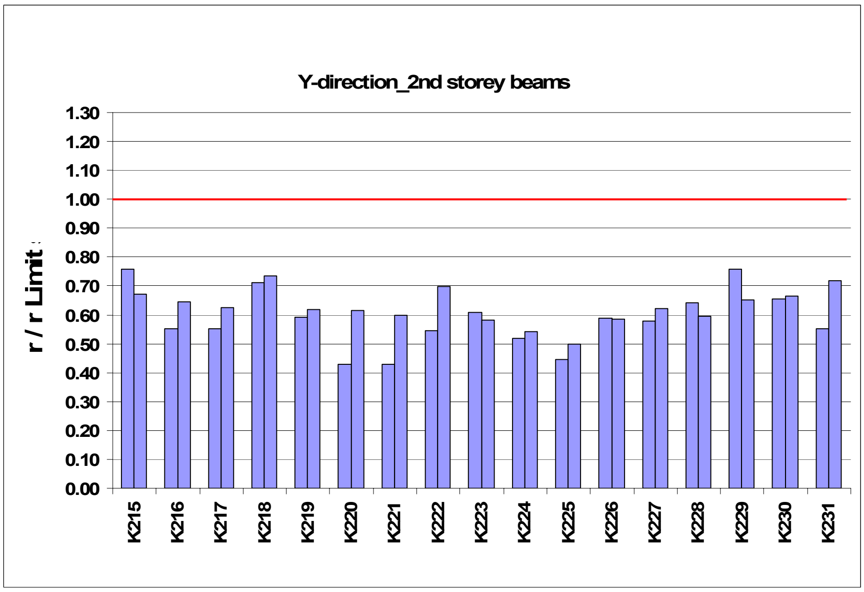


Figure 3.18 r / r_{Limit} for 2nd Storey Beams

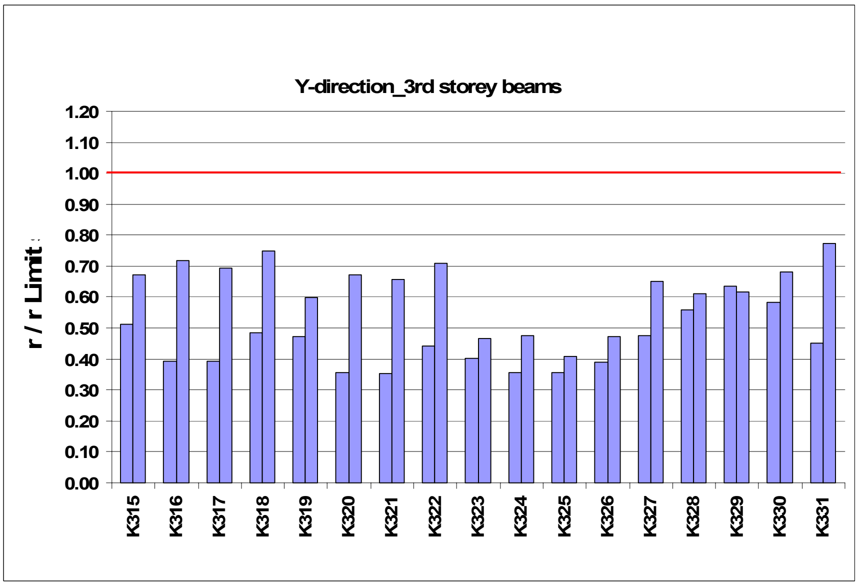


Figure 3.19 r / r_{Limit} for 3rd Storey Beams

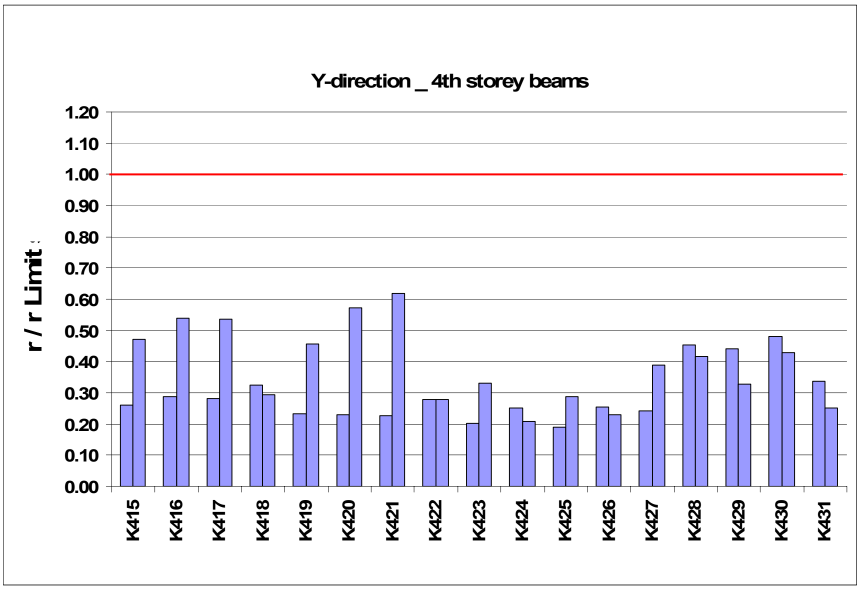


Figure 3.20 r / r_{Limit} for 4th Storey Beams

3.1.2 Global Performance of the Building

Global performance of the building is summarized in the tables below. These values are obtained from linear elastic analysis.

Table 3.3 Percentage of Unacceptable Beams and Columns

	+X Direction		+Y Direction	
Storey	**Beams (%)**	**Columns (%)**	**Beams (%)**	**Columns (%)**
1	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0

Table 3.4 Interstorey Drifts

	+X Direction			+Y Direction		
Storey	***H_i(m)***	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**	***H_i(m)***	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**
1	2.7	0.01341	0.005	2.7	0.01434	0.005
2	2.7	0.01699	0.006	2.7	0.0185	0.007
3	2.7	0.01335	0.005	2.7	0.01451	0.005
4	2.7	0.00765	0.003	2.7	0.00822	0.003

3.2 Non-Linear Analysis

The building is also assessed by non-linear procedures. These procedures will be explained for the next case study in detail. So that, for this case study, the results will be summarized.

3.2.1 Comparison of Column and Beam Section Strains with the Section Strain Limits

The ratios of ε to ε_{Limit} values are calculated for all member ends and presented in bar chart form for beams and columns separately in each storey.

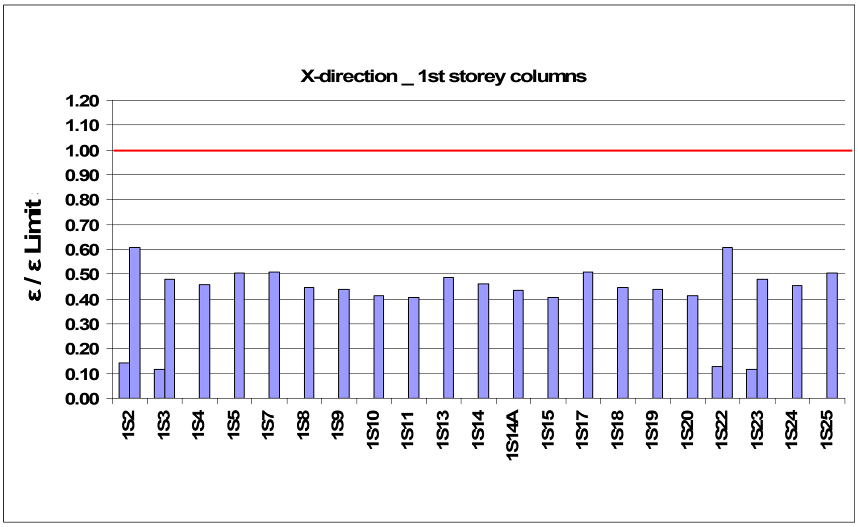


Figure 3.21 ε / ε_{Limit} for 1st Storey Columns

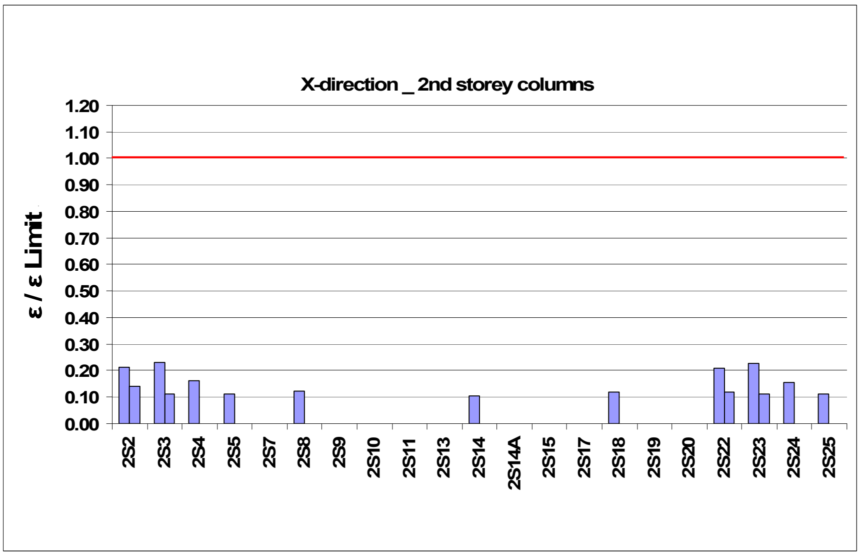


Figure 3.22 ε / ε_{Limit} for 2nd Storey Columns

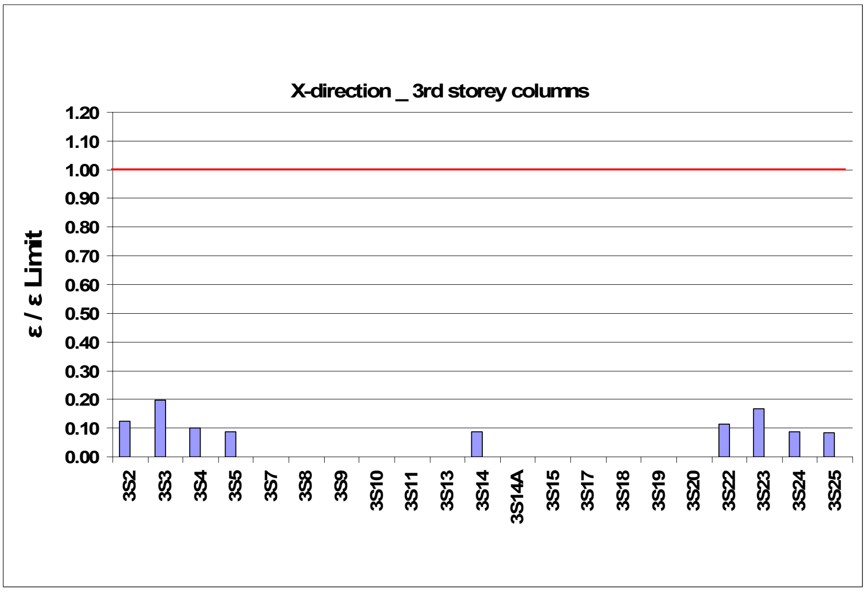


Figure 3.23 ε / ε_{Limit} for 3rd Storey Columns

In X – direction, all 4th storey columns satisfy the "**GV**" Limit State. Hence the graphics of 4th storey columns are not given.

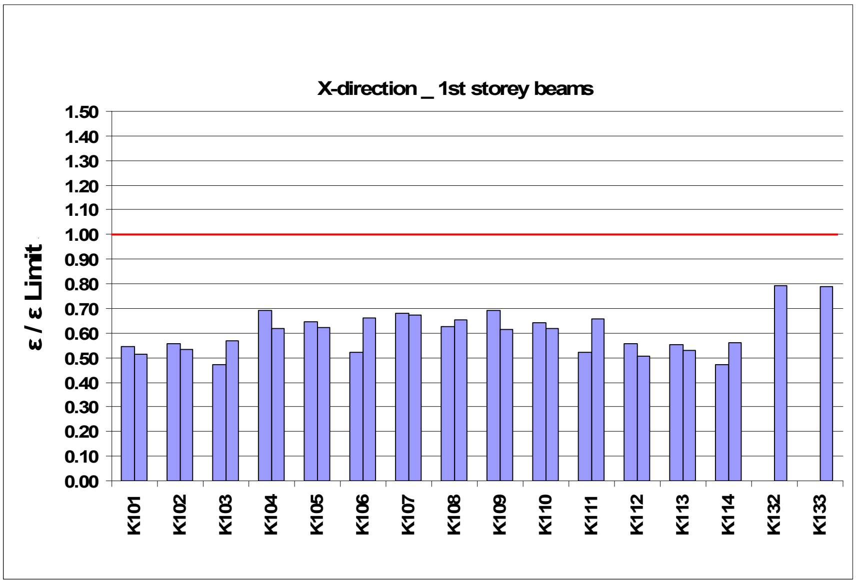


Figure 3.24 ε / ε_{Limit} for 1st Storey Beams

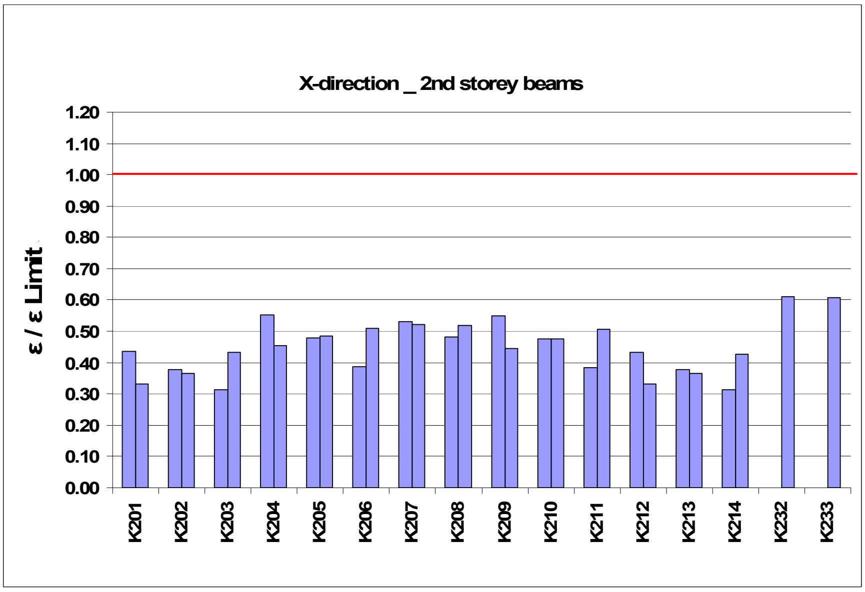


Figure 3.25 ε / ε_{Limit} for 2nd Storey Beams

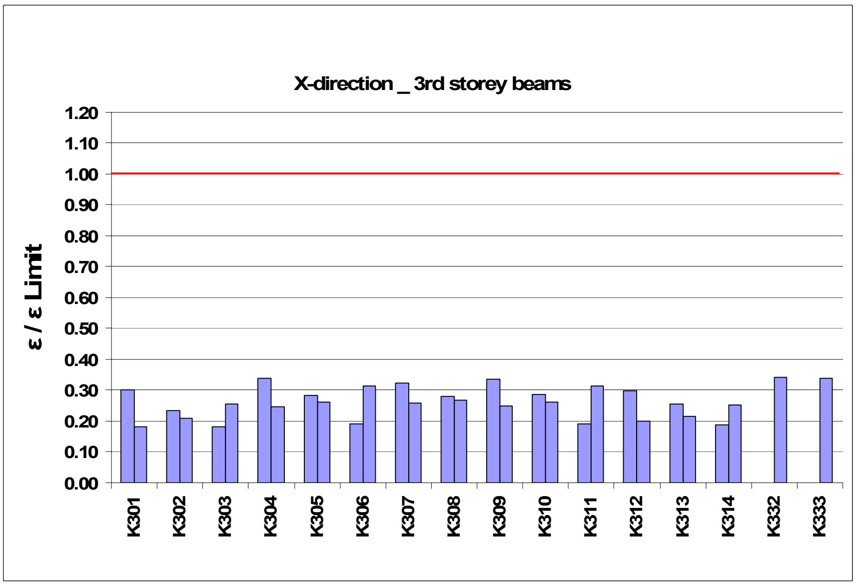


Figure 3.26 ε / ε_{Limit} for 3rd Storey Beams

In X – direction, all 4th storey beams satisfy the "**GV**" Limit State. Hence the graphics of 4th storey beams are not given.

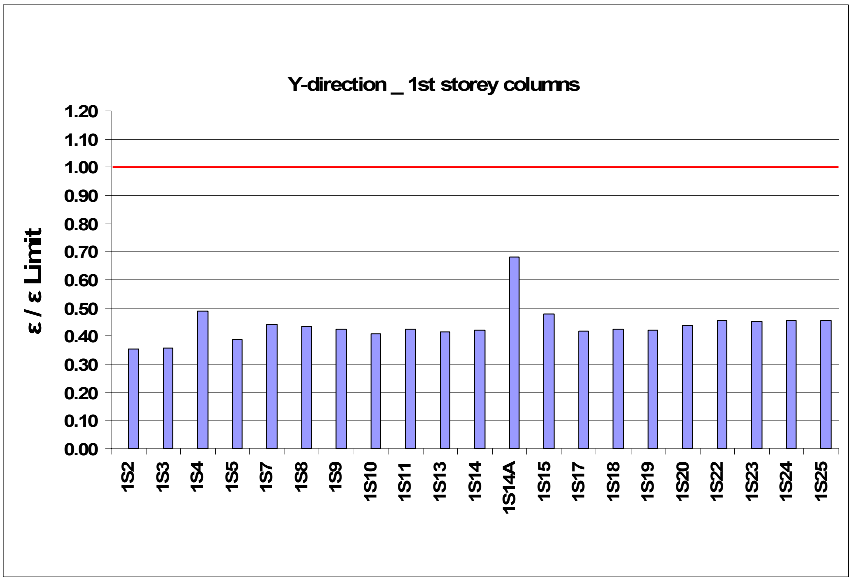


Figure 3.27 ε / ε_{Limit} for 1st Storey Columns

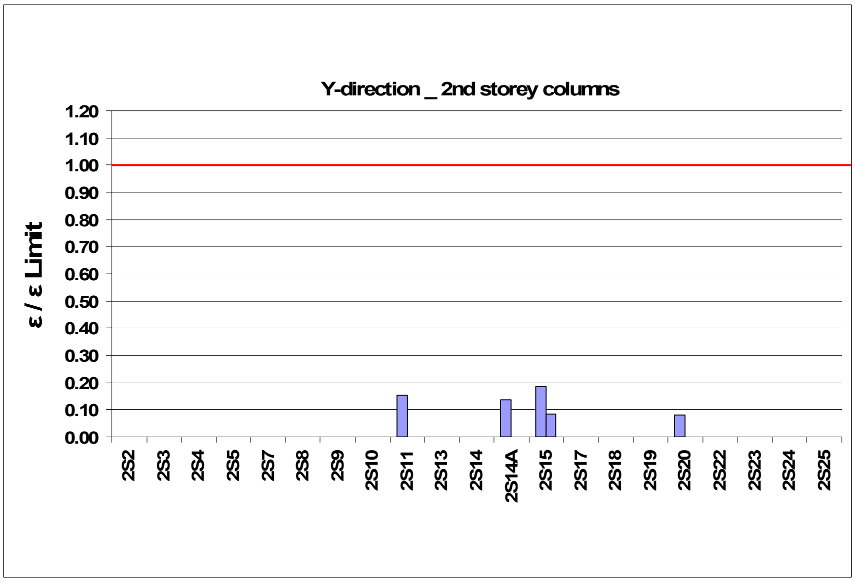


Figure 3.28 ε / ε_{Limit} for 2nd Storey Columns

In Y – direction, all 3rd and 4th storey columns satisfy the "**GV**" Limit State. Hence the graphics of 3rd and 4th storey columns are not given.

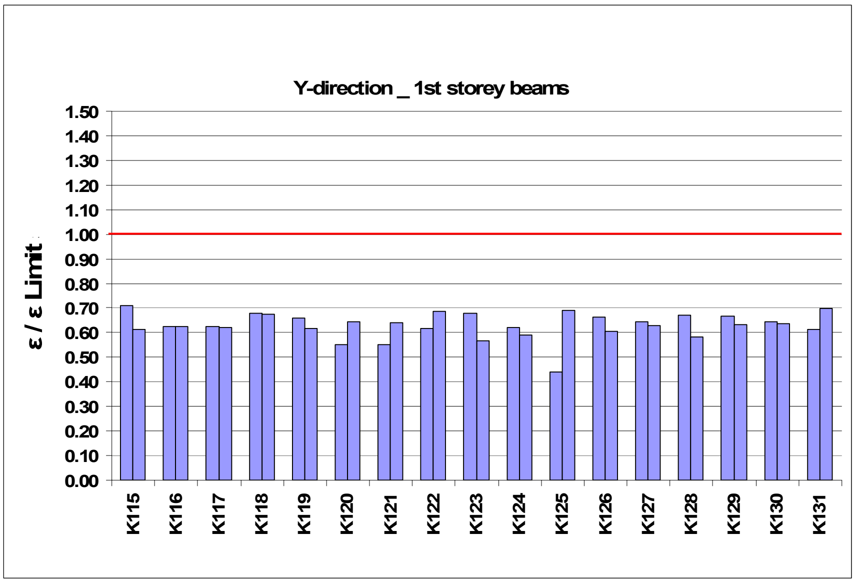


Figure 3.29 ε / ε_{Limit} for 1st Storey Beams

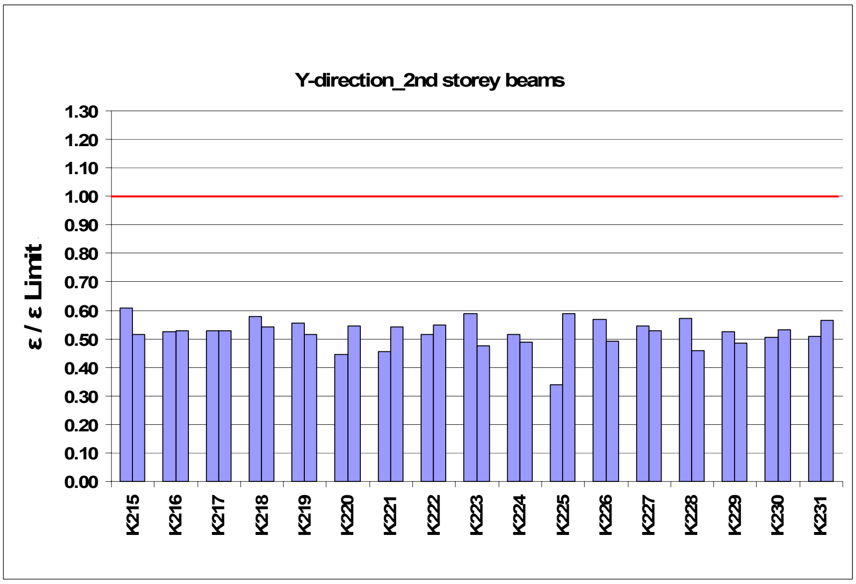


Figure 3.30 ε / ε_{Limit} for 2nd Storey Beams

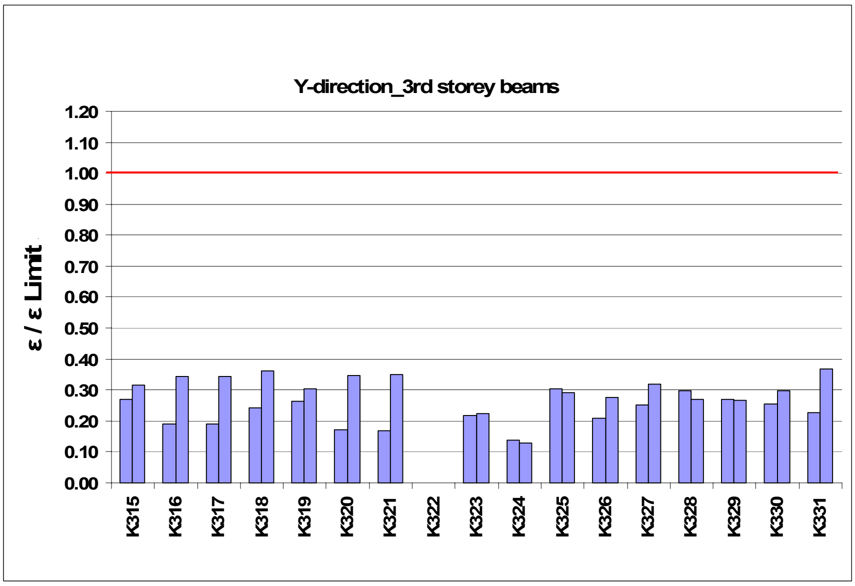


Figure 3.31 $\varepsilon / \varepsilon_{Limit}$ for 3rd Storey Beams

In Y – direction, all 4th storey beams satisfy the "**GV**" Limit State. Hence the graphics of 4th storey beams are not given.

3.2.2 Global Performance of the Building

Global performance of the building is summarized in the tables below. These values are obtained from nonlinear analysis.

Table 3.5 Percentage of Unacceptable Beams and Columns

	+X Direction		**+Y Direction**	
Storey	**Beams (%)**	**Columns (%)**	**Beams (%)**	**Columns (%)**
1	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0

Table 3.6 Interstorey Drifts

	+X Direction			**+Y Direction**		
Storey	$H_i(m)$	$(\Delta_i)_{max}$	$(\Delta_i)_{max}$ / H_i	$H_i(m)$	$(\Delta_i)_{max}$	$(\Delta_i)_{max}$ / H_i
1	2.7	0.03830	0.014	2.7	0.03872	0.014
2	2.7	0.03781	0.014	2.7	0.04067	0.015
3	2.7	0.02531	0.009	2.7	0.02809	0.010
4	2.7	0.01064	0.004	2.7	0.01113	0.004

CHAPTER IV

CASE STUDY 2
ASSESSMENT OF A RESIDENTIAL BUILDING DESIGNED TO 1975 TURKISH EARTHQUAKE CODE

In this case study, a four storey residential building designed according to 1975 Turkish Earthquake Code [11] is assessed in detail.

All calculations pertaining to the linear and nonlinear procedures are presented. The building consists of a moment resisting frame system. The frame system is made up of 7 axes in the short direction (X-direction) and symmetrical, whereas it is made up of 4 axes in the long direction (Y-direction) and not symmetrical. Each storey is 2.7m in height and has 287.8m² floor area. Each storey has the same structural plan. Typical storey structural plan view (Figure 4.1) and the member dimensions (Figure 4.2) are given below. Slab thicknesses are 12cm.

Storey masses, mass center coordinates and mass moment of inertias are given below (Table 4.1)

Table 4.1 Storey masses, mass center coordinates and mass moment of inertias

Storey	Mass (t)	Mass Center Coordinates		Mass Moment of Inertias ($t.m^2$)
		X (m)	Y (m)	
1	300.41	6.07	11.55	20357
2	300.41	6.07	11.55	20357
3	300.41	6.07	11.55	20357
4	222.79	6.00	11.50	14787

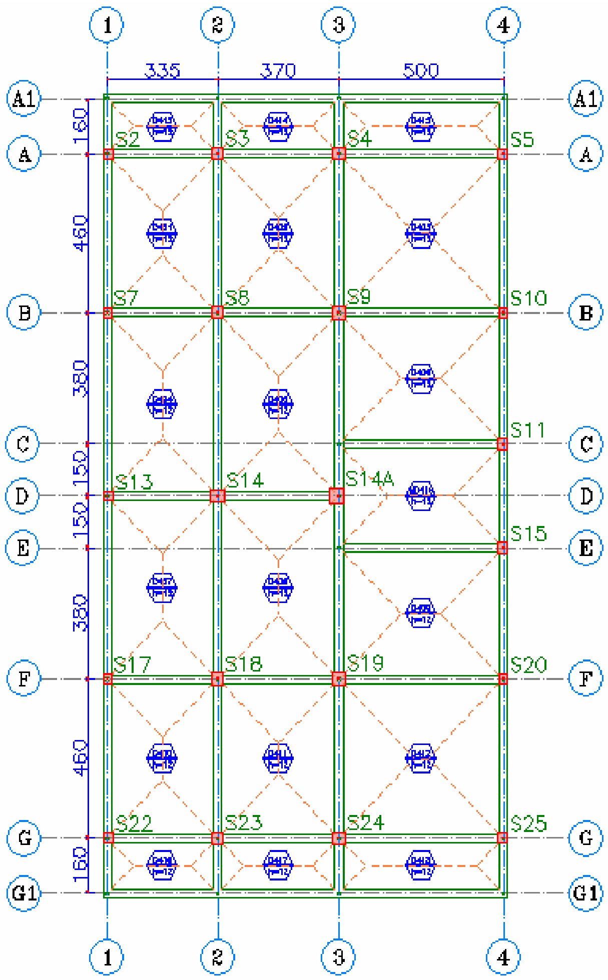


Figure 4.1 Typical Storey Structural Plan View

Column	bx (cm)	by (cm)
S2	30	30
S3	30	35
S4	35	35
S5	30	30
S7	25	30
S8	30	35
S9	35	40
S10	25	30
S11	30	30
S13	30	25
S14	40	35
S14A	40	45
S15	30	30
S17	25	30
S18	30	35
S19	35	40
S20	25	30
S22	30	30
S23	30	35
S24	35	35
S25	30	30

Figure 4.2 Column Dimensions

(Beam and column dimensions are same in each storey)

All beams are 25cmx50cm

The project and code parameters of existing building are given in Table 4.2 below. In this table, knowledge level and knowledge level factor are estimated according to the existence of the project of the building.

Table 4.2 Project and Code Parameters

Project Parameters of the Existing Building	**1975 Turkish Earthquake Code Parameters**
Project of the Building (Exist / Not exist): *Exist*	Earthquake Zone: *1*
Knowledge Level: *High*	Earthquake Zone Factor: *0.10*
Knowledge Level Factor: *1*	Building Importance Factor: *1.0*
Reinforcement Existing Factor: *1*	Soil Class: *Z3*
Existing Concrete Strength (Mean – Standard Deviation): *25 MPa*	Structure Type Factor: *0.8*
Existing Steel Strength (Mean–Standard Deviation): *420 MPa*	T_o : *0.6 s*
Target Performance Level: *Life Safety (At 50 years %10)*	
Live Load Participation Factor (n): *0.30*	

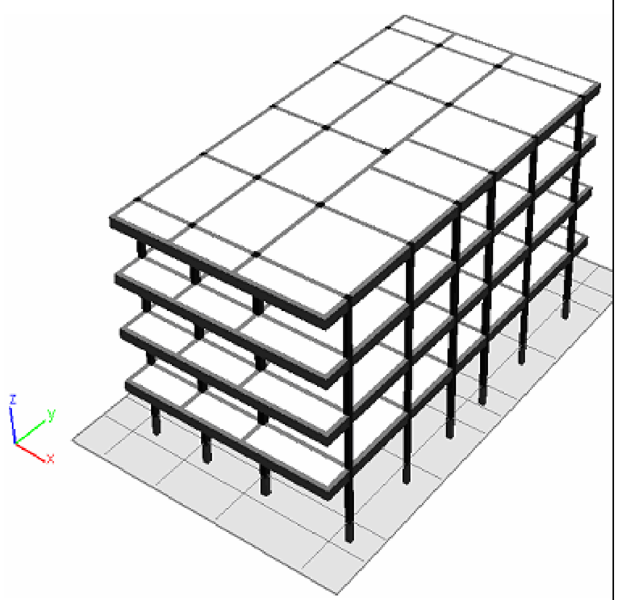


Figure 4.3 Three Dimensional Model

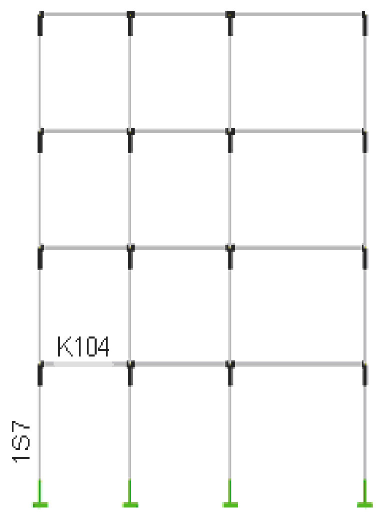


Figure 4.4 Frame B

(Rigid end zones, Example column and beam)

- All examples given below are for lateral loading in the +X direction.

- For linear elastic analysis, uncracked section stiffnesses are used in assessment.
  E_c = 30,250 MPa

4.1 Assessment of the Building by Linear Elastic Procedure

The vibration properties of the building in the fundamental mode are given below.

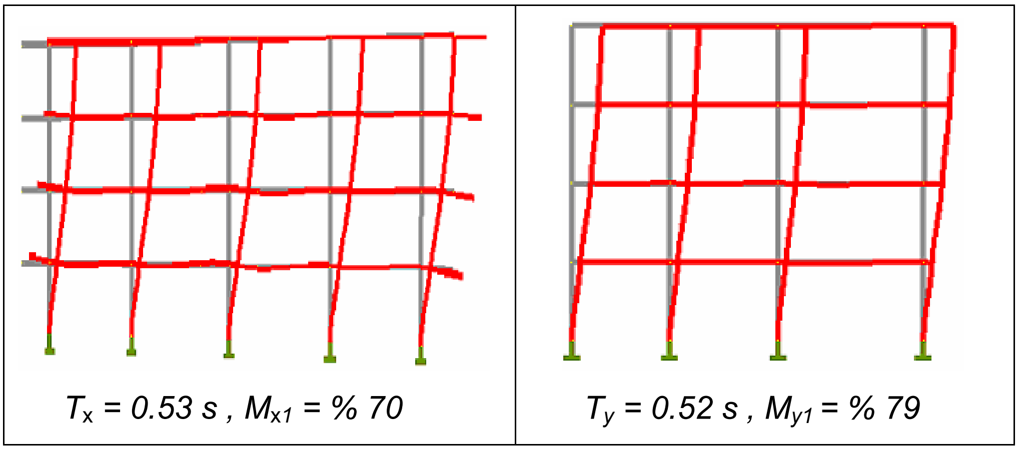


Figure 4.5 Natural vibration periods, mode shapes and effective mass ratios in X and Y directions

4.1.1 Calculation of the Equivalent Lateral Load Distribution

Base shear force calculation in X and Y direction:

$V_t = \lambda\ W\ A(T_1)\ /\ R_a(T_1)$ (4.1)

$A(T_1) = A_o\ I\ S(T_1)$ (4.2)

$R_a(T_1)$ =1, I=1 , A_o = 0.4, W = 11026.6 kN.

For T_x = 0.53s and T_y=0.52 s, $S(T)$ = 2.5g. By these values $A(T_1)$ = 1.0g is calculated. λ =0.85, V_t = 9372.6 kN.

ΔF_N = 0.0075 N V_t = 0.0075 x 4 x 9372.6 = 281.18 kN

4.1.2 Distribution of the Base Shear Force to the Stories

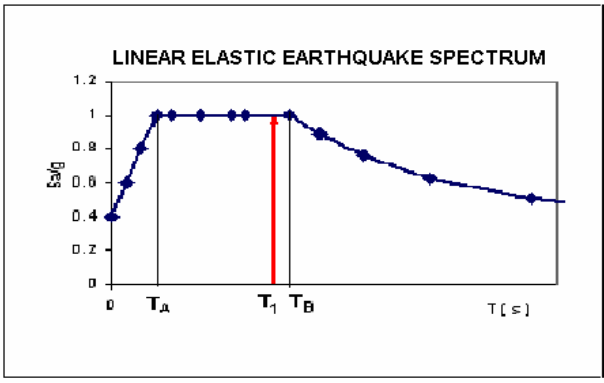
Distribution of the base shear force to the stories are calculated and summarized in Table 4.3 below. Linear elastic earthquake spectrum is also given below.

Table 4.3 Distribution of the Base Shear Force to the Floors

Storey	**Storey Weight *(W_i)* (kN)**	**Storey Height (m)**	***H_i* (m)**	***W_i H_i* (kNm)**	***F_i* (kN)**
4	2183.3	2.7	10.8	23580.1	3007.8
3	2944.0	2.7	8.1	23846.5	3041.8
2	2944.0	2.7	5.4	15897.7	2027.9
1	2944.0	2.7	2.7	7948.8	1013.9

$$F_{fi} = \frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j}$$

$$F_i = F_{fi}(V_t - \Delta F_N)$$



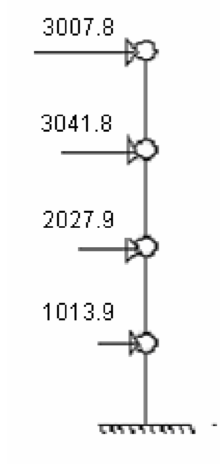


Figure 4.6 Linear Elastic Earthquake Spectrum

$T_A = 0.15s$, $T_B = 0.6s$, $T_{1x} = 0.53s$, $T_{1y} = 0.52s$

4.1.3 Analysis of the Building Under Vertical Loading (G+nQ) and the Lateral Loading (E)

The results of vertical and lateral load analysis are given below, in terms of the bending moment diagrams of a typical frame.

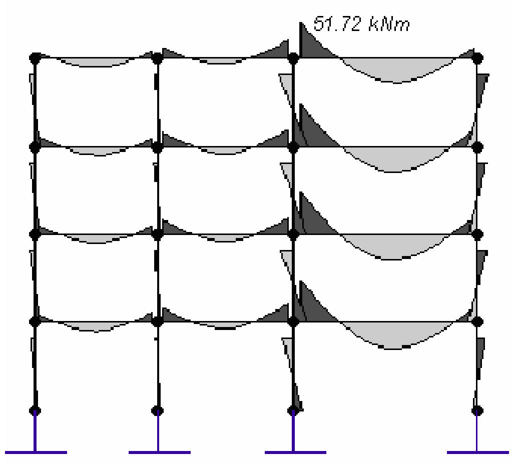


Figure 4.7 Moment Diagram of Frame B (M_D) under Vertical Loading

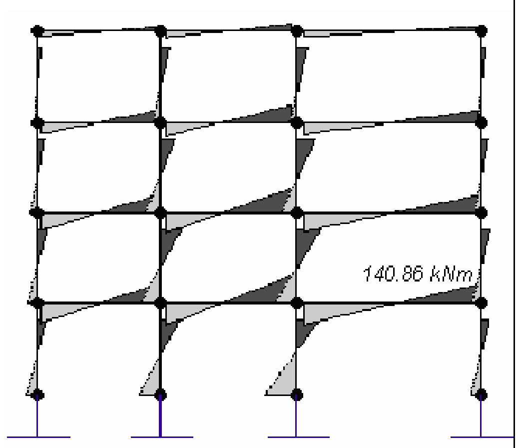
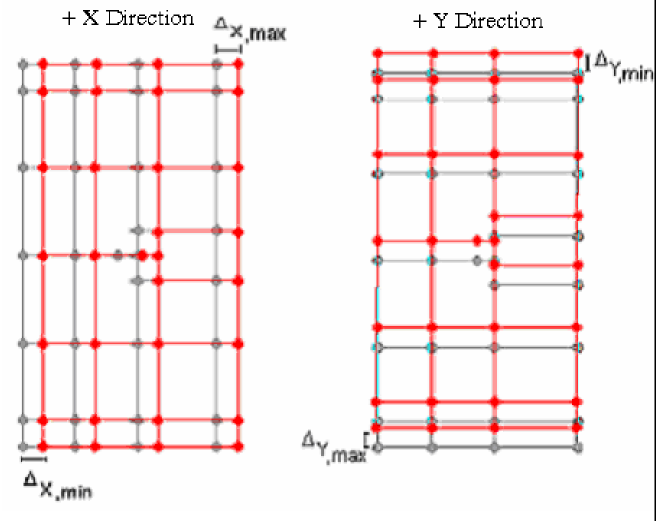


Figure 4.8 Moment Diagram of Frame B (M_E) under Lateral Loading

- In lateral load analysis, accidental eccentricities are not applied.
- η_b **check**: At each storey, in X and Y direction η_b **< 1.4** is satisfied.
- As an example, torsional irregularity check for 4th storey is given in Figure 4.9.



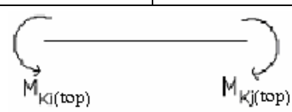
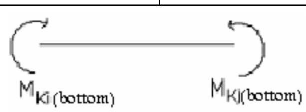
	+X direction	+Y direction
$(\Delta_i)_{max}$	0.01235	0.01197
$(\Delta_i)_{min}$	0.01233	0.01158
$(\Delta_i)_{ort}$	0.01234	0.01178
η_b	1.00	1.02

Figure 4.9 Torsional Irregularity Check

4.1.4 Calculation of Beam End Moment Capacities (M_K)

Top and bottom moment capacities of beam ends are calculated. As an example, beam end moment capacities of K104 (25cmx50cm) are given below.

Table 4.4 Beam End Moment Capacities of K104

	Top Moment Capacities		**Bottom Moment Capacities**		Stirrups at the end of the beams: Ø8/9 cm (Stirrups are same at both ends) Stirrups at middle of the beam : Ø8/20 cm
	i	***j***	***i***	***j***	
A_s (cm^2)	7.81	7.81	4.02	4.02	
M_K (kNm)	149.19	149.19	72.01	72.01	
					

Calculation of the Top Moment Capacities

d' = 25 mm , d = 500 – 25 = 475 mm , b_w = 250 mm , d'/d = 25 / 475 = 0.0526

$\rho = A_s / b_w\ d$ = 781 / (250x475) = 0.00658 , $\rho' = A_s' / b_w\ d$ = 402 / (250x475) = 0.0034

$\alpha = (\rho - \rho')\ f_{ym} / f_{cm}$ = (0.00658 – 0.0034) x 420 / 25 = 0.0534

d'/d = 0.0526 , S420 → α_c = 0.0966 > α , compression reinforcement is not yielded.

$\rho - \rho'$ = 0.00658 – 0.0034 = 0.00318 , ρ_b = 0.0209 , $\rho - \rho' < \rho_b$, section is under reinforced.

$\sigma_s' = 0.003\ (c - d')\ E_s / c = 600\ (c - 25) / c$

$0.85\ f_{cm}\ b_w\ k_1\ c + A_s'\ \sigma_s' - A_s\ f_{ym} = 0$

$0.85 \times 25 \times 250 \times 0.86 \times c + 402 \times 600 \times (c - 25) / c - 781 \times 420 = 0 \rightarrow c = 47$ mm

$\sigma_s' = 600 \times (c - 25) / c = 600 \times (47 - 25) / 47 = 280.85$ MPa

$M_r = 0.85\ f_{cm}\ b_w\ k_1\ c\ (d - k_1\ c / 2) + A_s'\ \sigma_s'\ (d - d')$

$M_r = 0.85 \times 25 \times 250 \times 0.86 \times 47 \times (475 - 0.86 \times 47/2) + 402 \times 280.85 \times (475 - 25)$ = ***149.19 kNm***

Calculation of the Bottom Moment Capacities

The same calculations are repeated for bottom moment capacities. As a result M_r = ***72.01 kNm*** *is calculated.*

4.1.5 Calculation of the Column Axial Loads

- For the columns of Frame B, the axial loads obtained under vertical loading (G+nQ) are (N_D), and those obtained from lateral loading are (N_E).
- Axial load calculations obtained from lateral loading (N_E) are based on the procedure in Appendix 7-A of the 2006 Turkish Earthquake Code.

EXAMPLE: Calculation of the axial load obtained from lateral loading N_E = – 226.11 kN for the column 1S7,

At first storey, the shear force transmitted from K104 ($V_{E,1}$):

Figure 4.10 Calculating the Residual Moment Capacities ΔM_E

$M_{Ki(bottom)}$ = 72.01 kNm , $M_{D,i}$ = – 10.44 kNm *(the moment obtained from the vertical loading)*

$\Delta M_{E,i} = M_{Ki(bottom)} - M_{D,i} = 72.01 - (-10.44) = 82.45$ kNm

$M_{Kj(top)}$ = 149.19 kNm , $M_{D,j}$ = 16.96 kNm *(the moment obtained from the vertical loading)*

$\Delta M_{E,j} = M_{Kj(top)} - M_{D,j} = 149.19 - 16.96 = 132.23$ kNm

$V_{E,1} = (\Delta M_{E,i} + \Delta M_{E,j}) / l_n = (82.45 + 132.23) / 3.075 = 69.82$ kN

$V_{E,1}$ =69.82 kN, *the force is transmitted to the column in opposite direction as –69.82 kN*

The same calculations are done for K204, K304, K404.

$V_{E,2} = -70.66$ kN , $V_{E,3} = -54.08$ kN, $V_{E,4} = -53.54$ kN.

$N_{E,1}$, is the sum of the shear forces transmitted from the beams on the top of the column 1S7 to this column:

$N_{E,1} = V_{E,1} + V_{E,2} + V_{E,3} + V_{E,4} = -69.82 + -70.66 + -54.08 + -53.54 = -248.1$ kN

$N_{D,1}$, 1S7 is the axial load obtained from the vertical loading: $N_{D,1}$ = 433.25 kN

$N_{D,1} + N_{E,1} = 433.25 + (-248.1) = 185.15$ *kN. For this axial load the moment capacity of the top end of* 1S7 is,

$M_{Kt} = 57.8$ *kNm.*

The moment capacity of the bottom end of 2S7 is calculated in the same way. As a result ,

$M_{Kb} = 53.95$ *kNm is calculated.*

The CBCR values of top end of 1S7 is calculated according to the equation (7A.2) of 2006 Turkish Earthquake Code:

$$CBCR = \frac{(M_{Kb} + M_{Kt})}{(M_{Ki(bottom)} + M_{Kj(top)})} = \frac{53.95 + 57.8}{72.01 + 0.0} = 1.55 \qquad (4.3)$$

The procedures used in calculating the CBCR value of the top end of 1S7 are repeated for calculating the CBCR values of the top ends of 2S7, 3S7 and 4S7. The CBCR values are calculated as *1.45*, *1.65* and *0.775*. In the same fashion, for the adjacent axes, the CBCR values of the top ends of 1S8, 2S8, 3S8 ve 4S8 are calculated as 1.15, 1.02, 1.12 and 0.519.

For the beam K404; $M_{Ki(bottom)} = 57.34$ *kNm*, $M_{Kj(top)} = 113.72$ *kNm*'dir.
But since for the top end of 4S7 ((i) end of K404), $CBCR = 0.775 < 1.0$, then in V_E calculation of K404,
$M_{Ki(bottom)} = 57.34 \times 0.775 = 44.44$ *kNm*,
and since for the top end of 4S8 ((j) end of K404), $CBCR = 0.519 < 1.0$, then
$M_{Kj(top)} = 113.72 \times 0.519 = 59.02$ *kNm.*
The above procedures are repeated by the modified beam capacities*:*

$M_{Ki(bottom)}$ = 44.44 kNm , $M_{D,i}$ = –6.85 kNm (the moment obtained from the vertical loading)

$\Delta M_{E,i} = M_{Ki(bottom)} - M_{D,i}$ = 44.44 – (–6.85) = 51.29 kNm

$M_{Kj(top)}$ = 59.02 kNm , $M_{D,j}$ = 13.27 kNm (the moment obtained from the vertical loading)

$\Delta M_{E,j} = M_{Kj(top)} - M_{D,j}$ = 59.02 –13.27 = 45.75 kNm

$V_{E,4} = (\Delta M_{E,i} + \Delta M_{E,j}) / l_n$ = (51.29 + 45.75) / 3.075 = 31.56 kN

At 4th storey, for 4S7; $N_{E,4} = V_{E,4}$ = –31.56 = – 31.56 kN

At 3rd storey, for 3S7; $N_{E,3} = V_{E,3} + V_{E,4}$ = –54.08 + –31.56 = – 85.64 kN

At 2nd storey, for 2S7; $N_{E,2} = V_{E,2} + V_{E,3} + V_{E,4}$ = –70.66 + –54.08 + – 31.56
= – 156.3 kN

At 1st storey, for 1S7; $N_{E,1} = V_{E,1} + V_{E,2} + V_{E,3} + V_{E,4}$
*= –69.82 + –70.66 + –54.08 + –31.56 = **– 226.11 kN.***

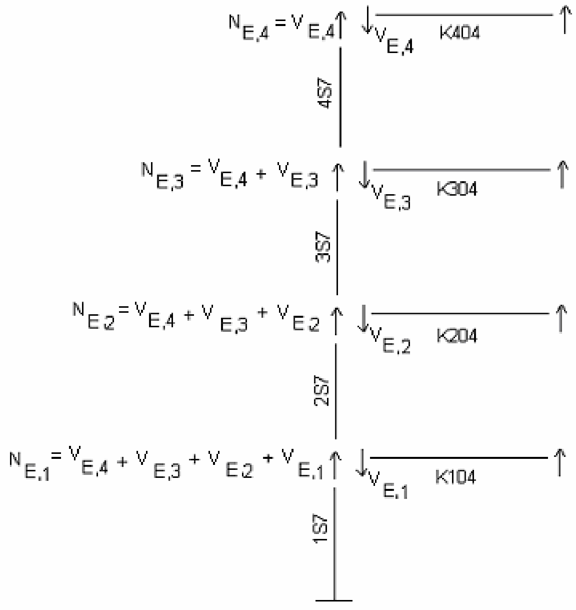


Figure 4.11 The Shear Forces Transmitted from the Beams

In this example, since CBCR < 1.0 only for the columns S7 and S8 at the 4th storey; then $M_{Ki(bottom)}$, $M_{Kj(top)}$ are reduced by multiplying with CBCR values for that storey. If *CBCR* < 1.0 were found for the other stories, then the same reduction should be applied to these stories.

In Table 4.5 below, 1st storey columns are abbreviated as 1S, 2nd storey columns are abbreviated as 2S, 3rd storey columns are abbreviated as 3S and 4th storey columns are abbreviated as 4S respectively.

Table 4.5 Axial Load Calculation for the Columns of Frame B

COLUMN	**N_D (kN)**	**N_E (kN)**	**N_D + N_E**
1S7	433.25	–226.11	207.13
1S8	697.87	20.29	718.16
1S9	916.97	23.15	940.13
1S10	496.59	178.38	674.97
2S7	316.01	–156.30	159.71
2S8	513.31	13.71	527.02
2S9	674.42	18.01	692.43
2S10	360.06	124.57	484.63
3S7	197.80	–85.64	112.16
3S8	328.96	6.86	335.82
3S9	432.91	7.29	440.20
3S10	224.14	71.50	295.63
4S7	80.41	–31.56	48.85
4S8	145.98	0.88	146.86
4S9	190.44	2.74	193.18
4S10	88.85	27.94	116.80

4.1.6 Calculation of the Column End Moment Capacities

EXAMPLE: The moment capacity of the ends of column 1S7 (25cmx30cm) for the axial load $N_D + N_E = 207.13\ kN$ is obtained from interaction diagram:

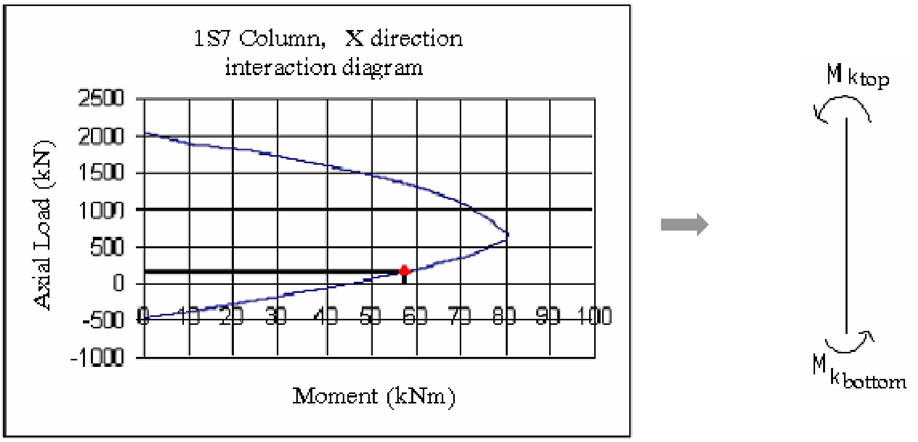
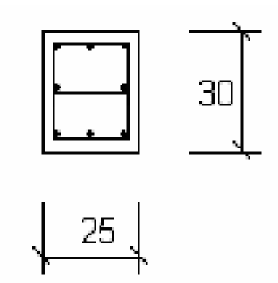


Figure 4.12(a) The Interaction Diagram of 1S7



Stirrups at the middle of the column: *Ø8 / 12cm*

Longitudinal reinforcement: *8 Ø14*

Stirrups at the end of the column: *Ø8 / 6cm*,

M_{Kt}	*58.86 kNm*
M_{Kb}	*58.86 kNm*

Figure 4.12(b) The Moment Capacities of Column 1S7

4.1.7 Shear Check for Beams and Columns

All beams and columns are controlled whether the shear failure occurs. Column 1S7 is controlled as an example. The procedure is given below for the example column.

Section shear capacities are calculated according to TS500,

$V_r = V_c + V_w = 0.8\ V_{cr} + V_w$ (4.4)

$V_c = 0.8 \times 0.65\ f_{ctm}\ b_w\ d\ (1 + \gamma\ N / A_c)$ (4.5)

$= 0.8 \times 0.65 \times 1.75 \times 300 \times 225 \times (1 + 0.07 \times 207.13 \times 1000 / (250 \times 300)) = 73300\ N$,

$V_w = A_{sw}\ f_{ywm}\ d / s = 150.8 \times 420 \times 225 / 120 = 118755\ N$ (4.6)

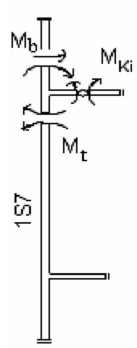
$V_r = 192.06\ kN$

The section shear demands are calculated according to Chapter 3.3.7 of the 2006 Turkish Earthquake Code, $V_E = (M_{Kt} + M_{Kb}) / l_n$. where, $M_{Kt} = 34.74\ kNm$, $M_{Kb} = 58.86\ kNm$, l_n *(clear length)* = $2.2m$,

$V_E = 42.6\ kN$.

Since at the top joint of the example column, the columns are stronger than the beams, M_{Kt} values should be calculated from Figure 3.5 of the 2006 Turkish Earthquake Code.

Calculation of M_{Kt} = 34.74 kNm



$$\sum M_K = M_{Ki} + M_{Kj} \qquad M_{Kt} = \frac{M_t}{M_t + M_b} \sum M_K$$

M_t	241.28 kNm
M_b	258.78 kNm
M_{Ki}	72.01 kNm
M_{Kj}	0 kNm

Figure 4.13 Calculation of M_{Kt}

M_b , M_t values are obtained from lateral load analysis. M_b, is the moment at the bottom end of 2S7 calculated from analysis; M_t, is the moment at the top end of 1S7 calculated from analysis. M_{Kj} and M_{Ki} values are top moment capacity of the right end of the beam at the left side of the joint (there is no beam at the left side of example joint) and bottom moment capacity of the left end of the beam at the right side of the joint.

$\Sigma M_K = 72.01 + 0 = 72.01\ kNm$

$M_{Kt} = 241.8 \times 72.01 / (241.8 + 258.78) = 34.74\ kNm$

Since $V_E < V_r$ then 1S7 column is **ductile.**

Shear check for beam K104 is also performed as an example beam. The procedure is given below for that beam.

Section shear capacities are calculated according to TS500,

$$V_r = V_c + V_w = 0.8\ V_{cr} + V_w \qquad (4.7)$$

For beam support section,

$V_c = 0.8 \times 0.65\ f_{ctm}\ b_w\ d = 0.8 \times 0.65 \times 1.75 \times 250 \times 475 = 108062.5\ N,$

$V_w = A_{sw}\ f_{ywm}\ d/s = 100.5 \times 420 \times 475 / 90 = 222775\ N,$

$V_{ri} = V_{rj} = 330.8\ kN.$

Shear demand is calculated according to the Chapter 3.4.5 of the 2006 Turkish Earthquake Code

For (i) end of the beam,

$$V_e = V_{dy} - (M_{Ki,(bottom)} + M_{Kj,\ (top)}) / l_n \qquad (4.8)$$

$V_{dy} = 27.95\ kN$, $M_{Ki(alt)} = 72.01\ kNm$, $M_{Kj(üst)} = 149.19\ kNm$, l_n *(clear span length)* $= 3.075\ m$,

$V_{Ei} = 43.98\ kN$

For (j) end of the beam,

$$V_E = V_{dy} + (M_{Ki(bottom)} + M_{Kj(top)}) / l_n \qquad (4.9)$$

V_{dy} = 27.95 kN, $M_{Ki(bottom)}$ = 72.01 kNm, $M_{Kj(top)}$ = 149.19 kNm, l_n (clear span length) = 3.075 m,
V_{Ej} = 99.88 kN.

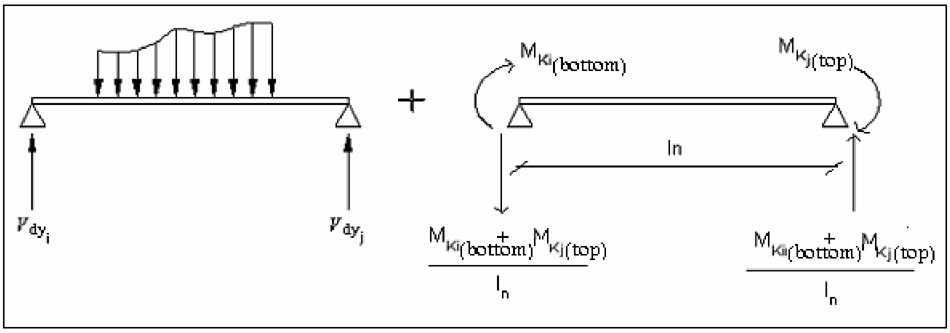


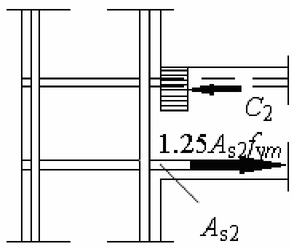
Figure 4.14 Calculation of Shear Forces Transmitted from Beams

Since V_E < *V_r* then K104 is **ductile.**

4.1.8 Shear Check for Joints

Shear check for the joint at the top of column 1S7 is also performed as an example joint. The procedure is given below for that joint.

The example joint is unconfined. Hence, *V_r = 843.8 kN* is calculated.
Section shear demand is calculated by Chapter 3.5.2.1 of the 2006 Turkish Earthquake Code.

$V_E = 1.25\, f_{ym}\, (A_{s1} + A_{s2}) - V_{E(col)}$ $A_{s1} = 0$, $A_{s2} = 402\ mm^2$, $f_{ym} = 420\ MPa$, *For 1S7* $V_{E(col)} = 42.6\ kN$, *for 2S7* $V_{E(col)} = 36.1\ kN$, $V_{E(col)} = min(42.6; 36.1) = 36.1\ kN$, $V_E = 175\ kN$ *Since* $V_E < V_r$ the joint is satisfactory.	 Figure 4.15 Joint Section

All joints of frame B are controlled similarly, and the results are presented in Table 4.6 below.

Table 4.6 Shear Check for the Joints (the joints at the top of the columns) of Frame B

JOINT	V_E (kN)	V_r (kN)
1S7	175.00	843.75
1S8	521.09	1181.25
1S9	498.20	1575.00
1S10	360.00	843.75
2S7	173.23	843.75
2S8	533.29	1181.25
2S9	532.51	1575.00
2S10	352.75	843.75
3S7	134.97	843.75
3S8	402.24	1181.25
3S9	400.33	1575.00
3S10	271.19	843.75
4S7	100.86	843.75
4S8	179.38	1181.25
4S9	304.76	1575.00
4S10	107.36	843.75

4.1.9 Calculation of "Demand / Capacity Ratios (r)" and "Limit values (r_{Limit})" of Beam and Column Sections

EXAMPLE: The limit values for column 1S7

$N = 207.13\ kN$, $A_c = 750\ cm^2$, $f_{cm} = 25\ MPa \rightarrow N / (A_c\ f_{cm}) = 0.11$
$V = V_E = 42.6\ kN$, $b_w d = 687.5\ cm^2$, $f_{ctm} = 1.75\ MPa \rightarrow V / (b_w\ d\ f_{ctm}) = 0.35$
The example column is confined.

Since the building is checked for "**Life Safety Performance Level**", the limit state of "**GV**" is considered.

"r_{Limit}" values are calculated by iterations from Table 7.3 of 2006 Turkish Earthquake Code with calculated $N / (A_c\ f_{cm})$ and $V / (b_w\ d\ f_{ctm})$ values. As a result "r_{Limit}" is calculated as **5.93.**

The moment caused by lateral loading (M_E), at the top end of the column is 241.28 kNm, at the bottom end of the column is 286.56 kNm. From the column moment capacity, the moment caused by vertical loading (M_D) is subtracted in order to find "**residual moment capacity**". Residual moment capacity at the top end is $\Delta M_{Kt} = 58.86 - 4.96 = 53.91\ kNm$, at the bottom end is $\Delta M_{Kb} = 58.86 - (-3.83) = 62.69\ kNm$..
"**Demand / Capacity Ratio (r)**" at the top end is $M_{Et} / \Delta M_{Kt} = 241.28 / 53.91 = 4.48$,
at the bottom end is $M_{Eb} / \Delta M_{Kb} = 286.56 / 62.69 = 4.57$.
At the top end of the column $r/r_{Limit} = 4.48 / 5.93 = 0.75$, at the bottom end of the column $r/r_{Limit} = 4.57 / 5.93 = 0.77$.

Since r/r_{Limit} values are less than 1 at both ends of the column, this column satisfies "**GV**" **limit state**. If at any end of the columns, r/r_{Limit} values were greater than 1, the column could not satisfy the '**GV**' limit state.
EXAMPLE : The limit values for the beam K104

(i) end: $\rho = 0.0034$, $\rho' = 0.00658$, $\rho_b = (0.85\, f_{cm} / f_{ym})\, k_1\, (0.003\, E_s / (0.003\, E_s + f_{ym})) = 0.0209$

$\rightarrow (\rho - \rho') / \rho_b = -0.152$

(j) end: $\rho = 0.00658$, $\rho' = 0.0034$, $\rho_b = (0.85\, f_{cm} / f_{ym})\, k_1\, (0.003\, E_s / (0.003\, E_s + f_{ym})) = 0.0209$

$\rightarrow (\rho - \rho') / \rho_b = 0.152$

(i) end : $V = 43.98$ kN, $b_w d = 1187.5$ cm², $f_{ctm} = 1.75$ MPa $\rightarrow V / (b_w\, d\, f_{ctm}) = 0.21$

(j) end : $V = 99.88$ kN, $b_w d = 1187.5$ cm², $f_{ctm} = 1.75$ MPa $\rightarrow V / (b_w\, d\, f_{ctm}) = 0.48$

Example beam is confined.
Since the building is checked for "**Life Safety Performance Level**", the limit state "**GV**" is considered.
"r_{Limit}" values are calculated by iterations from Table 7.2 of the 2006 Turkish Earthquake Code with the calculated $(\rho - \rho') / \rho_b$ and $V / (b_w\, d\, f_{ctm})$ values. As a result "r_{Limit}" is calculated as **7** for (i) end and **6.39** for (j) end.

The moment caused by the lateral loading (M_E) *at the* (i) end of the beam is *574.7* kNm, at the (j) end of the beam is *543* kNm. From the beam moment capacity, the moment caused by vertical loading (M_D) is subtracted in order to find the "**residual moment capacity**". Residual moment capacity at the (i) end is $\Delta M_{Ki} = 72.01 - (-10.44) = 82.45$ kNm, at the (j) end is $\Delta M_{Kj} = 149.19 - (16.96) = 132.23$ kNm..

Demand / Capacity Ratio (r) at (i) end is $M_{Ei} / \Delta M_{Ki}$ = *574.7 / 82.45* = ***6.97***, at (j) end is $M_{Ej} / \Delta M_{Kj}$ = *543 / 132.23* = ***4.11***

At (i) end of the beam r / r_{Limit} = *6.97 / 7 = 0.99*, at (j) end of the beam r / r_{Limit} = *4.11 / 6.39 = 0.64*

Since r/r_{Limit} values are less than 1 at both ends of the beam, this beam satisfies the "**GV**" **limit state**. If at any end of the beams, r/r_{Limit} values were greater than 1, the beam could not satisfy '**GV**' limit state.

4.1.10 Comparison of Demand / Capacity Ratios (r) with Limit Values (r_{Limit})

The ratios of r to r_{Limit} values are calculated for all member ends and presented in bar chart form for beams and columns separately in each storey.

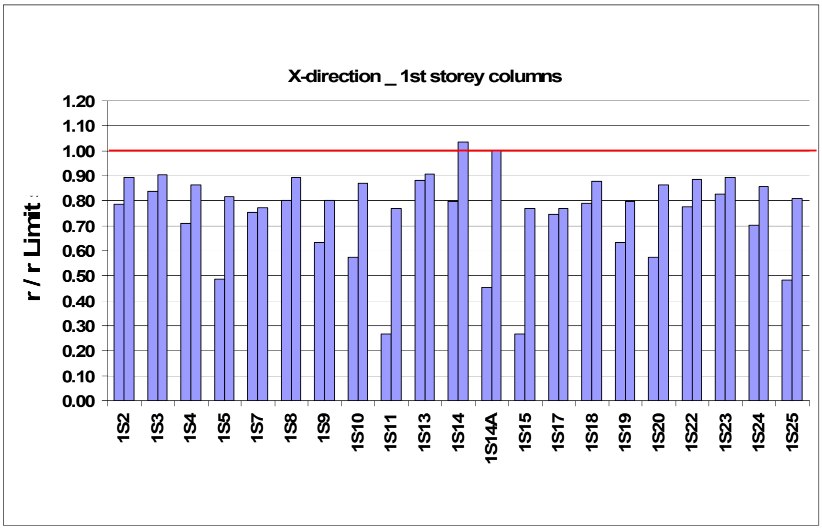


Figure 4.16 r / r_{Limit} Values for 1st Storey Columns

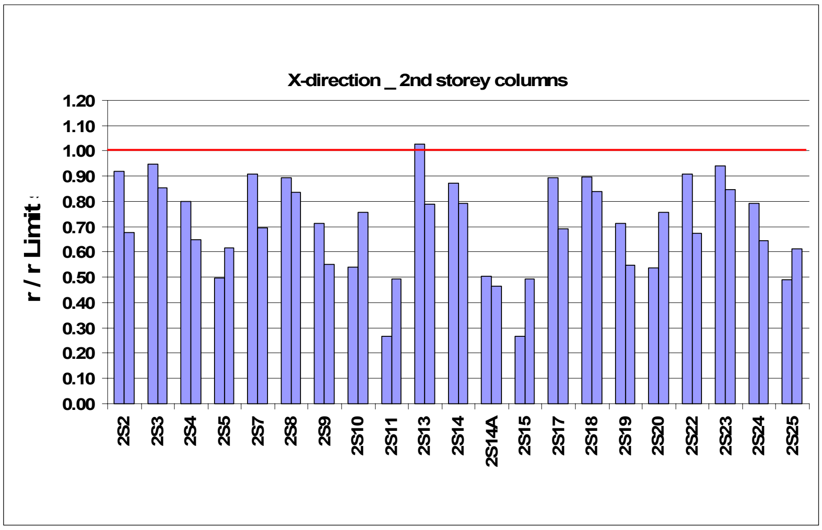


Figure 4.17 r / r_{Limit} Values for 2^{nd} Storey Columns

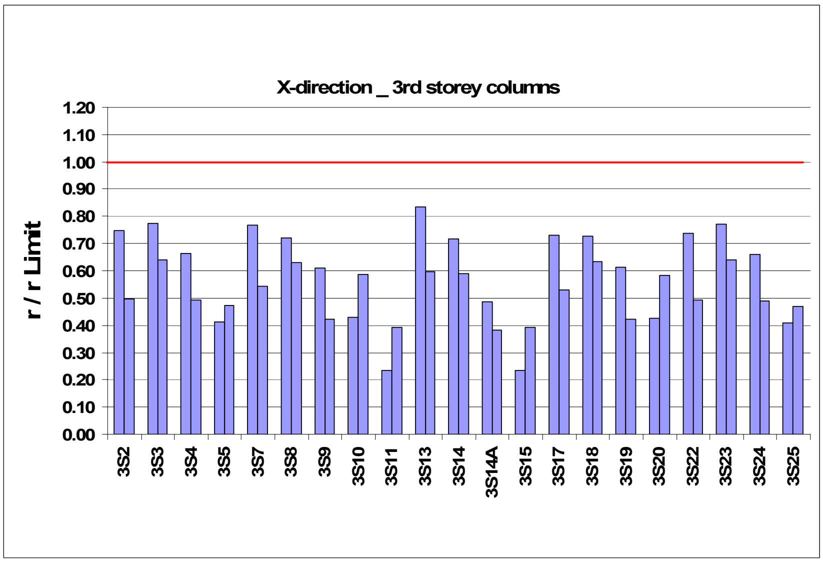


Figure 4.18 r / r_{Limit} Values for 3^{rd} Storey Columns

In X – direction, all 4^{th} storey columns satisfy the “**GV**” Limit State. Hence the graphics of 4^{th} storey columns are not given.

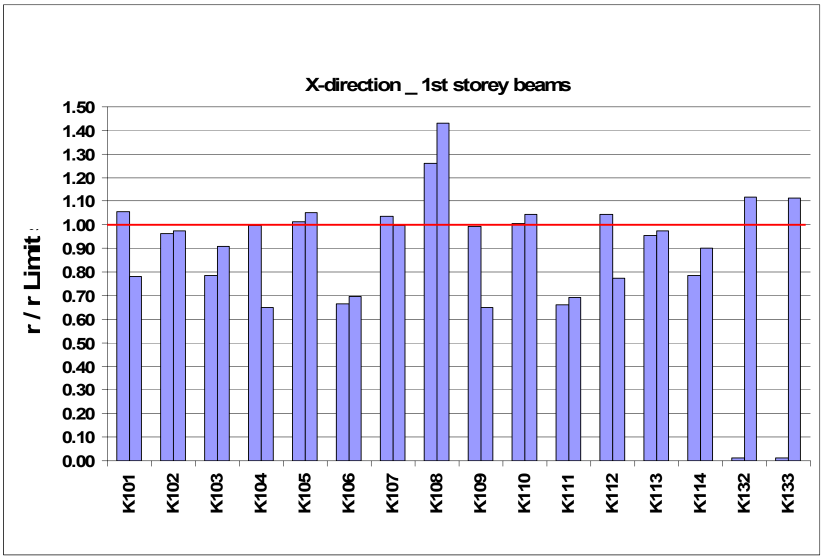


Figure 4.19 r / r_{Limit} Values for 1^{st} Storey Beams

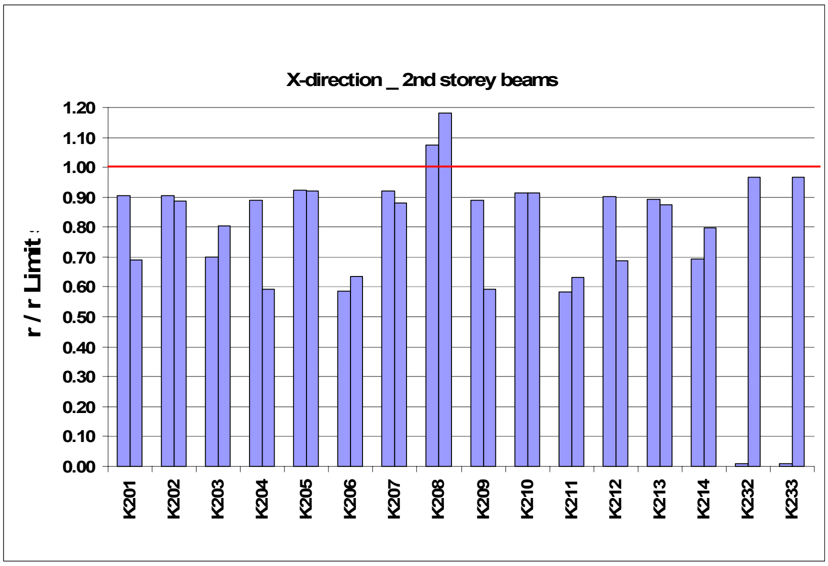


Figure 4.20 r / r_{Limit} Values for 2^{nd} Storey Beams

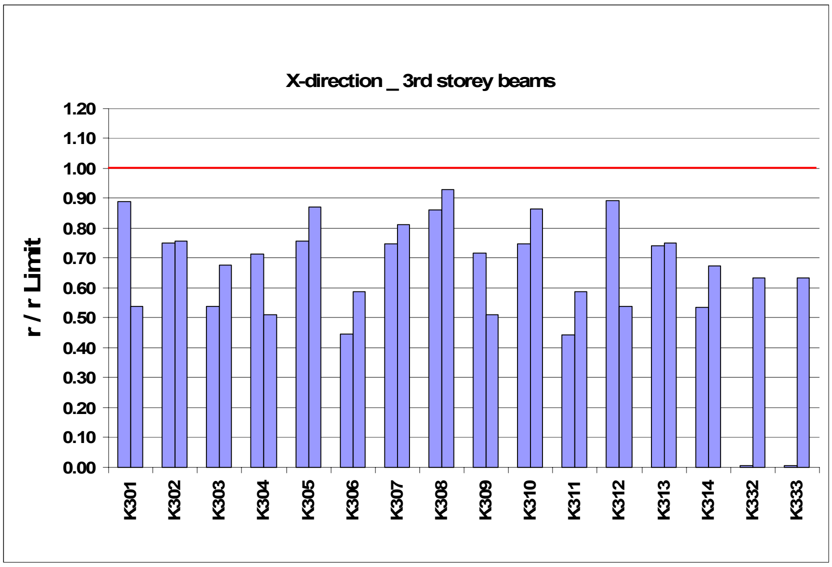


Figure 4.21 r / r_{Limit} Values for 3rd Storey Beams

In X – direction, all 4th storey beams satisfy the "**GV**" Limit State. Hence the graphics of 4th storey beams are not given.

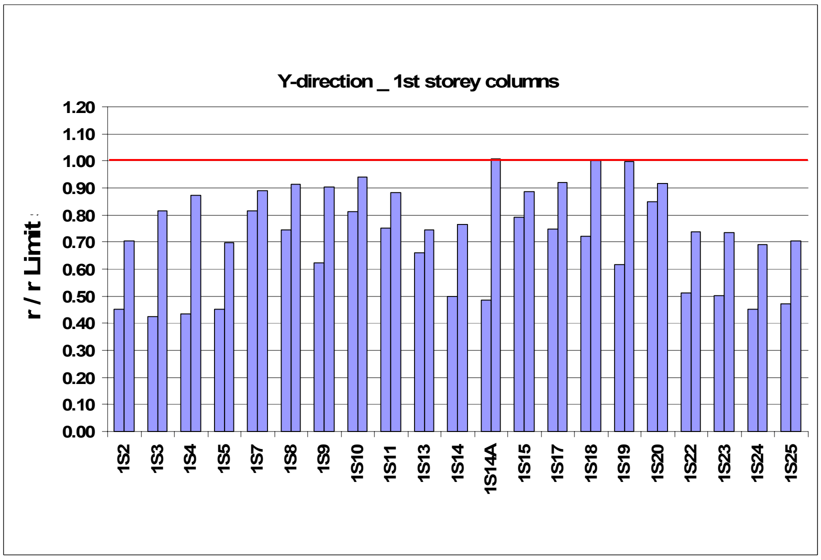


Figure 4.22 r / r_{Limit} Values for 1st Storey Columns

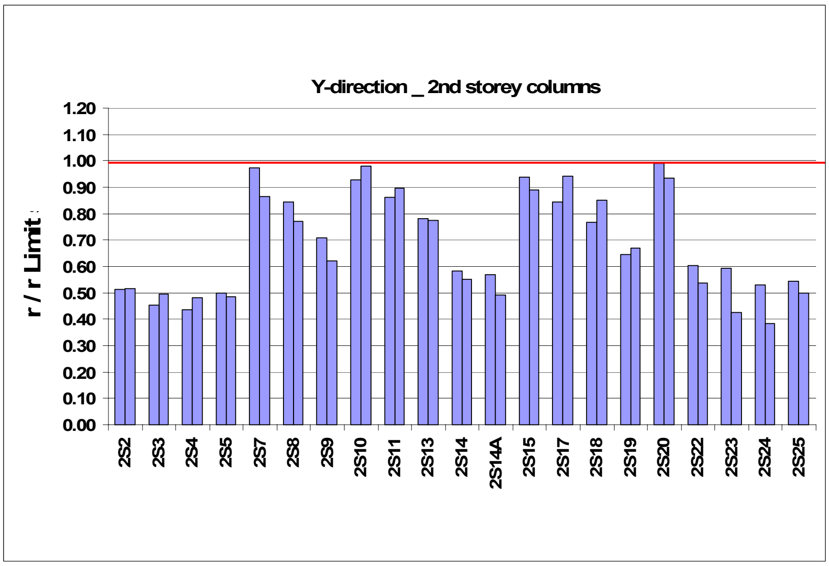


Figure 4.23 r / r_{Limit} Values for 2nd Storey Columns

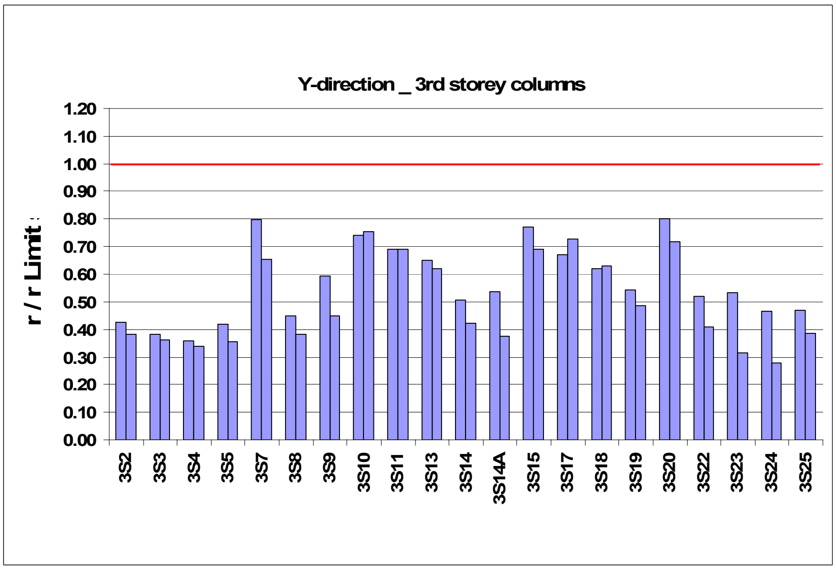


Figure 4.24 r / r_{Limit} Values for 3rd Storey Columns

In Y – direction, all 4th storey columns satisfy the “**GV**” Limit State. Hence the graphics of 4th storey columns are not given.

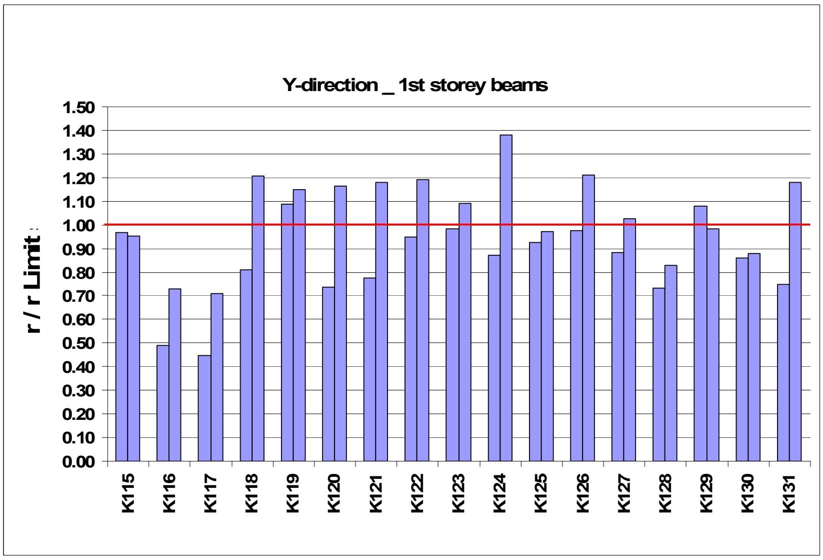


Figure 4.25 r / r_{Limit} Values for 1st Storey Beams

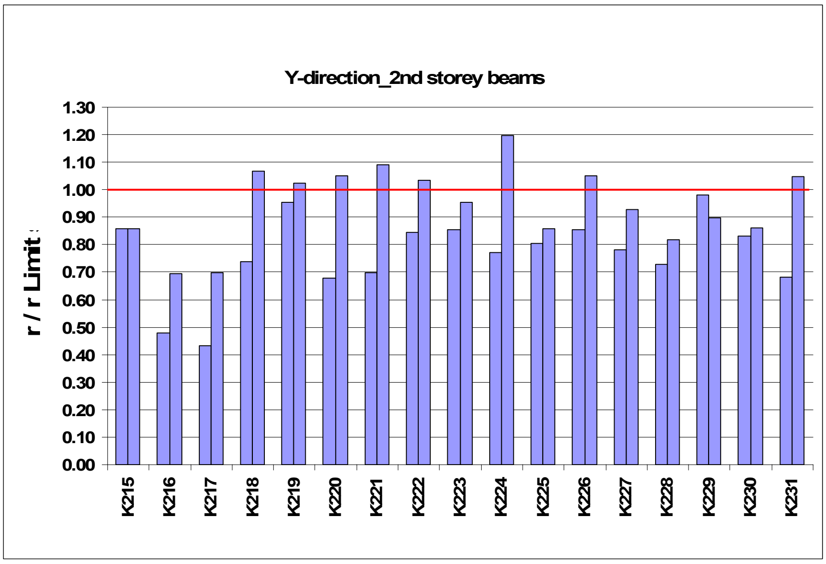


Figure 4.26 r / r_{Limit} Values for 2nd Storey Beams

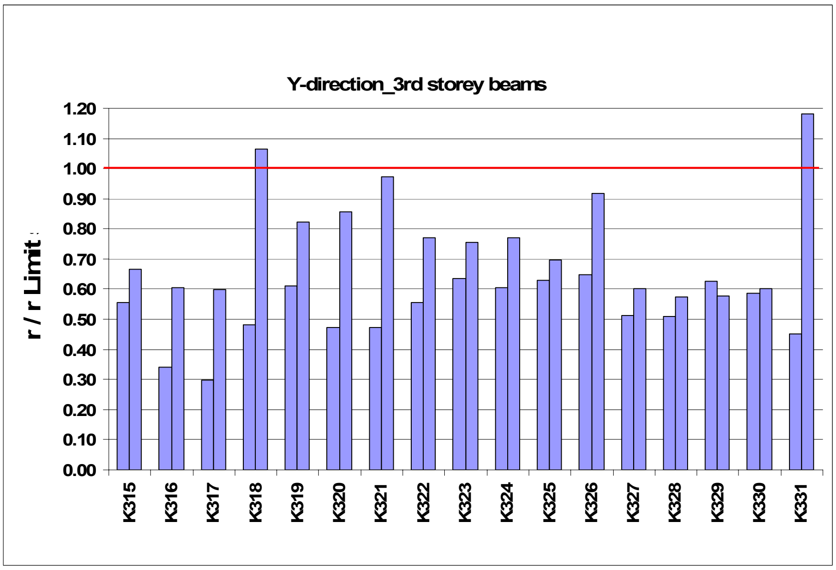


Figure 4.27 r / r_{Limit} Values for 3rd Storey Beams

In Y – direction, all 4th storey beams satisfy the "**GV**" Limit State. Hence the graphics of 4th storey beams are not given.

4.1.11 Global Performance of the Building

In Table 4.7 given below, the ratio of the number of unacceptable beams to all beams in the considered storey and in the considered direction, and the ratio of storey shear taken by unacceptable columns are given. For the values greater than 20%, the corresponding storey and the building performance are not acceptable.

In +X direction; 9 of 16 beams at 1st storey are not accepted. However, r / r_{Limit} values for 6 of these 9 beams are less than 1.05. These beams satisfy Collapse Prevention limit state. If the performances of these beams are considered as satisfactory, then the ratio of unacceptable beams are 3/16 = %19.

In the same manner, in +Y direction, at 1st storey the beams with r / r_{Limit} less than 1.05 can be assumed as satisfactory. Then the ratios of unacceptable beams are %59 at 1st storey, %24 at 2nd and %12 at 3rd storey.

In +Y direction, at 1st storey, the shear forces taken by unacceptable columns are %23.9, which is so close to %20. Hence, this storey is assumed to be satisfactory.

The following table is prepared with these values:

Table 4.7 Percentage of Unacceptable Beams and Columns

	+ X Direction		**+ Y Direction**	
Storey	**Beams (%)**	**Columns (%)**	**Beams (%)**	**Columns (%)**
1	19	18	59	24
2	6	4	24	0
3	0	0	12	0
4	0	0	0	0

Since the case study is a residential building, the target performance level is selected as “Life Safety Performance Level” and the calculated interstorey drifts should be less than 0.02. The results are given in the table below.

Table 4.8 Interstorey Drifts

	+ X Direction			**+ Y Direction**		
Storey	***H_i (m)***	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**	***H_i (m)***	**$(\Delta_i)_{max}$**	**$(\Delta_i)_{max}$ / H_i**
1	2.7	0.02548	0.009	2.7	0.02411	0.009
2	2.7	0.02953	0.011	2.7	0.02872	0.011
3	2.7	0.02265	0.008	2.7	0.02200	0.008
4	2.7	0.01235	0.005	2.7	0.01197	0.004

4.2 Assessment of the Building by Non-Linear Procedure

For non-linear analysis, cracked section stiffnesses are used in assessment .

For beams: $0.40\ EI_o$

For columns and shearwalls: if $N_D / (A_c f_{cm}) \leq 0.10 \Rightarrow 0.40\ EI_o$

If $N_D / (A_c f_{cm}) \geq 0.40 \Rightarrow 0.80\ EI_o$

The vibration properties of the building in the fundamental mode are given below.

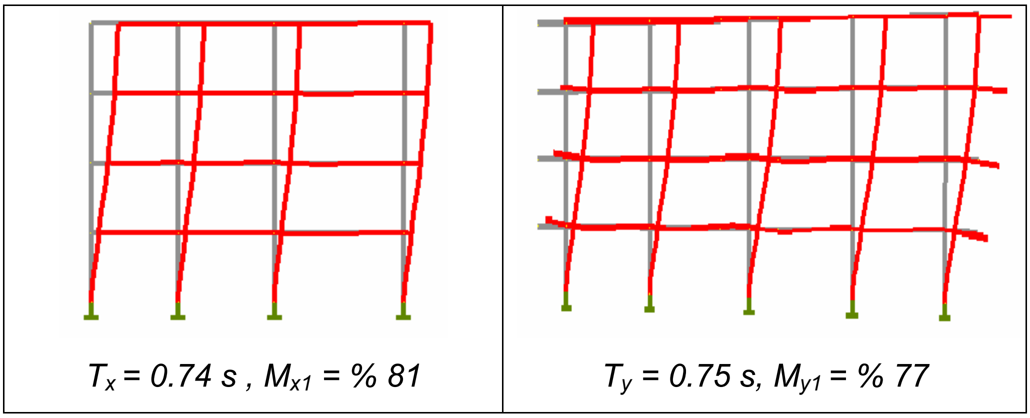


Figure 4.28 Natural Vibration Periods, Mode Shapes and Effective Mass Ratios in X and Y Directions

4.2.1 Calculation of Moment Curvature Relations of Beam Ends

EXAMPLE: Moment-curvature relations of the ends of the beam K104 (25cmx50cm) are calculated and converted into moment-rotation relations.

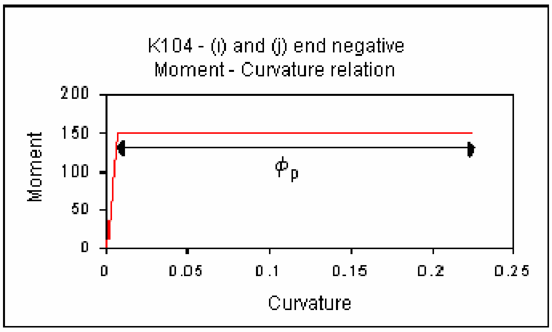


Figure 4.29 Moment-Curvature Relation for K104

The negative moment – curvature relations of (i) and (j) end of K104 are given in Figure 4.29 above (ϕ_p = plastic curvature). Moment – plastic curvature relation is obtained from moment – curvature relation and then converted into moment-plastic rotation relation as shown in Figure 4.30.

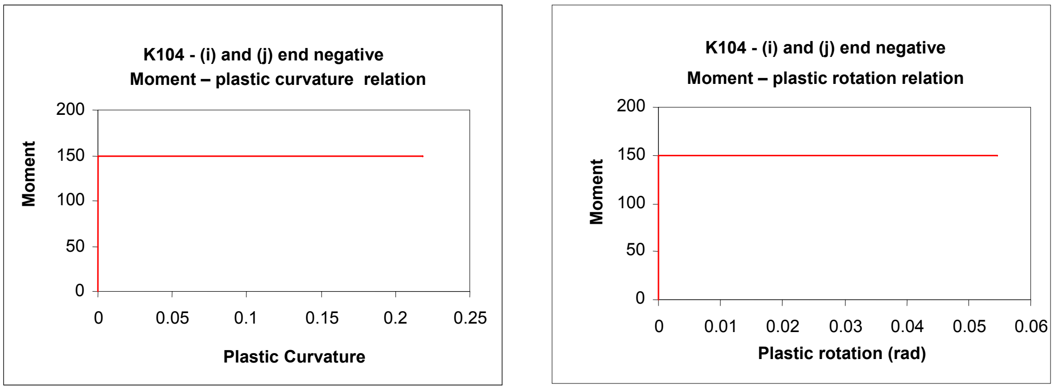


Figure 4.30 Negative Moment-Plastic Curvature and Moment-Plastic Rotation Relations

The positive moment – plastic curvature relations of (i) and (j) ends of K104 are also calculated. Moment - plastic rotation relations are calculated from moment – plastic curvature relations as given in Figure 4.31.

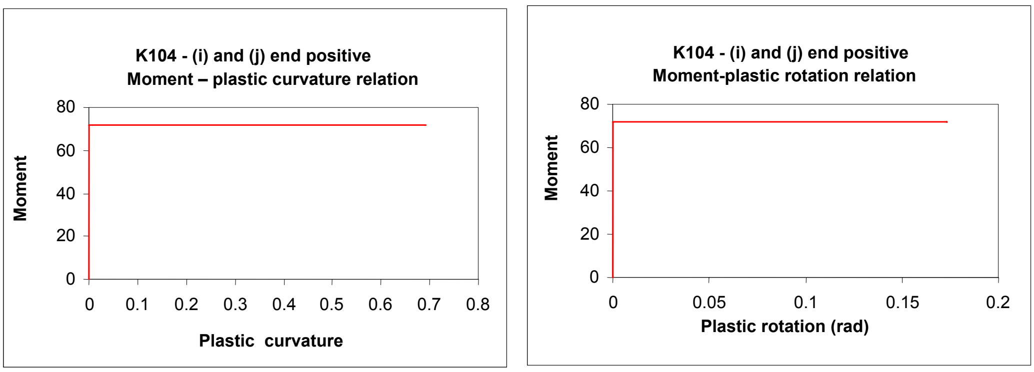


Figure 4.31 Positive Moment-Plastic Curvature and Moment-Plastic Rotation Relations

4.2.2 Calculation of Moment-Curvature Relations for Column Ends

EXAMPLE: The moment – plastic curvature relation of column 1S7 (25cmx30cm) is obtained under vertical loading (G+nQ) and converted into moment-plastic rotation relation.

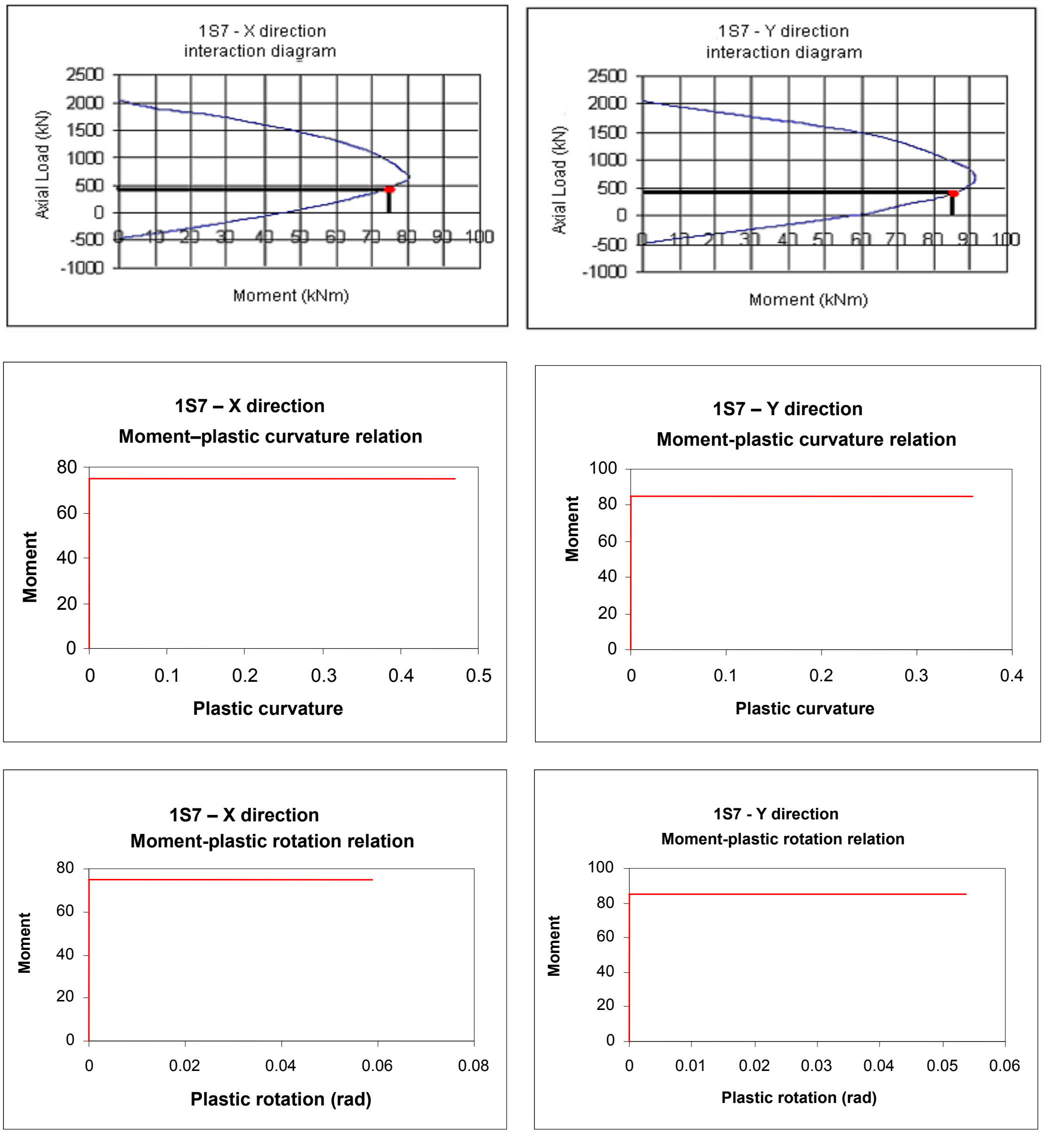


Figure 4.32 Interaction Diagrams, Moment-Plastic Curvature Relations, Moment-Plastic Rotation Relations in X and Y Directions

These moment-plastic rotation relations are updated at each step of the pushover analysis in accordance with the calculated axial loads. The results are shown in Figure 4.32 below.

4.2.3 Capacity Curve in X and Y Directions

Capacity curves are obtained in +X and +Y direction, and presented below.

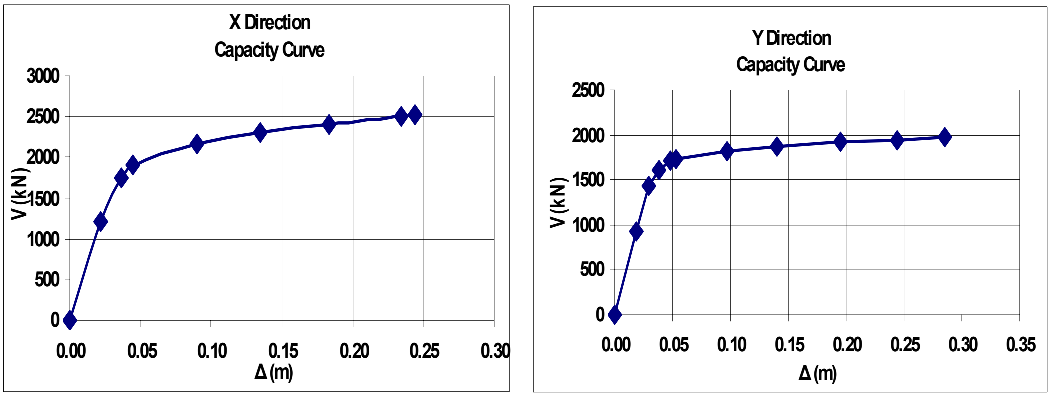


Figure 4.33 Capacity Curves in X and Y directions

4.2.4 Calculation of Performance Point in X and Y Directions

In +X and +Y directions, performance point is calculated by using the capacity curve and demand curve in accordance with the Code, as presented in Figure 4.34 and Figure 4.35

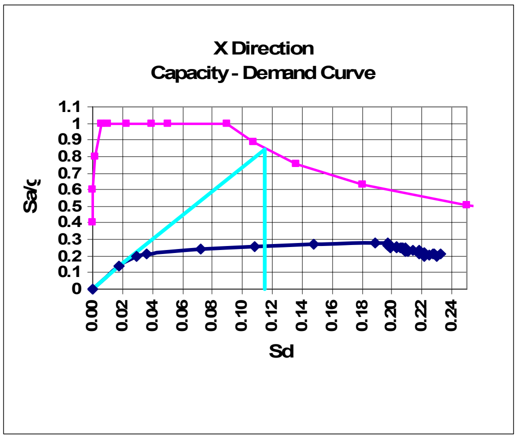


Figure 4.34 Capacity-Demand Curve in X Direction

T > T_B → $S_{di1} = S_{de1} = 0.115$

$u_{xN1} = \Phi_{xN1}\ \Gamma_{x1}\ S_{di1} = 0.04111 \times 30.12 \times 0.115$

$u_{xN1} = 0.142\ m$

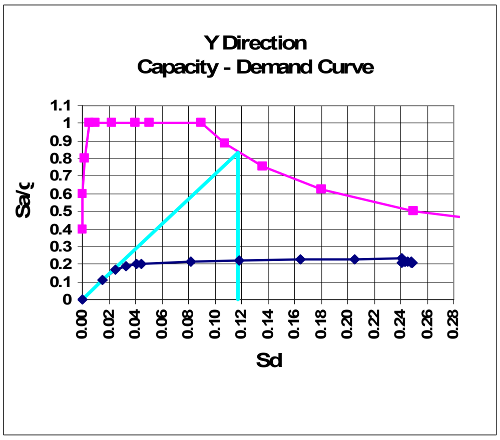


Figure 4.35 Capacity-Demand Curve in Y Direction

T > T_B → $S_{di1} = S_{de1} = 0.117$

$u_{yN1} = \Phi_{yN1}\ \Gamma_{y1}\ S_{di1} = 0.04034 \times 29.38 \times 0.117$

$u_{yN1} = 0.139\ m$

4.2.5 Shear Check for Beams and Columns

EXAMPLE: Shear check for column 1S7

Section shear capacity is calculated with $V_r = V_c + V_w$ according to the TS500. ($V_r = 192.06\ kN$).

Section shear demand is calculated from pushover analysis as $V_E = 54.2\ kN$.

EXAMPLE: Shear check for beam K104

Section shear capacity is calculated with $V_r = V_c + V_w$ according to the TS500. For support section $V_r = 330.8\ kN$.

Section shear demand is calculated from pushover analysis. For (i) end V_E = 44.4 kN, for (j) end V_E = 99.1 kN.

Since $V_E < V_r$, then the ends of 1S7 and the ends of K104 are ***ductile.***

4.2.6 Shear Check for Joints

EXAMPLE: The shear capacity of the joint at the top of column 1S7 is the same as in linear elastic procedure, which gives $V_r = 843.8\ kN$.

Section shear capacity is calculated from Chapter 3.5.2.1 of 2006 Turkish Earthquake Code

$$V_E = 1.25\, f_{ym}\, (A_{s1} + A_{s2}) - V_{E(col)} \qquad (4.10)$$

$A_{s1} = 0$, $A_{s2} = 402\ mm^2$, $f_{ym} = 420\ MPa$,

for 1S7 $V_{E(col)} = 54.19\ kN$, *for 2S7* $V_{E(col)} = 45.01\ kN$,

$V_{E(col)} = \min(54.19;\ 45.01) = 45.01\ kN$, $V_E = 166.04$.

Since $V_e < V_r$, then the example joint is satisfactory in shear. Comparison of shear capacity with shear demand in the other joints of Frame B are tabulated below.

Table 4.9 Shear Check for the Joints (the joints at the top of the columns) of Frame B

JOINT	***V_E (kN)***	***V_r(kN)***
1S7	166.04	843.75
1S8	539.22	1181.25
1S9	505.71	1575.00
1S10	366.33	843.75
2S7	185.58	843.75
2S8	560.48	1181.25
2S9	567.13	1575.00
2S10	373.07	843.75
3S7	155.37	843.75
3S8	458.18	1181.25
3S9	466.09	1575.00
3S10	299.36	843.75
4S7	117.87	843.75
4S8	219.00	1181.25
4S9	355.70	1575.00
4S10	123.16	843.75

4.2.7 Calculation of Strains at Member Sections

EXAMPLE: The top end of the column 1S7 (25cmx30cm) does not yield.
Calculation of the total curvature at the bottom end of column 1S7:

The plastic rotation at the bottom end of the column is obtained from pushover analysis:

$\theta_p = 0.014567\ rad$

Plastic curvature (ϕ_p) is calculated with dividing the obtained plastic rotation (θ_p) by plastic hinge length ($L_p = 0.25 / 2 = 0.125m$):

$\phi_p = 0.014567 / 0.125 = 0.116536\ 1/m$

Calculated plastic curvature (ϕ_p) is added to the yield curvature (ϕ_y) in order to obtain total curvature (ϕ_t) :

$\phi_y = 0.018124$; $\phi_t = 0.13466$

Concrete strain (ε_c) and steel strain (ε_s) corresponding to the obtained total curvature (ϕ_t) are calculated, and compared with the limit values ($\varepsilon_{cg(GV)}$, $\varepsilon_{s(GV)}$) :

$\varepsilon_1 = \varepsilon_c = 0.003968$, $\varepsilon_{cg(GV)} = 0.0135 \rightarrow \varepsilon_c / \varepsilon_{cg(GV)} = 0.29$

$\varepsilon_2 = \varepsilon_s = 0.02216$, $\varepsilon_{s(GV)} = 0.040 \rightarrow \varepsilon_s / \varepsilon_{s(GV)} = 0.55$

$\varepsilon / \varepsilon_{Limit} = max(0.29, 0.55) = 0.55$

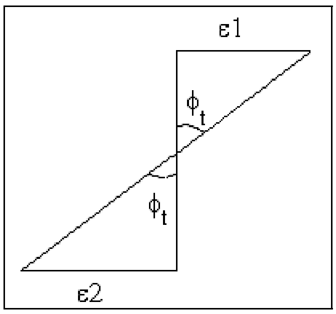


Figure 4.36 Bottom Section of Column 1S7

Since $\varepsilon / \varepsilon_{Limit} < 1.0$ for the top and bottom sections of 1S7 , then the column satisfies **"GV" Limit State**.

The same calculations are repeated for (i) end and (j) end of K104.

Concrete strain (ε_c) and steel strain (ε_s) are calculated and compared to the limit states:

K104 (i) end: $\varepsilon_1 = \varepsilon_c = 0.000735$, $\varepsilon_{cg(GV)} = 0.0135 \rightarrow \varepsilon_c / \varepsilon_{cg(GV)} = 0.05$

$\varepsilon_2 = \varepsilon_s = 0.03685$, $\varepsilon_{s(GV)} = 0.040 \rightarrow \varepsilon_s / \varepsilon_{s(GV)} = 0.92$

$\varepsilon / \varepsilon_{Limit} = max(0.05, 0.92) = 0.92$

K104 (j) end: $\varepsilon_1 = \varepsilon_c = 0.002022$, $\varepsilon_{cg(GV)} = 0.0135 \rightarrow \varepsilon_c / \varepsilon_{cg(GV)} = 0.15$

$\varepsilon_2 = \varepsilon_s = 0.03036$, $\varepsilon_{s(GV)} = 0.040$ ➔ $\varepsilon_s / \varepsilon_{s(GV)} = 0.76$

$\varepsilon / \varepsilon_{Limit} = max(0.15, 0.76) = 0.76$

Since $\varepsilon / \varepsilon_{Limit} < 1.0$ for the (i) end and (j) end of K104, then the beam satisfies **"GV" Limit State**

4.2.8 Comparison of Column and Beam Section Strains with the Section Strain Limits

The ratios of ε to ε_{Limit} are calculated for all member ends and presented in bar chart form for beams and columns separately in each storey.

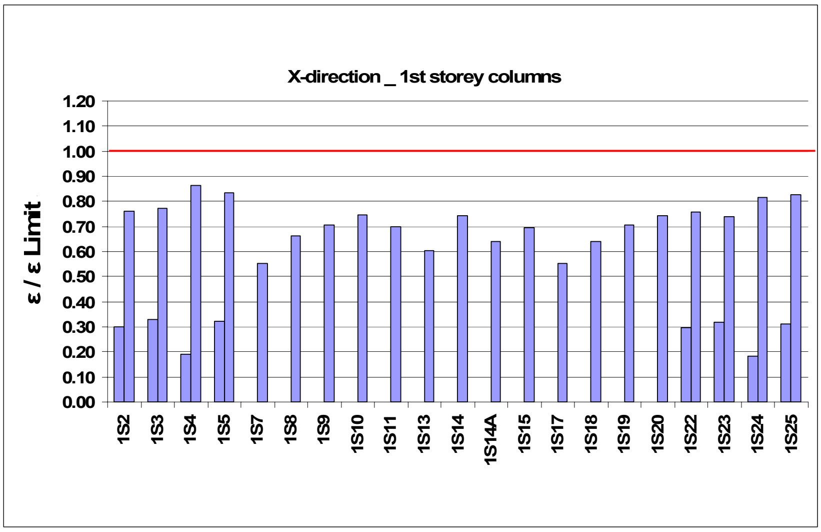


Figure 4.37 ε / ε_{Limit} Values for 1st Storey Columns

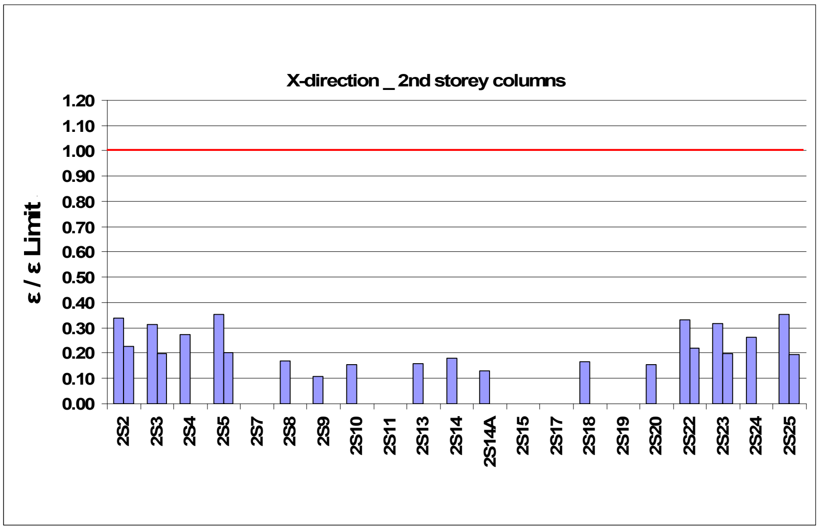


Figure 4.38 ε / ε_{Limit} Values for 2nd Storey Columns

In X – direction, all 3rd and 4th storey columns satisfy the "**GV**" Limit State. Hence the graphics of 3rd and 4th storey columns are not given.

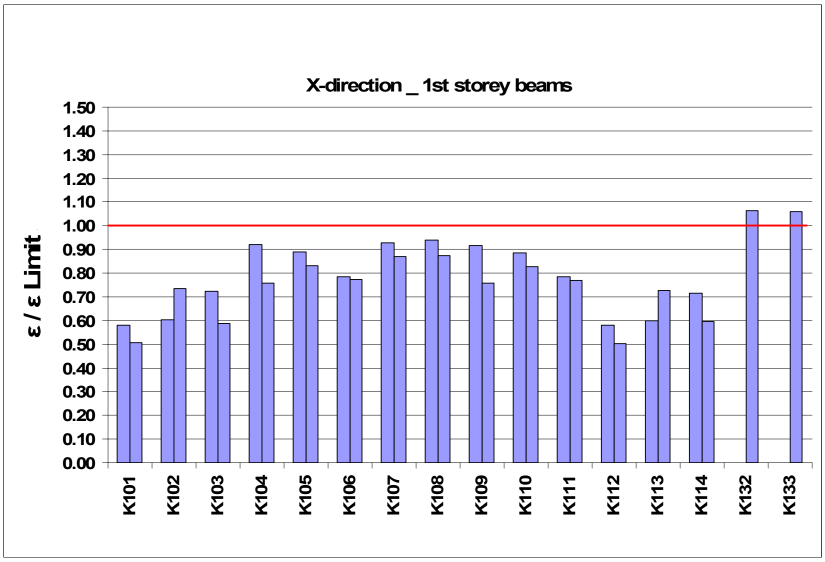


Figure 4.39 ε / ε_{Limit} Values for 1st Storey Beams

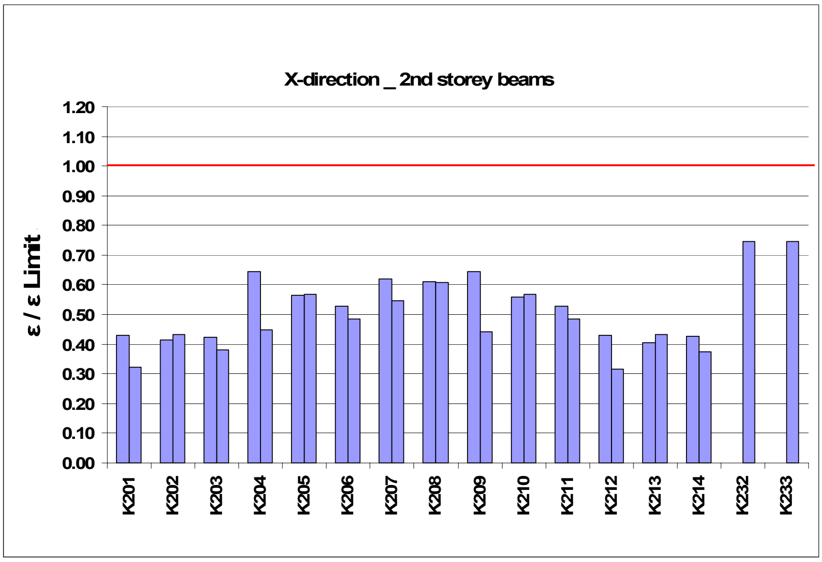


Figure 4.40 $\varepsilon / \varepsilon_{Limit}$ Values for 2nd Storey Beams

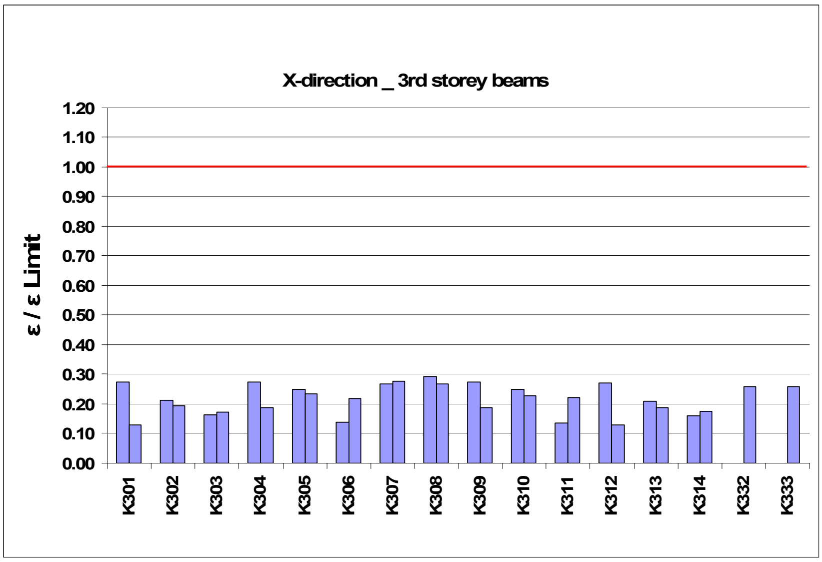


Figure 4.41 $\varepsilon / \varepsilon_{Limit}$ Values for 3rd Storey Beams

In X – direction, all 4th storey beams satisfy the "**GV**" Limit State. Hence the graphics of 4th storey beams are not given.

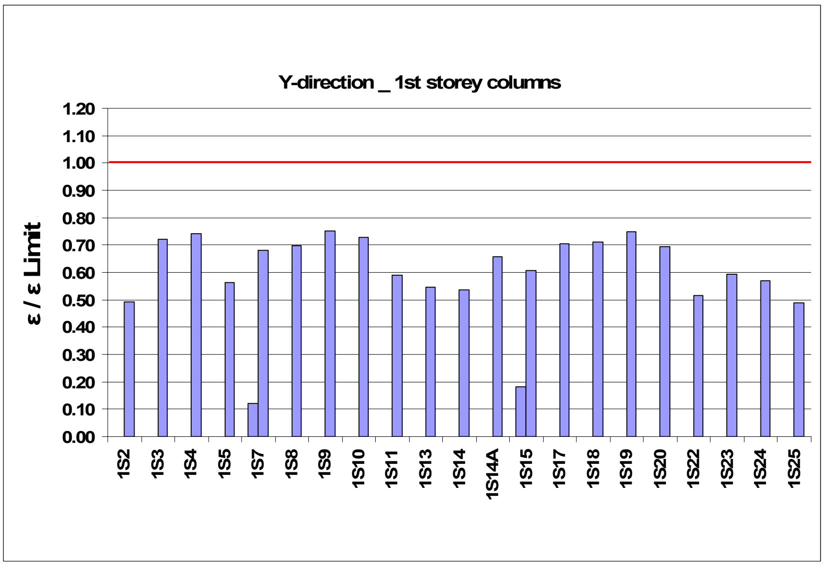


Figure 4.42 $\varepsilon / \varepsilon_{Limit}$ Values for 1st Storey Columns

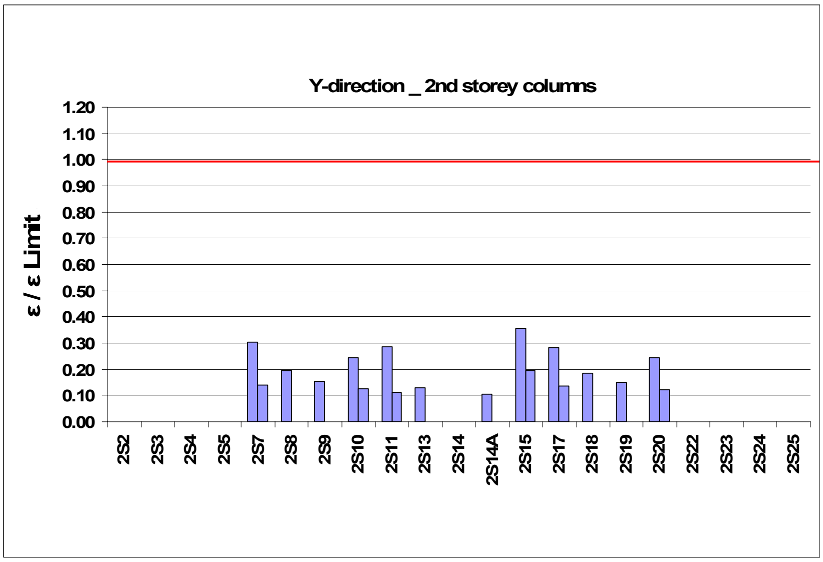


Figure 4.43 $\varepsilon / \varepsilon_{Limit}$ Values for 2nd Storey Columns

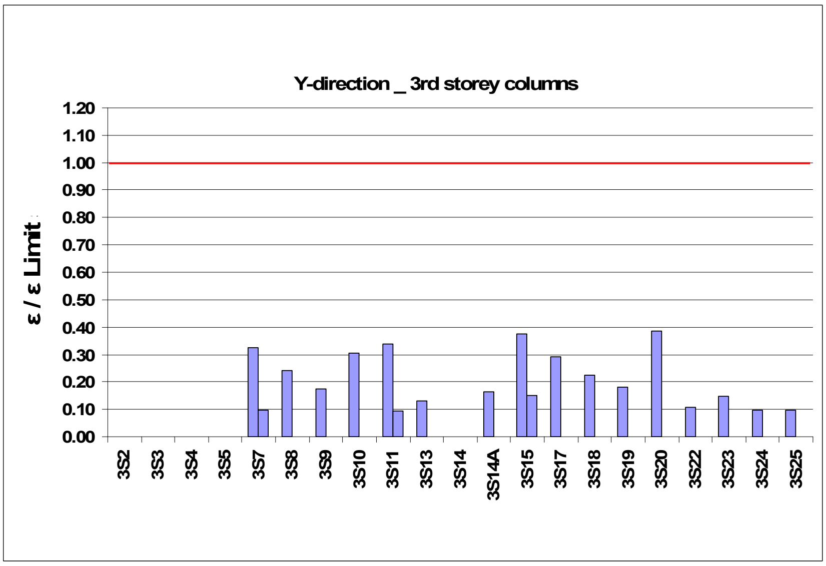


Figure 4.44 $\varepsilon / \varepsilon_{Limit}$ Values for 3rd Storey Columns

In Y – direction, all 4th storey columns satisfy the “**GV**” Limit State. Hence the graphics of 4th storey columns are not given.

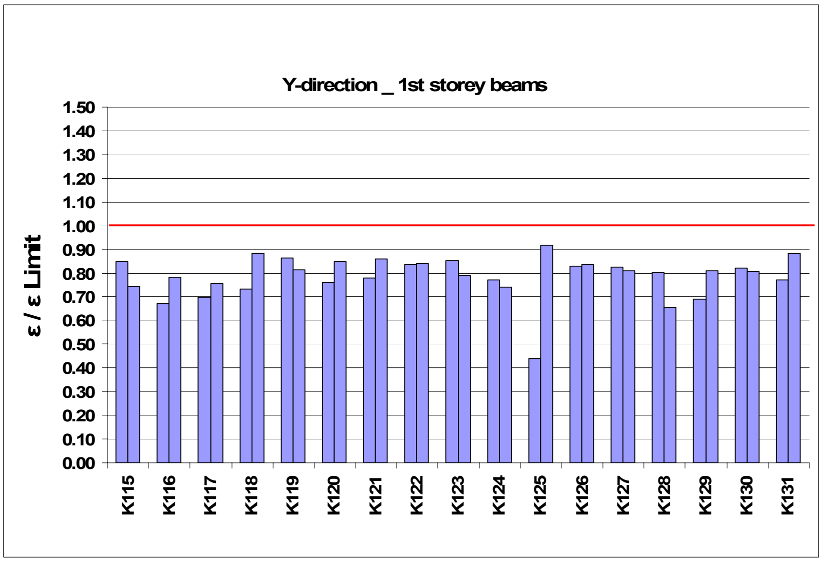


Figure 4.45 $\varepsilon / \varepsilon_{Limit}$ Values for 1st Storey Beams

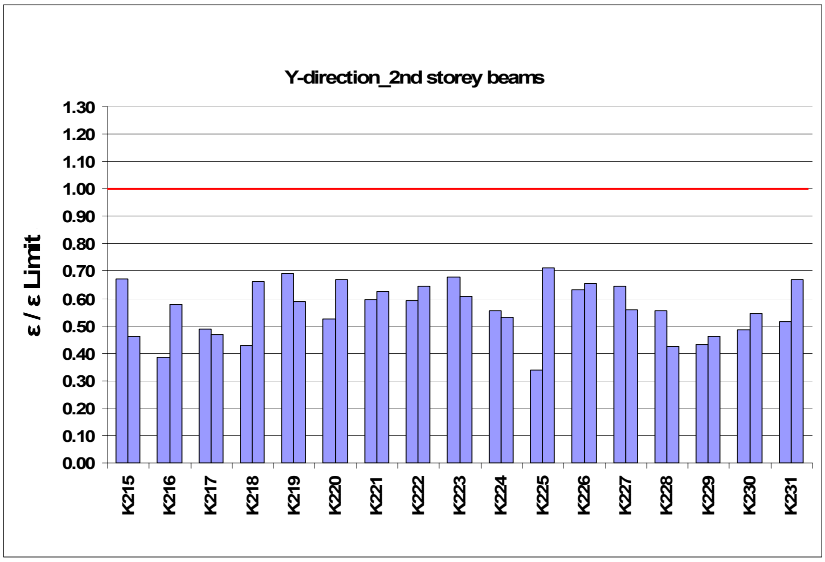


Figure 4.46 $\varepsilon / \varepsilon_{Limit}$ Values for 2nd Storey Beams

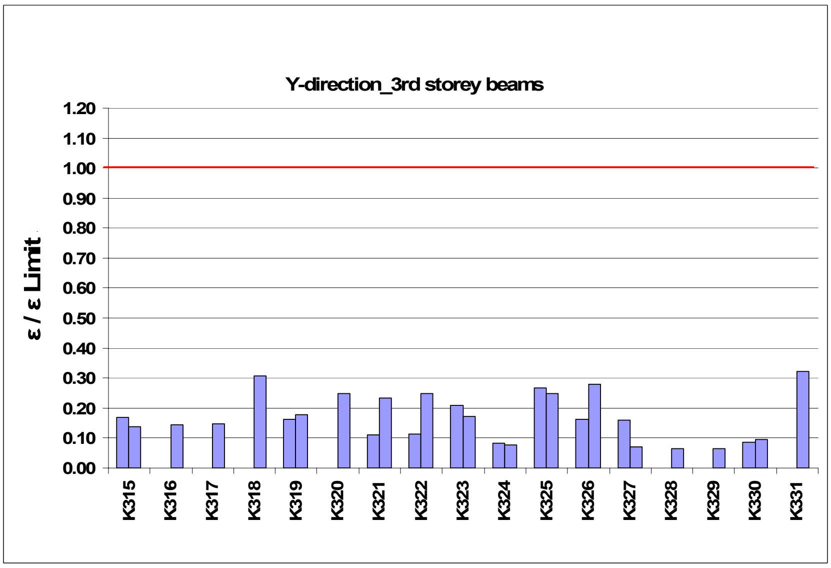


Figure 4.47 $\varepsilon / \varepsilon_{Limit}$ Values for 3rd Storey Beams

In Y – direction, all 4th storey beams satisfy the "**GV**" Limit State. Hence the graphics of 4th storey beams are not given.

4.2.9 Global Performance of the Building

In the table given below, the ratio of the number of unacceptable beams to all beams of the considered storey and in the considered direction, and the ratio of storey shear taken by unacceptable columns are presented.

Table 4.10 Percentage of Unacceptable Beams and Columns

	+X Direction		+Y Direction	
Storey	**Beams (%)**	**Columns (%)**	**Beams (%)**	**Columns (%)**
1	12.5	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0

Since the case study is a residential building, the target performance level is selected as “Life Safety Performance Level” and the calculated interstorey drifts should be less than 0.02. The results are given in the table below.

Table 4.11 Interstorey Drifts

	+X Direction			+Y Direction		
Storey	$H_i(m)$	$(\Delta_i)_{max}$	$(\Delta_i)_{max}$ / H_i	$H_i(m)$	$(\Delta_i)_{max}$	$(\Delta_i)_{max}$ / H_i
1	2.7	0.05368	0.020	2.7	0.05030	0.019
2	2.7	0.04918	0.018	2.7	0.04997	0.019
3	2.7	0.02787	0.010	2.7	0.03002	0.011
4	2.7	0.01014	0.004	2.7	0.00872	0.003

The building is also assessed by nonlinear procedure in FEMA [4] in order to compare the results. The comparative results will be given in Chapter VI.

CHAPTER V

CASE STUDY 3

ASSESSMENT OF A SCHOOL BUILDING BEFORE AND AFTER RETROFITTING

In this case study, a four storey school building is first assessed before retrofitting. Then, the same building after retrofitting is also assessed in order to compare the results.

5.1 Properties of Existing School Building Before Retrofitting

The existing building consists of a moment resisting frame system with shearwalls. The shearwalls are at the outer axes in the short direction, and at the inner axes in the long direction. The shearwalls in the long direction are only at the first storey.

The frame system is made up of 4 axes in the long direction (X-direction), whereas it is made up of 11 axes in the short direction (Y-direction) and symmetrical. Each storey is 3.1m in height. The area of first floor is 642.4m^2, the other floors are 581.3m^2. Each storey has the same structural plan. Typical storey structural plan view (Figure 5.2) and the member dimensions are given below. Slab thicknesses are 10cm. The column and beam dimensions are same at each storey.

Figure 5.1 Front View of the School Building

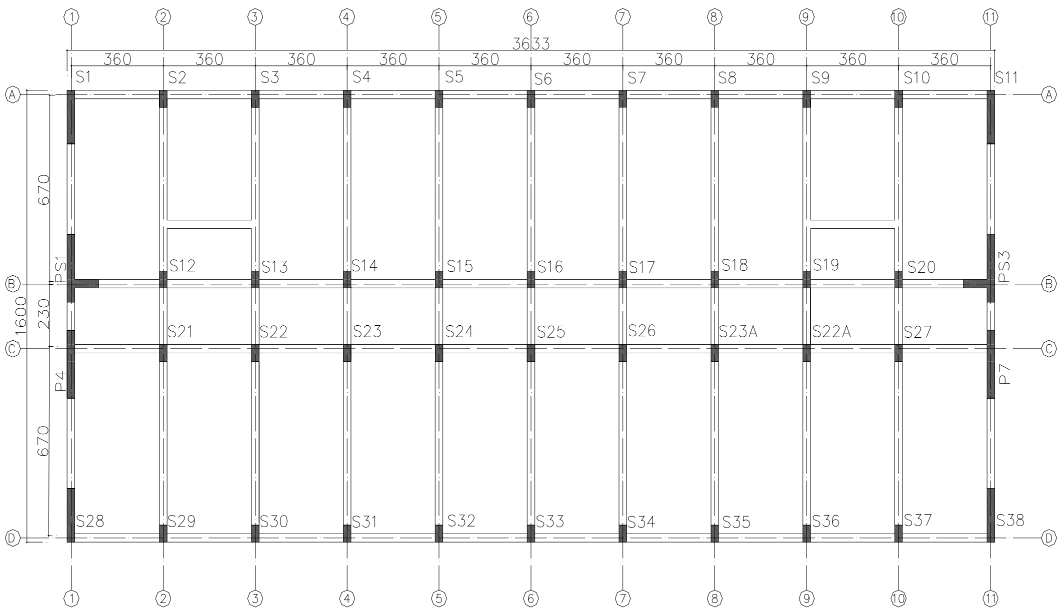


Figure 5.2 Typical Structural Plan View

Corner columns are (S1,S11,S28,S38) 30cmx190cm, all other columns are 30cmx60 cm.

The shearwalls PS1 , PS3, P4 and P7 are 30cmx240 cm.

The shearwalls P5 and P6 are on the C-axis between the columns S22-S23 and S23A-S22A. Their dimensions are 30cmx360cm and they are at first storey only.

In X-direction, the beams between the columns S2-S3 and the columns S9-S10 are 30cmx70cm, all other beams in X-direction are 30cmx50cm.

In Y-direction, the beams between the B–C axes are 30cmx50cm, all other beams in Y-direction are 30cmx70cm.

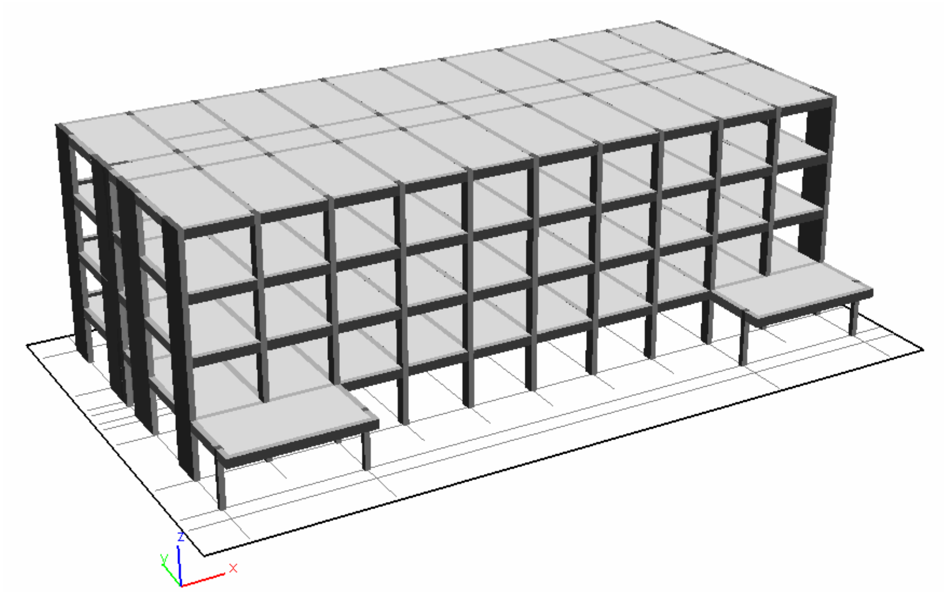


Figure 5.3 Three Dimensional Model

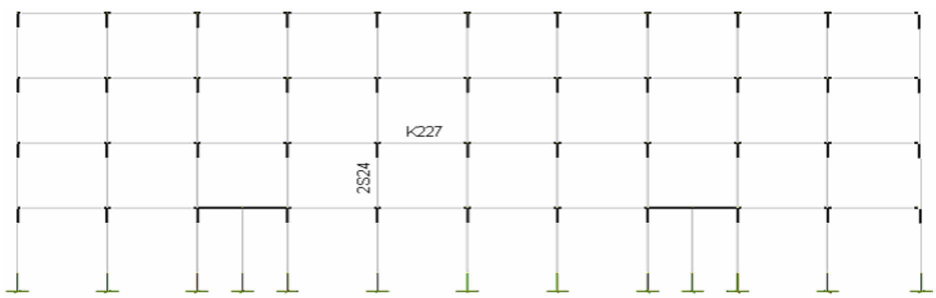


Figure 5.4 Frame C, Rigid End Zones, Example Column and Beam

Storey masses, mass center coordinates and mass moment of inertias are given below (Table 5.1)

Table 5.1 Storey Masses, Mass Center Coordinates and Mass Moment of Inertias

Storey	**Mass (t)**	**Mass Center Coordinates**		**Mass Moment of Inertias ($t.m^2$)**
		X (m)	**Y (m)**	
1	916.30	18.00	7.46	152800.00
2	854.20	18.00	8.22	133900.00
3	854.20	18.00	8.22	133900.00
4	614.50	18.00	8.21	97552.23

Table 5.2 Project and Code Parameters of the Building

Project Parameters of the Existing Building	**1998 Turkish Earthquake Code Parameters**
Project of the Building (Exist / Not exist): *Exist*	Earthquake Zone: *1*
Knowledge Level: *High*	Earthquake Zone Factor: *0.4*
Knowledge Level Factor: *1*	Building Importance Factor:*1.0*
Reinforcement Existing Factor: *1*	Soil Class: *Z3*
Existing Concrete Strength (Mean – Standard Deviation): *8.5 MPa*	
Existing Steel Strength (Mean–Standard Deviation): *220 MPa*	
Target Performance Level: *Life Safety (At 50 years %2), Immediate Occupancy(At 50 years %50)*	
Live Load Participation Factor (n): *0.60*	

5.1.1 Linear Elastic Analysis of Existing School Building Before Retrofitting

The existing school building is only assessed by linear elastic procedures. Since these procedures are explained for the previous case studies in detail, for this case study, the results will be summarized.

5.1.2 Global Performance of the Building

Global performance of the building is summarized in the tables below. These values are obtained from linear elastic analysis for life safety and immediate occupancy performance levels. The values calculated for life safety performance level are given in Table 5.3

Table 5.3 Percentage of Unacceptable Beams and Columns

	+X Direction		**+Y Direction**	
Storey	**Beams (%)**	**Columns (%)**	**Beams (%)**	**Columns (%)**
1	100	95	100	99
2	100	100	100	92
3	100	99	100	64
4	100	94	100	90

5.2 Properties of Existing School Building After Retrofitting

The existing building is retrofitted with shearwalls. The added shearwalls are at the outer axes in long direction and at inner axes in short direction. The dimensions of these shearwalls are given below.

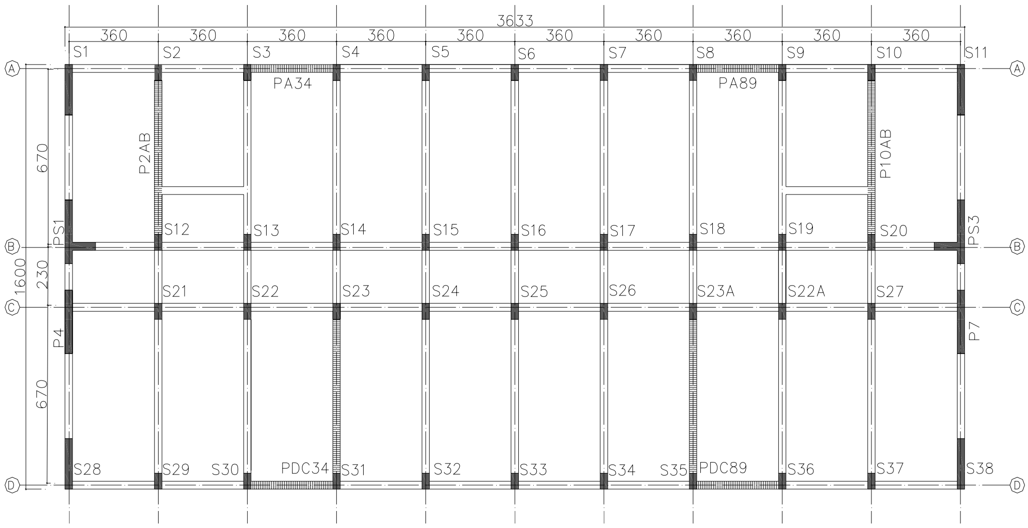


Figure 5.5 Typical Structural Plan View

The added shearwalls are shown in dashed form in the Figure 5.5

The shearwalls PA34 and PA89 are 25cmx390cm. P2AB and P10AB are 25cmx700 cm.

PDC34 and PDC89 are modeled as L-shaped shearwalls.

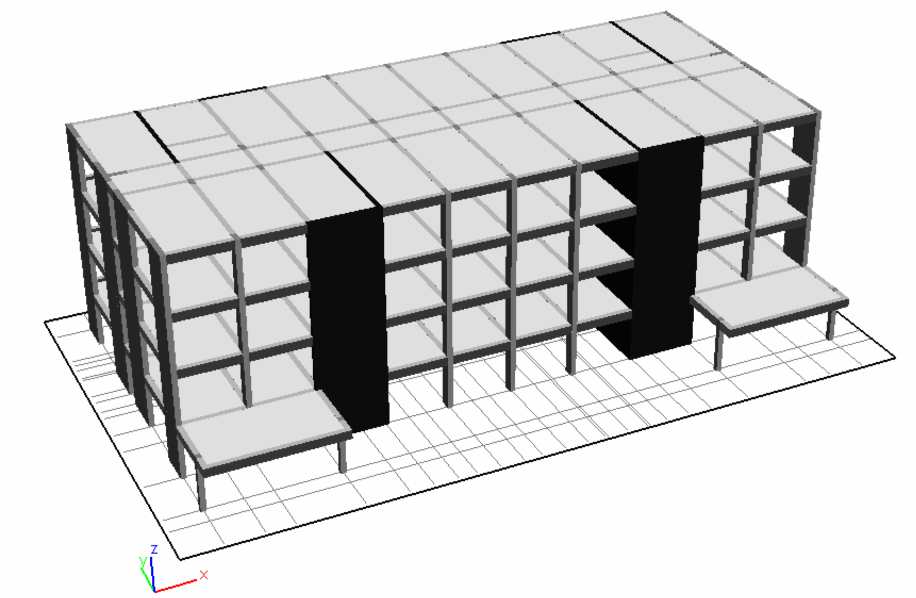


Figure 5.6 Three Dimensional Model

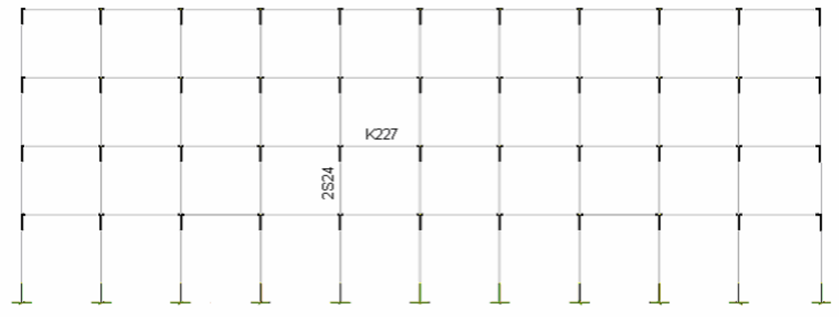


Figure 5.7 Frame C, Rigid End zones, Example Column and Beam

In order to prevent irregularity in plan, the existing shearwalls P5 and P6 which are at first storey only, are demolished.

Storey masses, mass center coordinates and mass moment of inertias are given below (Table 5.1)

Table 5.4 Storey Masses, Mass Center Coordinates and Mass Moment of Inertias

Storey	**Mass (t)**	**Mass Center Coordinates**		**Mass Moment of Inertias ($t.m^2$)**
		X (m)	**Y (m)**	
1	916.30	18.00	7.46	152800.00
2	854.20	18.00	8.22	133900.00
3	854.20	18.00	8.22	133900.00
4	614.50	18.00	8.21	97552.23

Table 5.5 Project and Code Parameters of the Building

Project Parameters of the Existing Building	**1998 Turkish Earthquake Code Parameters**
Project of the Building (Exist / Not exist): *Exist*	Earthquake Zone: *1*
Knowledge Level: *High*	Earthquake Zone Factor: *0.4*
Knowledge Level Factor: *1*	Building Importance Factor:*1.0*
Reinforcement Existing Factor: *1*	Soil Class: *Z3*
Existing / Added Member Concrete Strength (Mean–Standard Deviation): *8.5 MPa / 20 MPa*	
Existing / Added Member Steel Strength (Mean –Standard Deviation):*220 MPa / 420 MPa*	
Target Performance Level: *Life Safety (At 50 years %2), Immediate Occupancy(At 50 years %50)*	
Live Load Participation Factor (n): *0.60*	

5.2.1 Linear Elastic Analysis of Existing School Building After Retrofitting

The retrofitted school building is first assessed by linear elastic procedure. The results will be summarized in the following Chapter 5.2.2

5.2.2 Global Performance of the Building

Global performance of the building is summarized in the tables below. These values are obtained from linear elastic procedure for life safety performance level.

Table 5.6 Percentage of Unacceptable Beams and Columns

	+X Direction		+Y Direction	
Storey	**Beams (%)**	**Columns (%)**	**Beams (%)**	**Columns (%)**
1	**91**	8	**93**	13
2	**92**	13	**100**	0
3	**94**	17	**100**	0
4	**89**	35	**79**	8

Table 5.7 Interstorey Drifts

	+X Direction		+Y Direction	
Storey	$H_i(m)$	$(\Delta_i)_{max} / H_i$	$H_i(m)$	$(\Delta_i)_{max} / H_i$
1	3.1	0.00210	3.1	0.00069
2	3.1	0.00365	3.1	0.00105
3	3.1	0.00406	3.1	0.00110
4	3.1	0.00376	3.1	0.00096

5.2.3 Non-linear Elastic Analysis of Existing School Building After Retrofitting

The retrofitted school building is then assessed by non-linear procedures. The results are summarized in the following Section 5.2.4

5.2.4 Global Performance of the Building

Global performance of the building is summarized in the tables below. These values are obtained from non-linear procedure for life safety performance level.

Table 5.8 Percentage of Unacceptable Beams and Columns

	+X Direction		**+Y Direction**	
Storey	**Beams (%)**	**Columns (%)**	**Beams (%)**	**Columns (%)**
1	0	0	17	0
2	3	0	14	0
3	3	0	17	0
4	3	0	0	0

Table 5.9 Interstorey Drifts

	+X Direction		**+Y Direction**	
Storey	$H_i(m)$	$(\Delta_i)_{max} / H_i$	$H_i(m)$	$(\Delta_i)_{max} / H_i$
1	3.1	0.00675	3.1	0.00349
2	3.1	0.00967	3.1	0.00429
3	3.1	0.01016	3.1	0.00451
4	3.1	0.01020	3.1	0.00452

The building is also assessed by nonlinear procedure in FEMA[4] in order to compare the results. The comparative results will be given in Chapter VI.

CHAPTER VI

DISCUSSION OF RESULTS

The linear elastic procedure and nonlinear procedure in the 2006 Turkish Earthquake Code are implemented to different buildings. In addition to these procedures, the nonlinear procedure in FEMA [4] is also applied to these buildings as a reference in comparative evaluation. In this chapter, the results of the procedures defined above are presented comparatively for the selected buildings.

First building is a residential building, which was designed according to the 1998 Turkish Earthquake Code [10]. In the graphical presentations, this building is abbreviated as ” RESIDENTIAL 1998 ”. Second building is also a residential building, which was designed according to the 1975 Turkish Earthquake Code [11]. This building is abbreviated as “ RESIDENTIAL 1975 “. Finally, the third building is a retrofitted school building, which is abbreviated as “ SCHOOL RETROFITTED “

a) RESIDENTIAL 1998

The nonlinear procedure in FEMA is not applied to the first building, however the other two procedures in the Turkish Code are applied. The results obtained from the first storey columns of this building in +X and +Y directions are shown below in Figure 6.1 and Figure 6.2. It can be observed that both procedures lead to acceptable results for the columns of a building designed to the 1998 Turkish Earthquake Code [10]. However, the linear procedure is more conservative as expected.

In the +X direction, the average of (r / r_{Limit}) / (ε / $ε_{Limit}$) values is calculated for the bottom ends of the first storey columns, and determined as 1.37. In the +Y direction, the same average value is calculated as 1.42. Hence, it can be concluded that linear elastic procedure is about %40 more conservative than the nonlinear procedure for yielding columns.

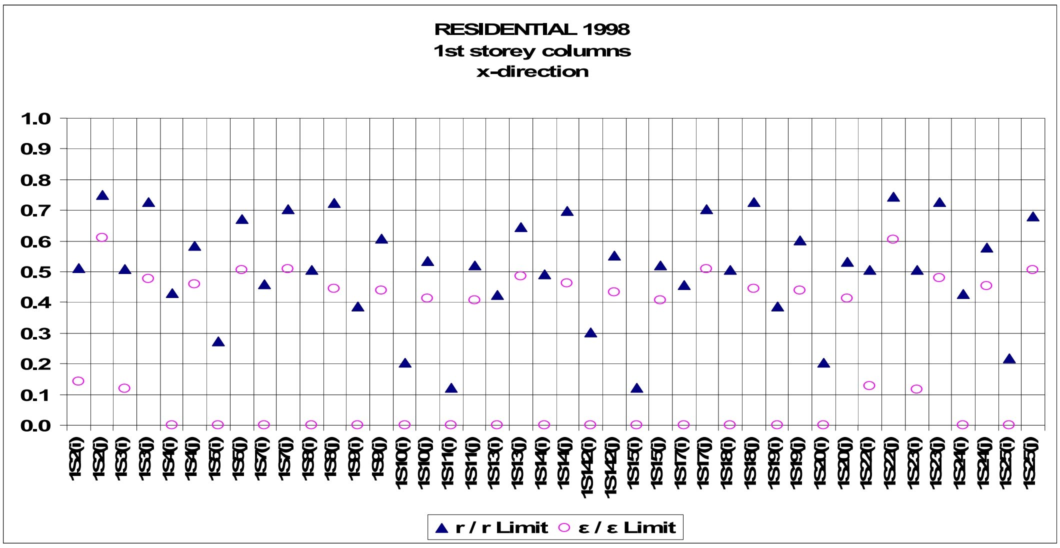


Figure 6.1 First Storey Columns of RESIDENTIAL 1998 in X Direction

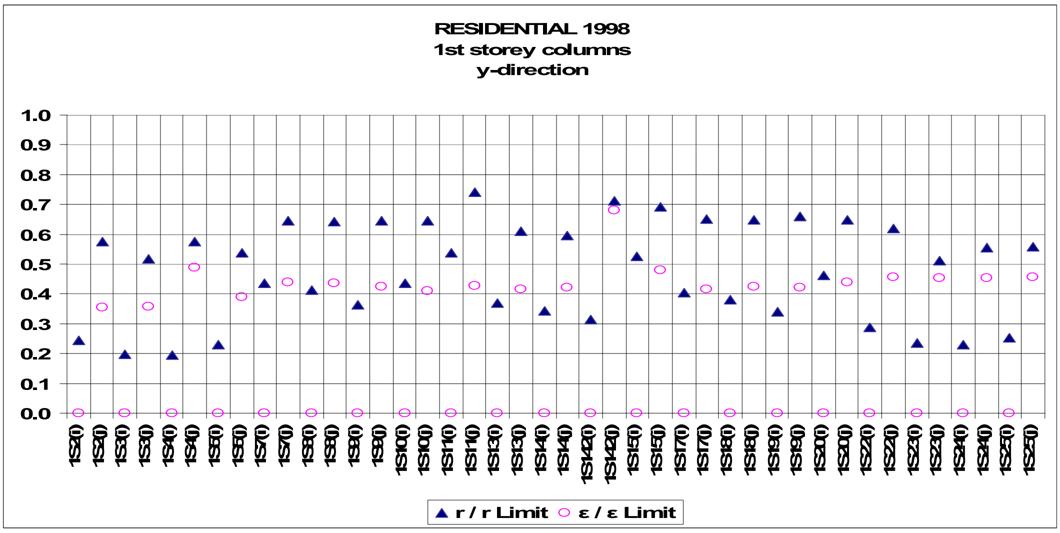


Figure 6.2 First Storey Columns of RESIDENTIAL 1998 in Y Direction

In the + X and + Y directions, similar average values are also calculated for the beams of the first three stories. In the +X direction, the average value for the first, second and third stories are 1.03, 1.28 and 2.15 respectively. In the +Y direction, the average values for the first, second and third stories are 1.02, 1.16 and 2.06 respectively. However, all beams are acceptable according to both procedures. The linear elastic procedure perhaps overestimates the demands in the upper stories.

The comparative figures for beams are given below in Figure 6.3 to Figure 6.8.

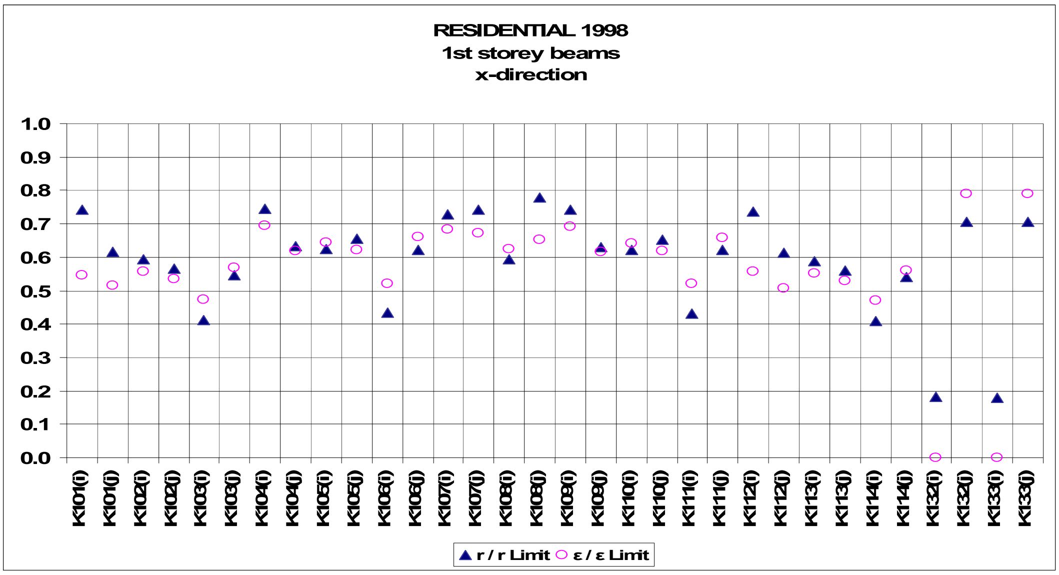


Figure 6.3 First Storey Beams of RESIDENTIAL 1998 in X Direction

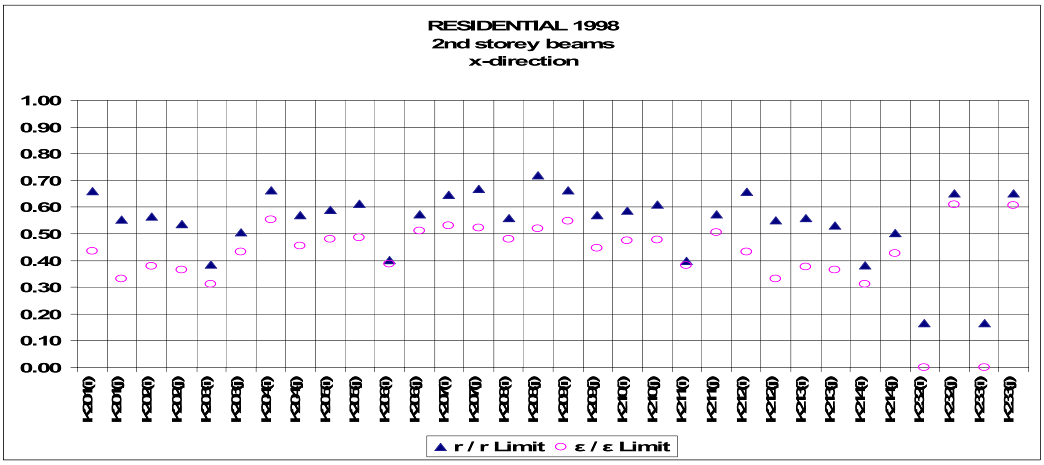


Figure 6.4 Second Storey Beams of RESIDENTIAL 1998 in X Direction

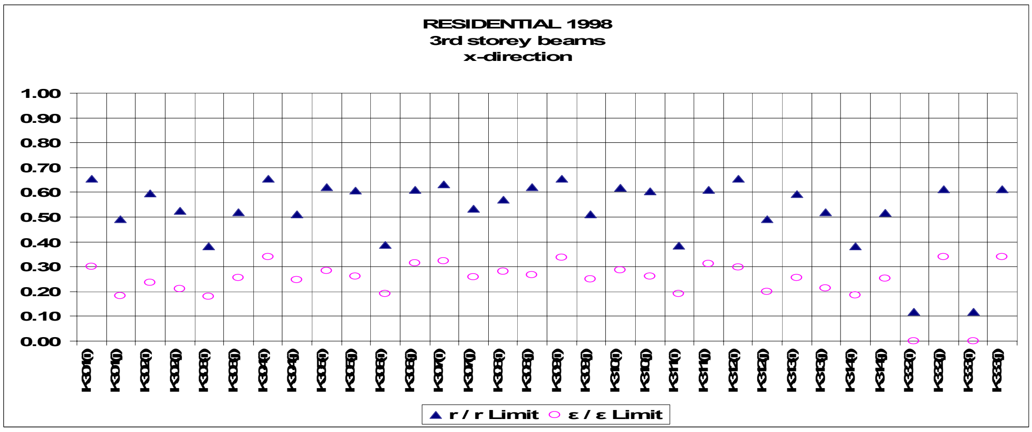


Figure 6.5 Third Storey Beams of RESIDENTIAL 1998 in X Direction

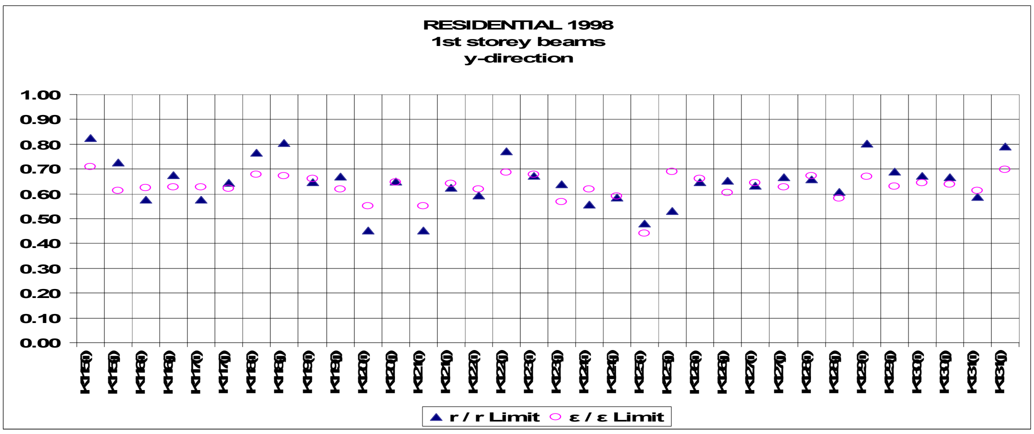


Figure 6.6 First Storey Beams of RESIDENTIAL 1998 in Y Direction

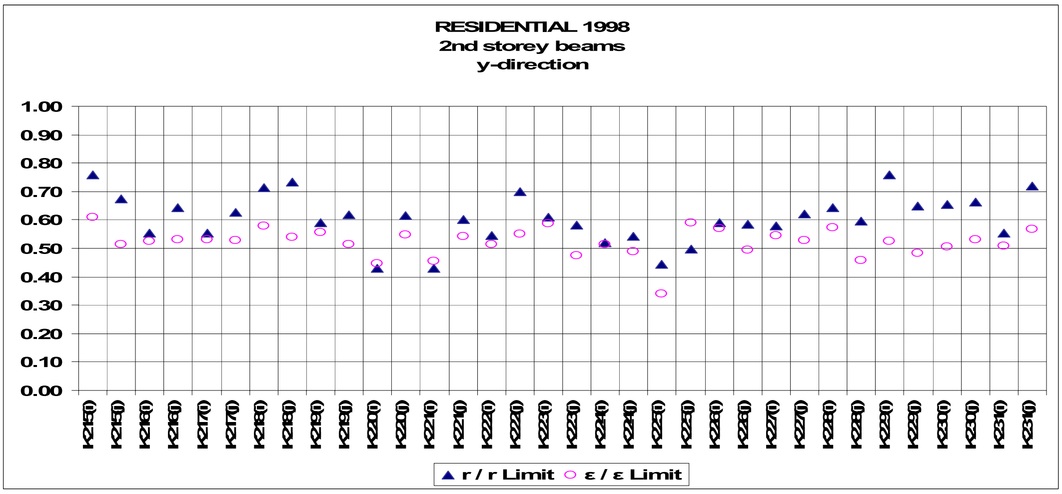


Figure 6.7 Second Storey Beams of RESIDENTIAL 1998 in Y Direction

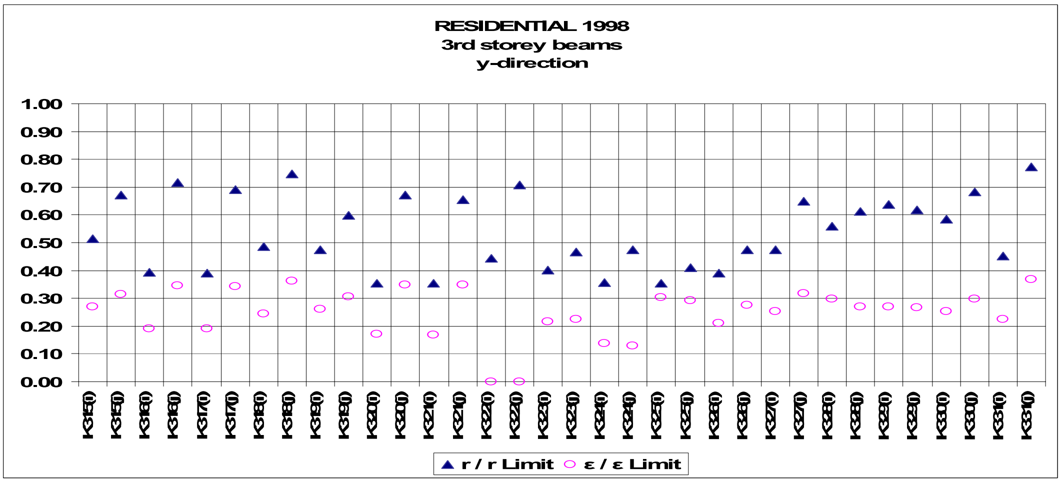


Figure 6.8 Third Storey Beams of RESIDENTIAL 1998 in Y Direction

b) RESIDENTIAL 1975

In addition to two procedures in the 2006 Turkish Earthquake Code, the nonlinear procedure in FEMA is also applied to the second building. The results belonging to the first storey columns of this building in +X and +Y directions are given below in Figure 6.9 and Figure 6.10.

In the +X direction, the average of (r / r_{Limit}) / (ε / ε_{Limit}) and (θp / θp_{Limit}) / (ε / ε_{Limit}) values are calculated for the bottom ends of the first storey columns. The average of (r / r_{Limit}) / (ε / ε_{Limit}) values is determined as 1.21 and the average of (θp / θp_{Limit}) / (ε / ε_{Limit}) values is determined as 1.67. In the +Y direction, these average values are calculated as 1.34 and 1.70, respectively.

Hence, it can be concluded that the acceptance limits for conforming columns in FEMA are about %70 more conservative than those in the 2006 Turkish Earthquake Code for nonlinear procedures. However, FEMA is also more conservative compared to the linear procedure in the Turkish Code. It is interesting to note that columns of a building satisfying the 1975 Code are not accepted by FEMA whereas they are acceptable according to both procedures in the 2006 Turkish Code.

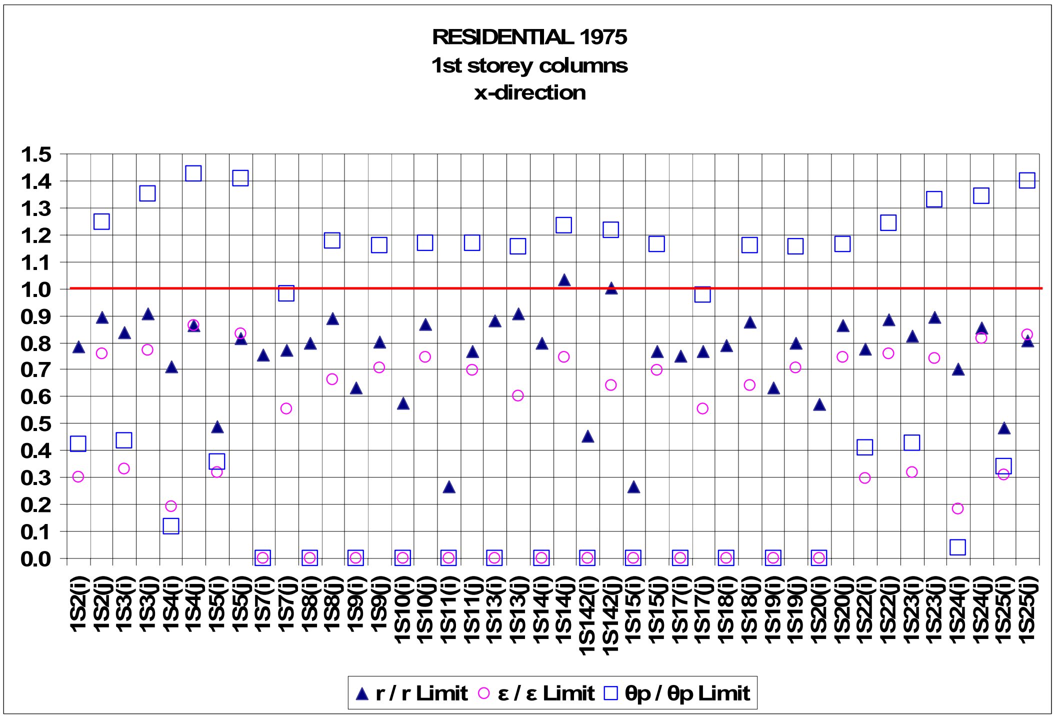


Figure 6.9 First Storey Columns of RESIDENTIAL 1975 in X Direction

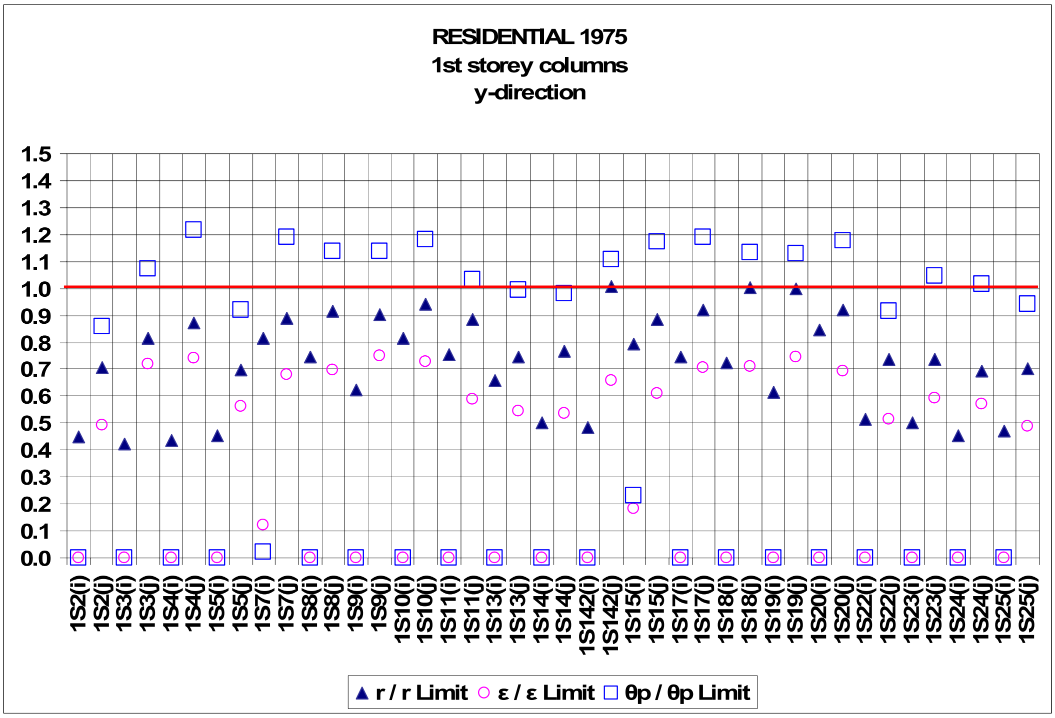


Figure 6.10 First Storey Columns of RESIDENTIAL 1975 in Y Direction

In the + X and + Y directions, the average of (r / r_{Limit}) / (ε / ε_{Limit}) and (θp / θp_{Limit}) / (ε / ε_{Limit}) values are also calculated for the beams of the first two stories. In the +X direction, the average value of (r / r_{Limit}) / (ε / ε_{Limit}) for the first storey is 1.24 and1.69 for the second storey, whereas the average value of (θp / θp_{Limit}) / (ε / ε_{Limit}) for the first storey is 1.15 and 1.10 for the second storey. In the +Y direction, the average values are 1.21, 1.55 for the first storey beams and 1.20, 1.17 for the second storey beams respectively.

The comparative figures for beams are given below in Figure 6.11 to Figure 6.14.

These results indicate that the linear elastic procedure in the 2006 Turkish Code is slightly more conservative than the nonlinear procedure in the same code for the first storey beams which undergo large post elastic response. The difference increases in the upper stories, probably due to overestimation of demands by the linear elastic procedure. On the other hand, the comparison of nonlinear procedures in the 2006 Code and FEMA for confined beams lead to closer agreement, where FEMA is about %20 more conservative.

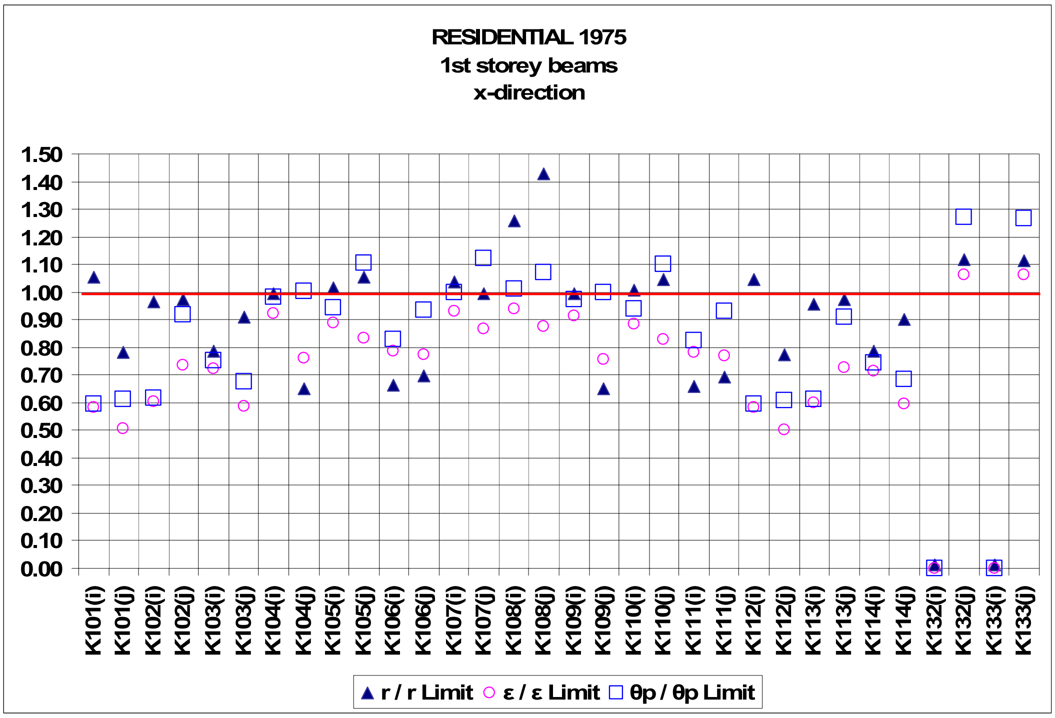


Figure 6.11 First Storey Beams of RESIDENTIAL 1975 in X Direction

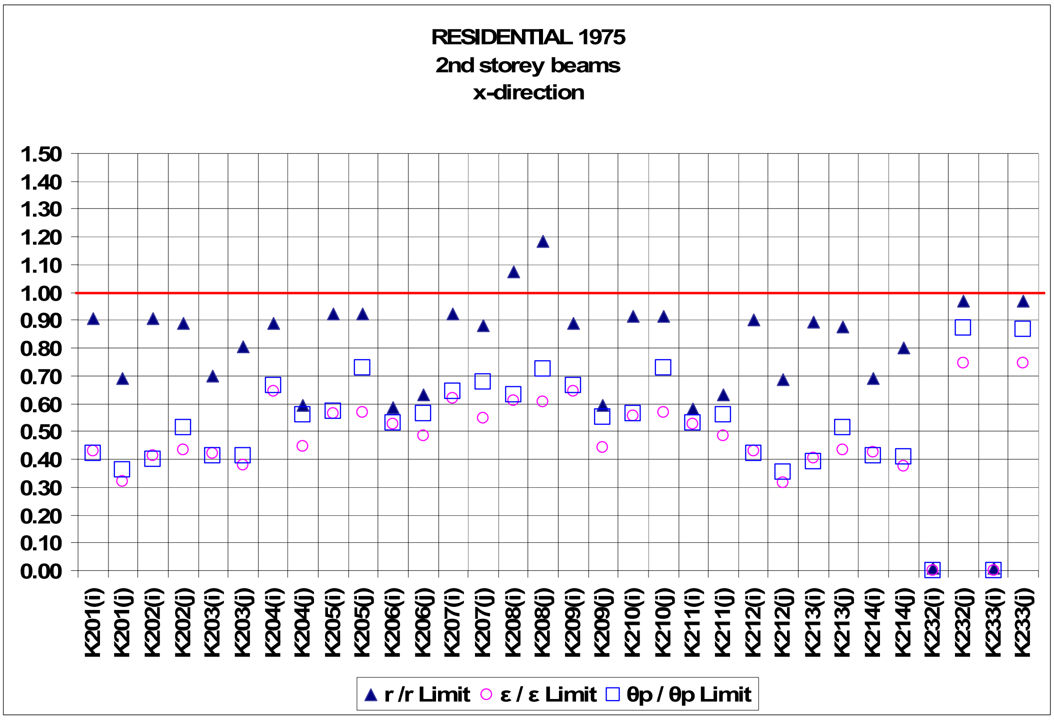


Figure 6.12 Second Storey Beams of RESIDENTIAL 1975 in X Direction

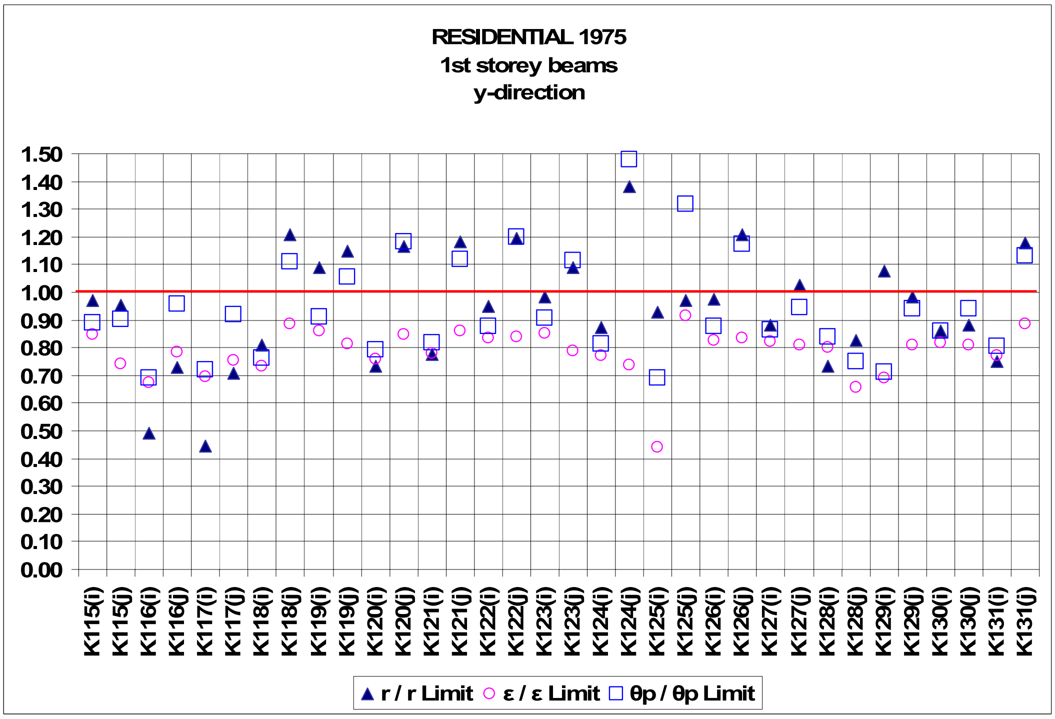


Figure 6.13 First Storey Beams of RESIDENTIAL 1975 in Y Direction

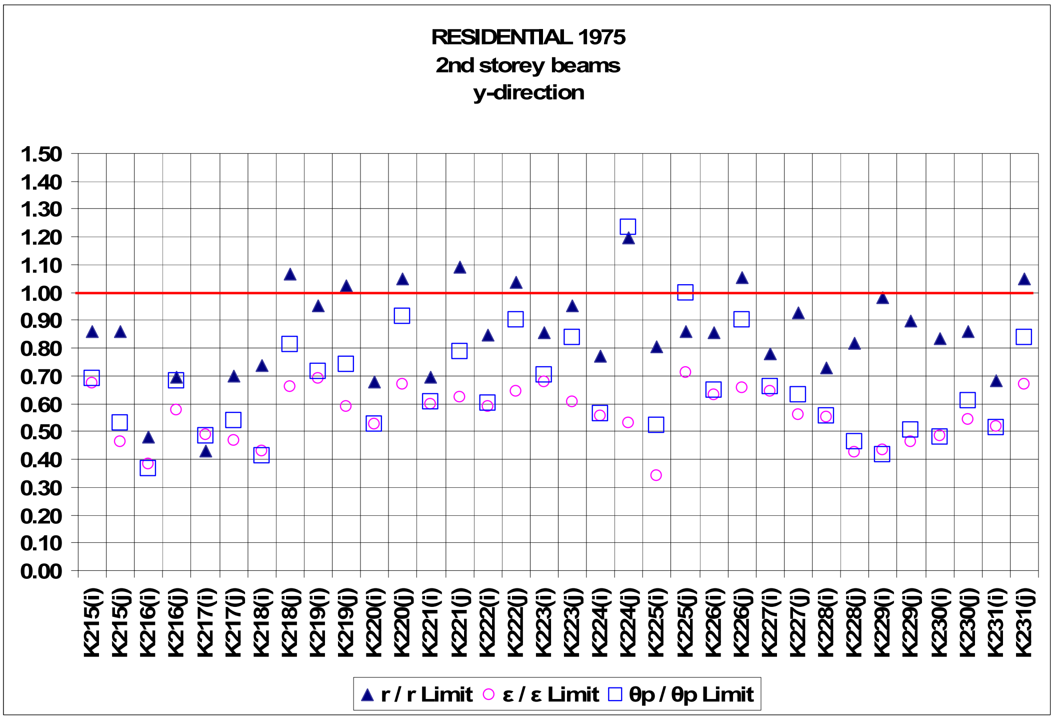


Figure 6.14 Second Storey Beams of RESIDENTIAL 1975 in Y Direction

c) SCHOOL RETROFITTED

The third building is also assessed by both procedures in the 2006 Turkish Earthquake Code and the nonlinear procedure in FEMA. The results belonging to the first storey columns of this building in the +X and +Y directions are given below in Figure 6.15 and Figure 6.16

In the +X direction, the average of (r / r_{Limit}) / (ε / ε_{Limit}) and (θp / θp_{Limit}) / (ε / ε_{Limit}) values are calculated for the bottom ends of the first storey columns. The average of (r / r_{Limit}) / (ε / ε_{Limit}) values is determined as 1.73 and the average of (θp / θp_{Limit}) / (ε / ε_{Limit}) values is determined as 3.35.

In the +Y direction, the same average values are calculated as 1.29 and 2.37, respectively.

This building also consists of shearwalls. Same calculations are performed for the first storey shearwalls.

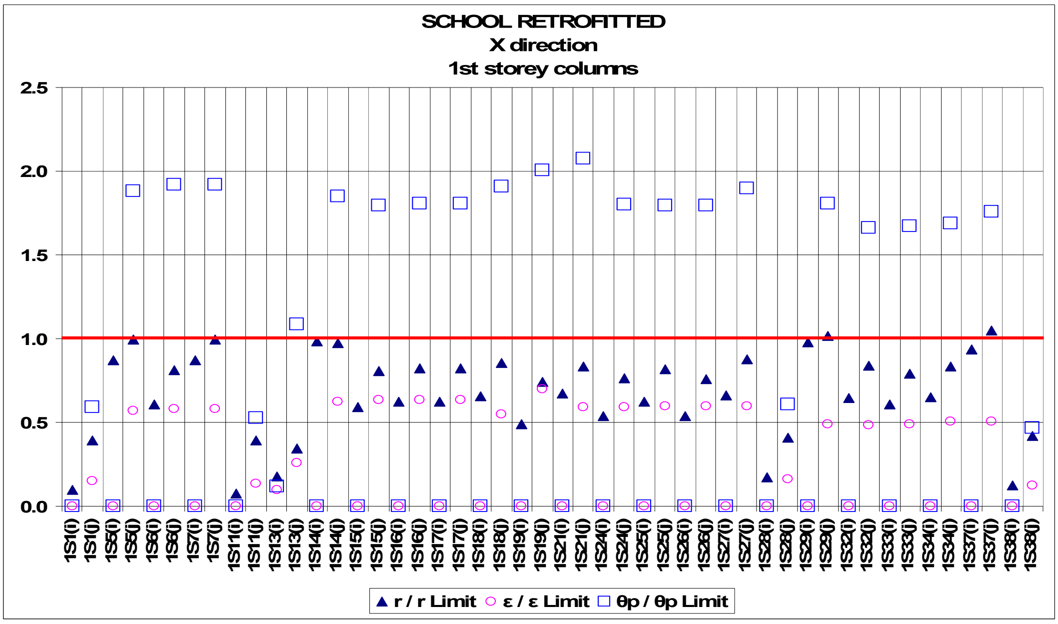


Figure 6.15 First Storey Columns of SCHOOL RETROFITTED in X Direction

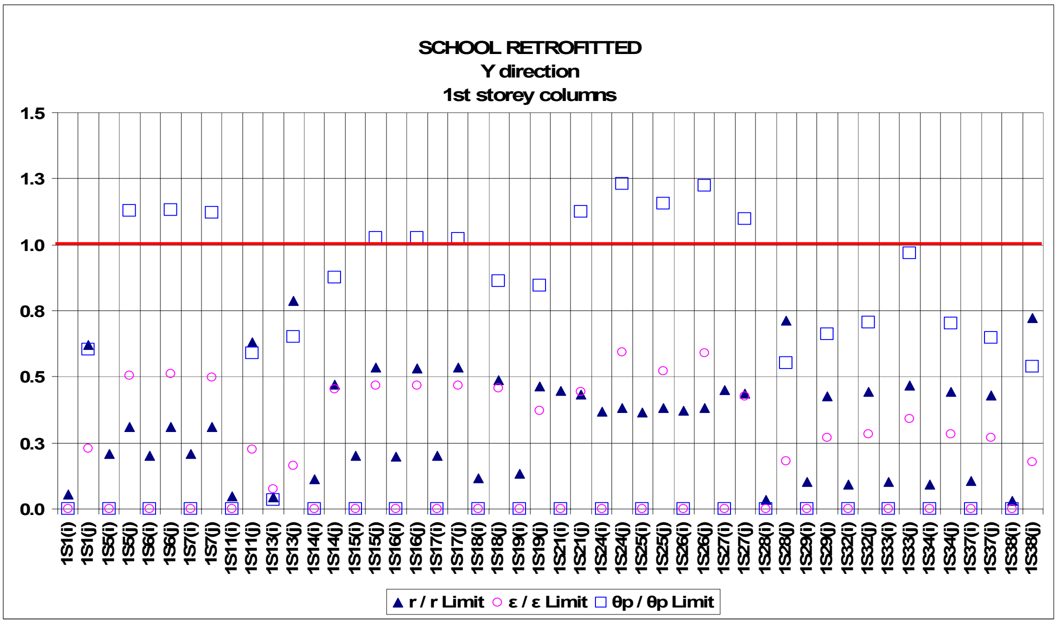


Figure 6.16 First Storey Columns of SCHOOL RETROFITTED in Y Direction

The existing shearwalls (1PS1, 1PS3, 1P4, 1P7) were unconfined, whereas the added shearwalls (1PA34, 1PA89, 1PD34, 1PD89) were confined. The comparative figures for shearwalls are given below in Figure 6.17 and Figure 6.18

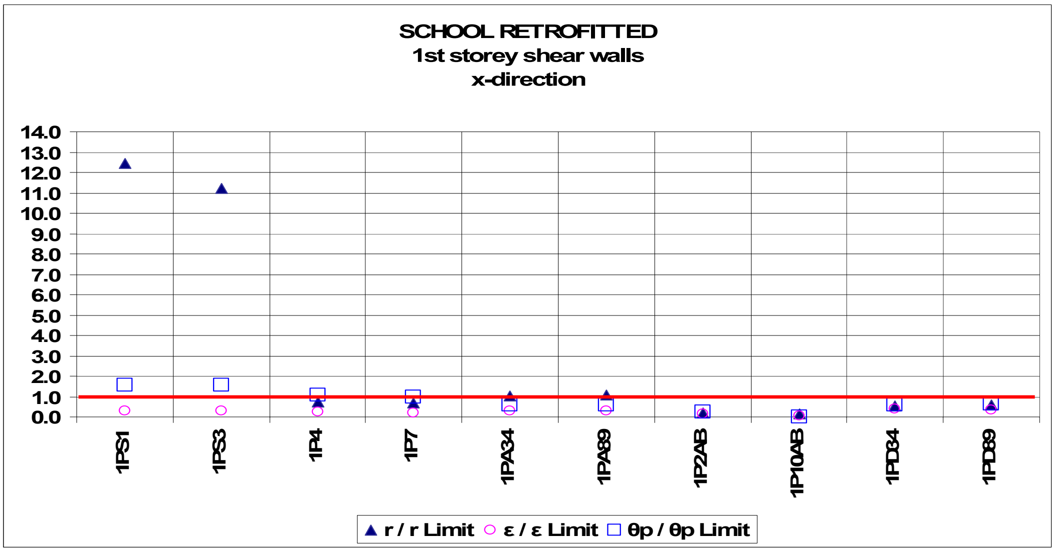


Figure 6.17 First Storey Shearwalls of SCHOOL RETROFITTED in X Direction

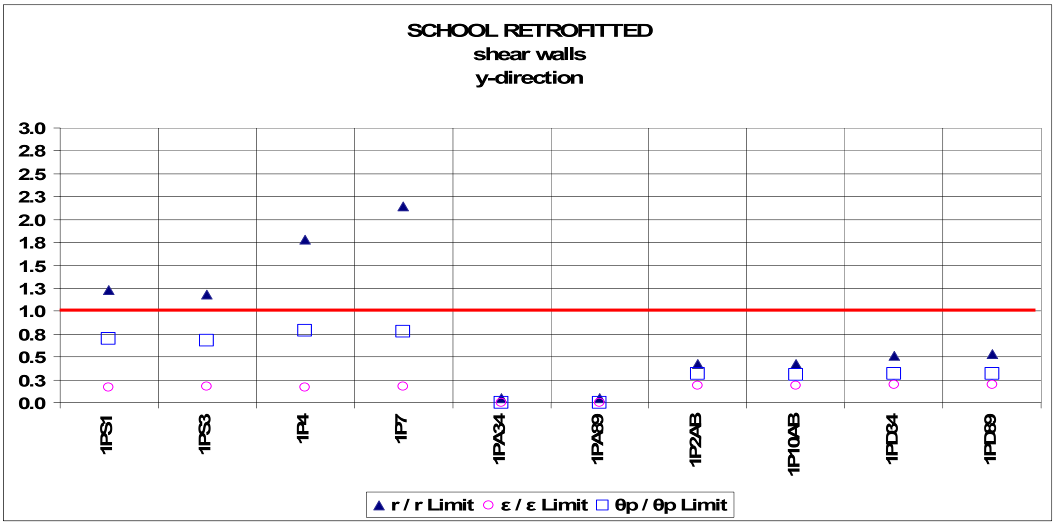


Figure 6.18 First Storey Shearwalls of SCHOOL RETROFITTED in Y Direction

In the +X direction, the average of $(r / r_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ values are calculated as 22.3 for unconfined shearwalls and 2.30 for confined shearwalls; whereas the average of $(\theta p / \theta p_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ values are calculated as 5.13 for unconfined shearwalls and 1.47 for confined shearwalls.

In the +Y direction, the average $(r / r_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ values are calculated as 9.30 for unconfined shearwalls and 2.44 for confined shearwalls; and respectively $(\theta p / \theta p_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ values are calculated as 4.32 for unconfined shearwalls and 1.62 for confined shearwalls.

In the + X and + Y directions, the average of $(r / r_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ and $(\theta p / \theta p_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ values are also calculated for the beams of all stories. In +X direction, the average value of $(r / r_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ for first storey is 5.23 second storey is 7.36, third storey is 10.92 and fourth storey is 4.12 whereas the average value of $(\theta p / \theta p_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ for the first storey is 1.99, second storey is 2.02, third storey is 2.01 and fourth storey is 1.94. In +Y direction, the average values are 4.82, 11.79, 8.94, 4.96 for $(r / r_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ and for $(\theta p / \theta p_{Limit}) / (\varepsilon / \varepsilon_{Limit})$ 1.83, 2.46, 2.29, 2.29 respectively.

The comparative figures for beams are given below in Figure 6.19 to Figure 6.26

These results indicate that non-conforming columns in the post elastic range are assessed very differently by the nonlinear procedures in the 2006 Turkish Code and FEMA. FEMA is 2.5 – 3 times more conservative than the Turkish Code. Linear elastic procedure in the 2006 Turkish Code, on the other hand, is 1.5 – 2 times more conservative for non-conforming columns than the nonlinear procedure. This order also prevails in the comparative evaluation of shearwalls and beams that do not satisfy the confinement requirements.

For the newly added shearwalls satisfying the code design requirements, linear elastic procedure in the 2006 Turkish Code is at least twice more conservative than the nonlinear procedure whereas FEMA nonlinear procedure is also about 1.5 times more conservative than the 2006 Turkish Code.

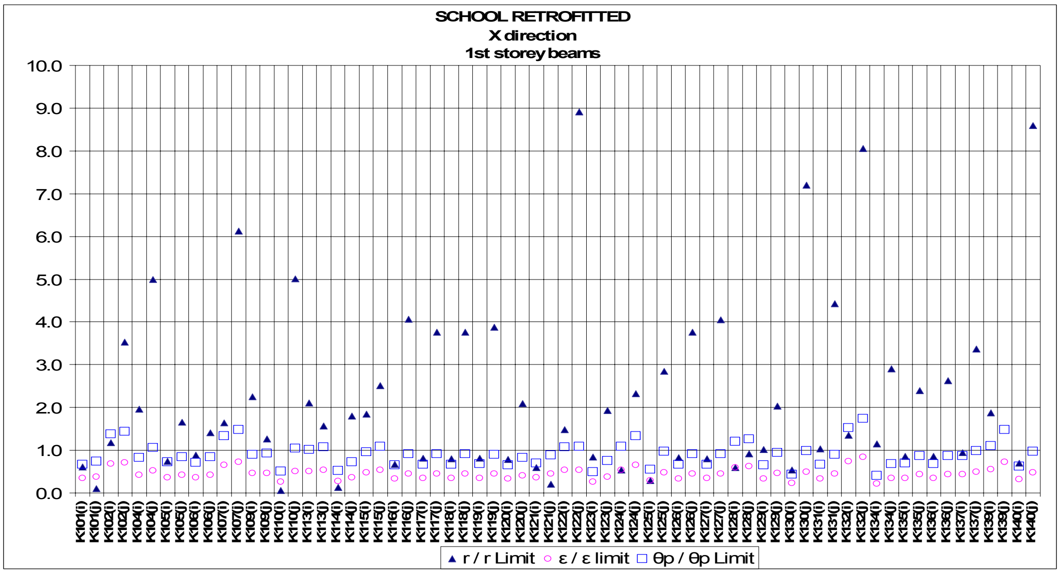


Figure 6.19 First Storey Beams of SCHOOL RETROFITTED in X Direction

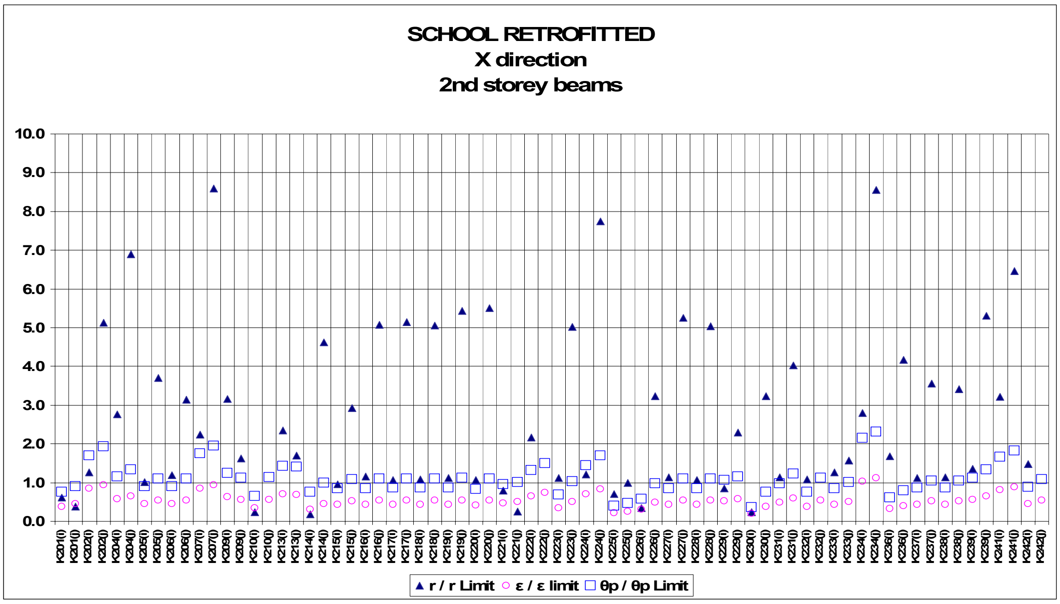


Figure 6.20 Second Storey Beams of SCHOOL RETROFITTED in X Direction

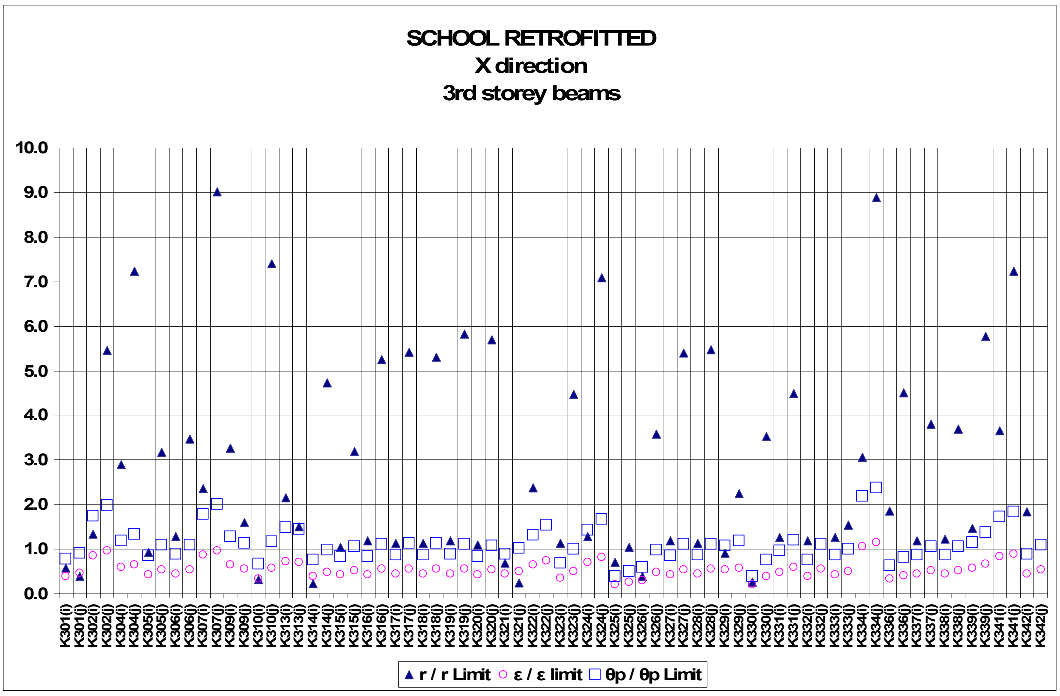


Figure 6.21 Third Storey Beams of SCHOOL RETROFITTED in X Direction

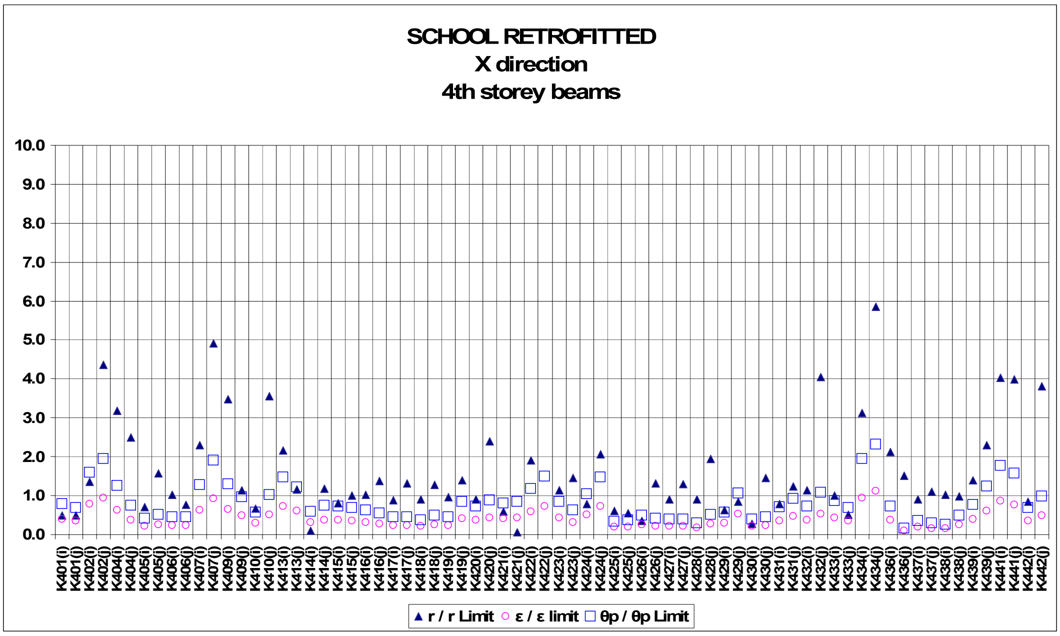


Figure 6.22 Fourth Storey Beams of SCHOOL RETROFITTED in X Direction

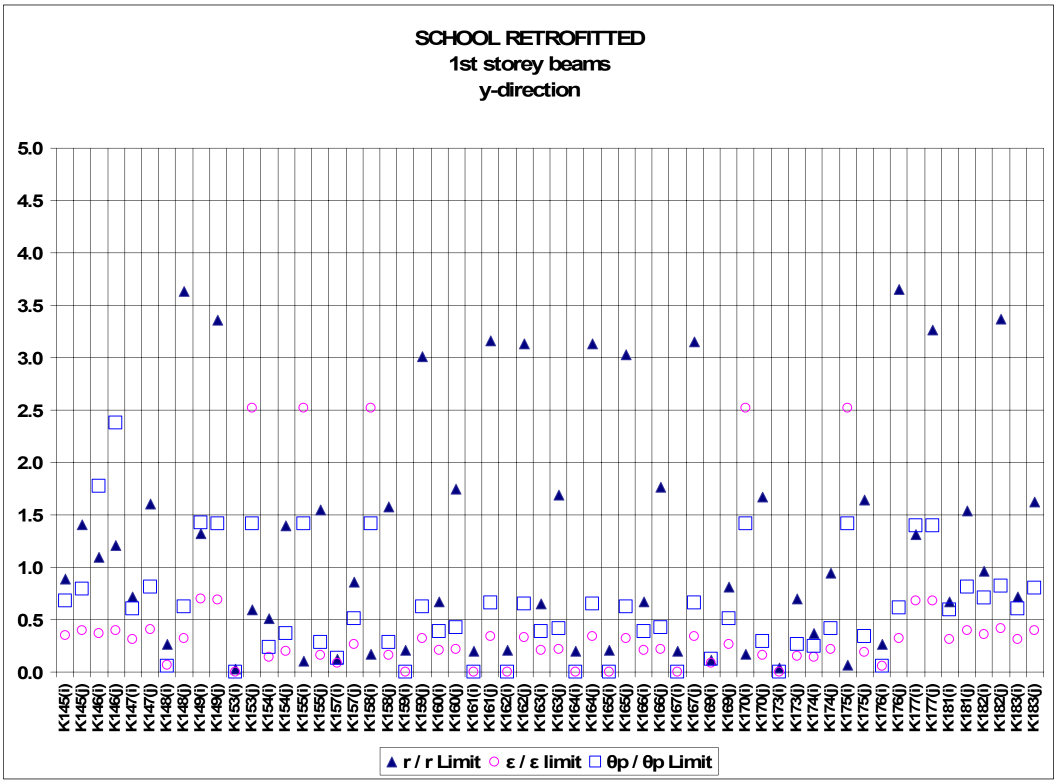


Figure 6.23 First Storey Beams of SCHOOL RETROFITTED in Y Direction

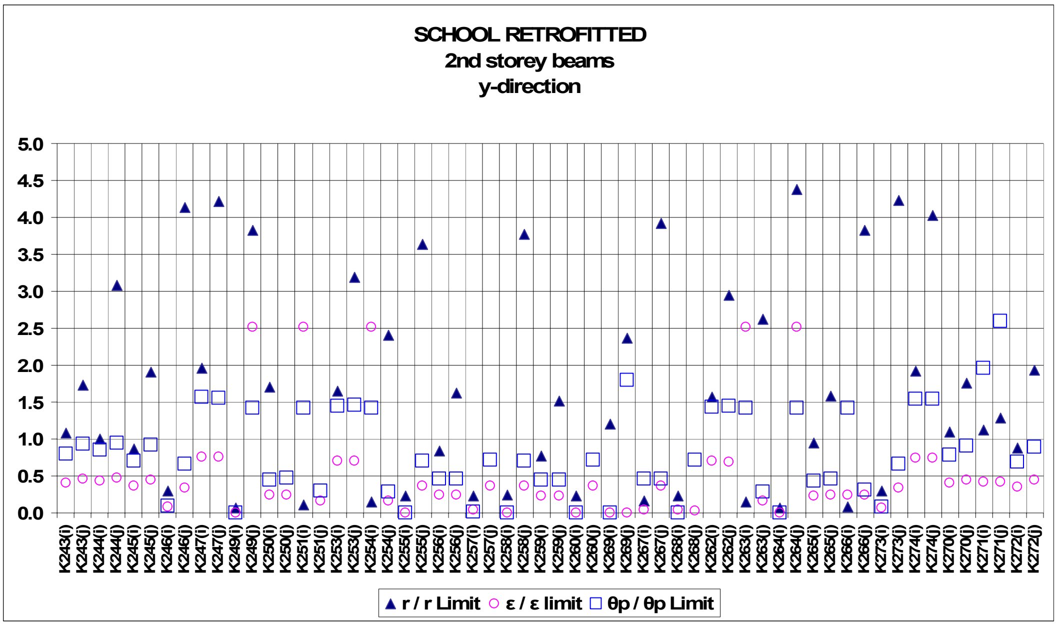


Figure 6.24 Second Storey Beams of SCHOOL RETROFITTED in Y Direction

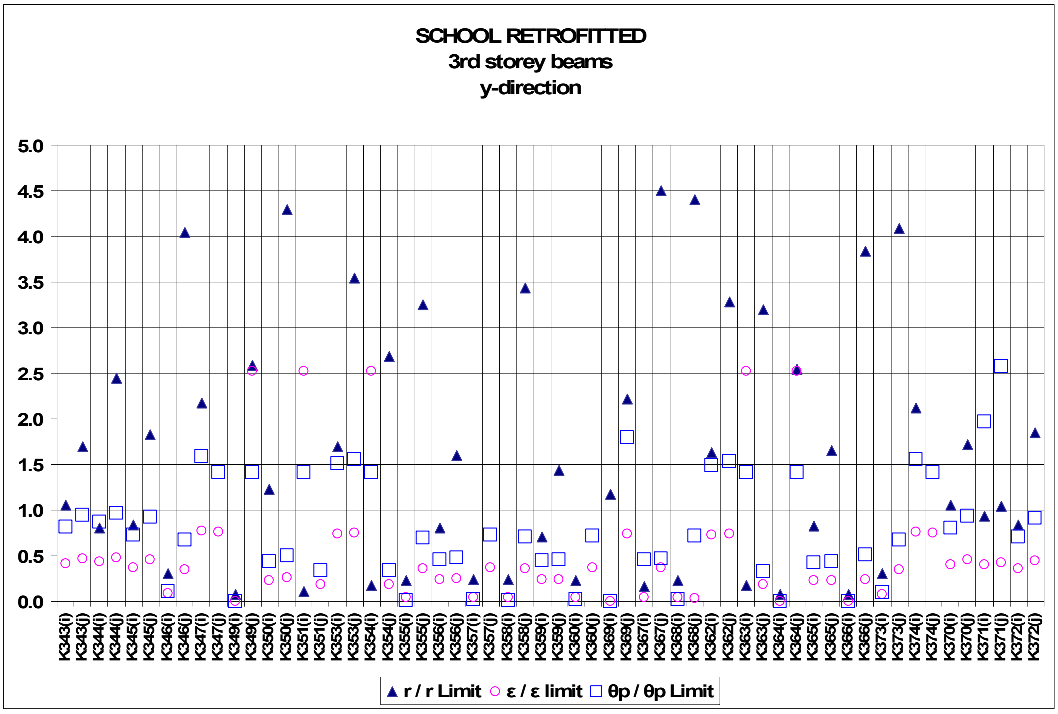


Figure 6.25 Third Storey Beams of SCHOOL RETROFITTED in Y Direction

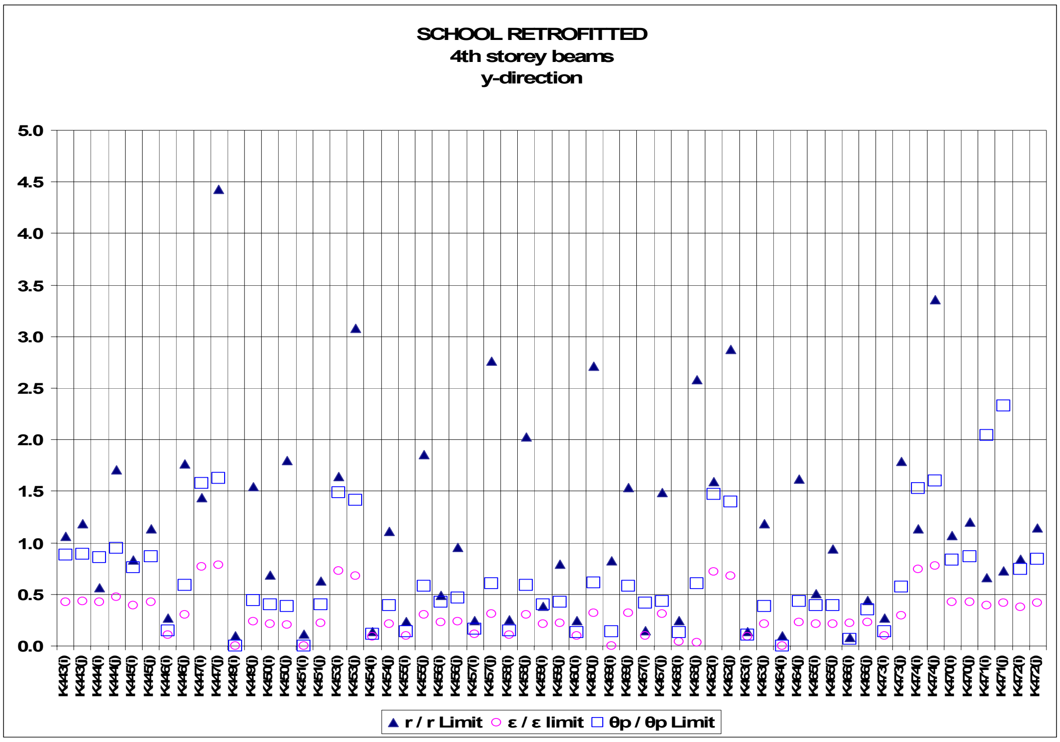


Figure 6.26 Fourth Storey Beams of SCHOOL RETROFITTED in Y Direction

CHAPTER VII

CONCLUSIONS

All results on the normalized acceptability of members obtained in Chapter VI are summarized in Table 7.1.a to Table 7.1.c below.

Table 7.1.a Comparison of Member Acceptabilities in Different Code Procedures for Residential 1998

RESIDENTIAL : 1998 CODE	r / r_{Limit}	$\varepsilon / \varepsilon_{Limit}$	$\theta p / \theta p_{Limit}$
1st storey column bases (confined), X	0.64	0.47	-
1st storey column bases (confined), Y	0.62	0.44	-
1st storey beams (confined), X	0.63	0.60	-
1st storey beams (confined), Y	0.65	0.63	-
2nd storey beams (confined), X	0.58	0.45	-
2nd storey beams (confined), Y	0.61	0.52	-

Table 7.1.b Comparison of Member Acceptabilities in Different Code Procedures for Residential 1975

RESIDENTIAL : 1975 CODE	r / r_{Limit}	$\varepsilon / \varepsilon_{Limit}$	$\theta p / \theta p_{Limit}$
1st storey column bases (confined), X	0.86	0.72	1.22
1st storey column bases (confined), Y	0.84	0.64	1.07
1st storey beams (confined), X	0.93	0.77	0.89
1st storey beams (confined), Y	0.95	0.79	0.94
2nd storey beams (confined), X	0.83	0.50	0.56
2nd storey beams (confined), Y	0.86	0.56	0.66

Table 7.1.c Comparison of Member Acceptabilities in Different Code Procedures for School Retrofitted

RETROFITTED : SCHOOL	r / r_{Limit}	$\varepsilon / \varepsilon_{Limit}$	$\theta p / \theta p_{Limit}$
1st storey column bases (unconfined), X	0.76	0.49	1.59
1st storey column bases (unconfined), Y	0.49	0.38	0.90
1st storey shearwall bases (confined), X	0.59	0.26	0.44
1st storey shearwall bases (confined), Y	0.47	0.19	0.31
1st storey shearwall bases (unconfined), Y	1.58	0.17	0.73
1st storey beams (unconfined), X	2.63	0.45	0.90
1st storey beams (unconfined), Y	1.41	0.51	0.70
2nd storey beams (unconfined), X	4.23	0.54	1.09
2nd storey beams (unconfined), Y	2.51	0.57	0.87

Since, the bottom ends of the first storey columns yield in general, the normalized acceptability values give more reliable results for them. Hence, 1st storey column bases only are included in the tables above.

The post elastic response is dominant at the bottom storey beams. Therefore, acceptability comparisons can be made more accurately for bottom storey beams. The demands for top storey beams so usually overestimated by the linear elastic analysis. Hence, the first and second storey beams are included in the tables in order to obtain reliable comparisons.

The results of nonlinear procedure in the 2006 Turkish Earthquake Code do not match with the results of nonlinear procedure in FEMA. In fact, nonlinear procedure of FEMA is much more conservative. This is due to lower acceptance limits for the plastic rotations since the demands are same in two methods. Hence, the acceptability limits of nonlinear procedure in Turkish Code should be re-assessed.

Linear elastic procedure is more conservative than the nonlinear procedure as expected. However, especially for unconfined members, linear elastic procedure is too much conservative compared to the nonlinear procedure. Hence, the acceptability limits of linear elastic and nonlinear procedures should be modified in order to obtain more compliance.

REFERENCES

[1] SAP2000, Nonlinear Version Structural Analysis Program, Computers and Structures, Inc.

[2] New Zealand Society for Earthquake Engineering, 2003, Assessment and Improvement of the Structural Performance of Buildings in Earthquake, page C28-C32.

[3] Applied Technology Council, ATC-40, 1996, ‘Seismic Evaluation and Retrofit of Concrete Buildings, Volume 1-2, Redwood City, California.

[4] Federal Emergency Management Agency (FEMA), 2000, ‘Prestandard and Commentary for the Seismic Rehabilitation of Buildings, FEMA-356.

[5] European Committee for Standardization. Eurocode 8: Design Provisions for Earthquake Resistance of Structures. Part 3: Strengthening and Repair of Buildings. Bruxelles, 2001.

[6] Mander, J.B., Priestley, M.J.N., Park, R. 1988, Theoretical Stress-Strain Model for Confined Concrete, Journal of Structural Division (ASCE), 114(8), 1804-1826.

[7] New Zealand Society for Earthquake Engineering, 2003, Assessment and Improvement of the Structural Performance of Buildings in Earthquake, page C64.

[8] Priestley, Myths and Fallacies in Earthquake Engineering, Revisited, page 11.

[9] Turkish Earthquake Code: Specifications for the Buildings to be Constructed in Disaster Areas, 2006, Ministry of Public Works and Settlement, Ankara, Turkey

[10] Turkish Earthquake Code: Specifications for the Buildings to be Constructed in Disaster Areas, 1998, Ministry of Public Works and Settlement, Ankara, Turkey

[11] Turkish Earthquake Code: Specifications for the Buildings to be Constructed in Disaster Areas, 1975, Ministry of Public Works and Settlement, Ankara, Turkey

[12] Turkish Standard Institute, 2000, 'TS-500 Requirements for Design and Construction of Reinforced Concrete Structures.

[13] Ersoy U., 1997, 'Reinforced Concrete', Middle East Technical University, Ankara, Turkey.

[14] Günay M.S., 2003, 'Simplified Procedures for Seismic Assessment of Medium Rise Reinforced Concrete Building Structures', Master Thesis, Department of Civil Engineering, METU, Ankara.

[15] Park R., 1998, 'Seismic Assessment and Retrofitting of Reinforced Concrete Buildings', Proceedings of the Fifth International Conference on Tall Buildings, Hong Kong, Volume 1, 56-77.

APPENDIX

STRESS-STRAIN DIAGRAMS USED IN CALCULATING MOMENT-CURVATURE RELATIONS

For obtaining moment – curvature curves, one of the concrete stress – strain diagrams should be selected. In this study. Mander Model (1988) is selected for concrete stress – strain diagram.

Obtaining concrete stress – strain diagrams by Mander Model is composed of the following steps:

The longitudinal compressive concrete stress fc is given by :

$$f_c = \frac{f_{cc}\ x\ r}{r - 1 + x^r} \tag{A.1}$$

f_{cc} = compressive strength of confined concrete and calculated by following formula :

$$f_{cc} = \lambda_c f_{co} \qquad ; \qquad \lambda_c = 2.254\sqrt{1 + 7.94\frac{f_e}{f_{co}}} - 2\frac{f_e}{f_{co}} - 1.254 \tag{A.2}$$

f_{co} = unconfined concrete compressive strength

f_e = effective lateral confining stress; for rectangular sections, the mean value of the following values :

$$f_{ex} = k_e \rho_x f_{yw} \quad ; \quad f_{ey} = k_e \rho_y f_{yw} \tag{A.3}$$

f_{yw} = yield strength of transverse reinforcement

k_e = confinement effectiveness coefficient

ρ_x = ratio of the volume of transverse confining steel to the volume of confined concrete core in x-direction

ρ_y = ratio of the volume of transverse confining steel to the volume of confined concrete core in y-direction

$$k_e = \left(1 - \frac{\sum a_i^2}{6 b_o h_o}\right)\left(1 - \frac{s}{2 b_o}\right)\left(1 - \frac{s}{2 h_o}\right)\left(1 - \frac{A_s}{b_o h_o}\right)^{-1} \tag{A.4}$$

a_i = ith clear distance between adjacent longitudinal bars

b_o , d_o = core dimensions to centerlines of perimeter hoop

s = center to center spacing or pitch of spiral or circular hoop

A_s = longitudinal steel area

The other parameters are:

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \quad ; \quad \varepsilon_{cc} = \varepsilon_{co}[1 + 5(\lambda_c - 1)] \quad ; \quad \varepsilon_{co} \cong 0.002 \tag{A.5}$$

$$r = \frac{E_c}{E_c - E_{sec}} \quad ; \quad E_c \cong 5000\sqrt{f_{co}} \quad [MPa] \quad ; \quad E_{sec} = \frac{f_{cc}}{\varepsilon_{cc}} \tag{A.6}$$

$$\varepsilon_{cu} = 0.004 + \frac{1.4\, \rho_s\, f_{yw}\, \varepsilon_{su}}{f_{cc}} \tag{A.7}$$

$$\rho_s = \rho_x + \rho_y \tag{A.8}$$

The other parameters are summarized in Figure A.1:

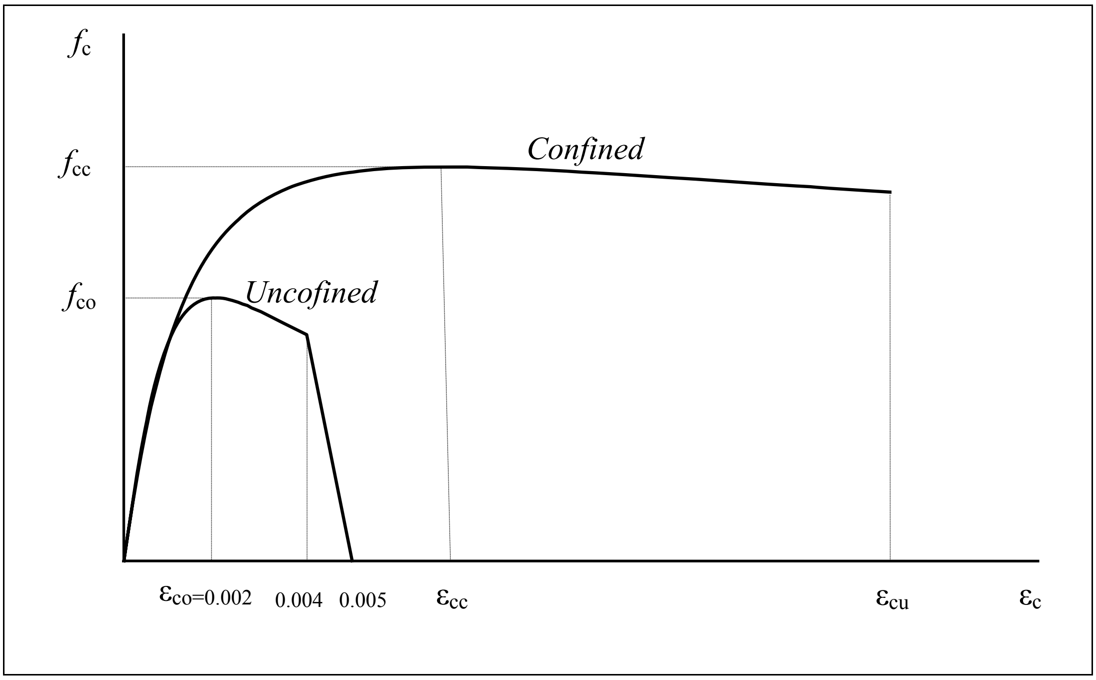


Figure A.1 Stress – Strain Model for Concrete

The stress – strain model for steel used in this study is as follows (Figure A.2) :

$$
\begin{aligned}
f_s &= E_s\,\varepsilon_s && (\varepsilon_s \le \varepsilon_{sy})\\
f_s &= f_{sy} && (\varepsilon_{sy} < \varepsilon_s \le \varepsilon_{sh})\\
f_s &= f_{su} - (f_{su} - f_{sy})\,\frac{(\varepsilon_{su} - \varepsilon_s)^2}{(\varepsilon_{su} - \varepsilon_{sh})^2} && (\varepsilon_{sh} < \varepsilon_s \le \varepsilon_{su})
\end{aligned}
\qquad \text{(A.9)}
$$

$E_s = 2*10^5$ MPa

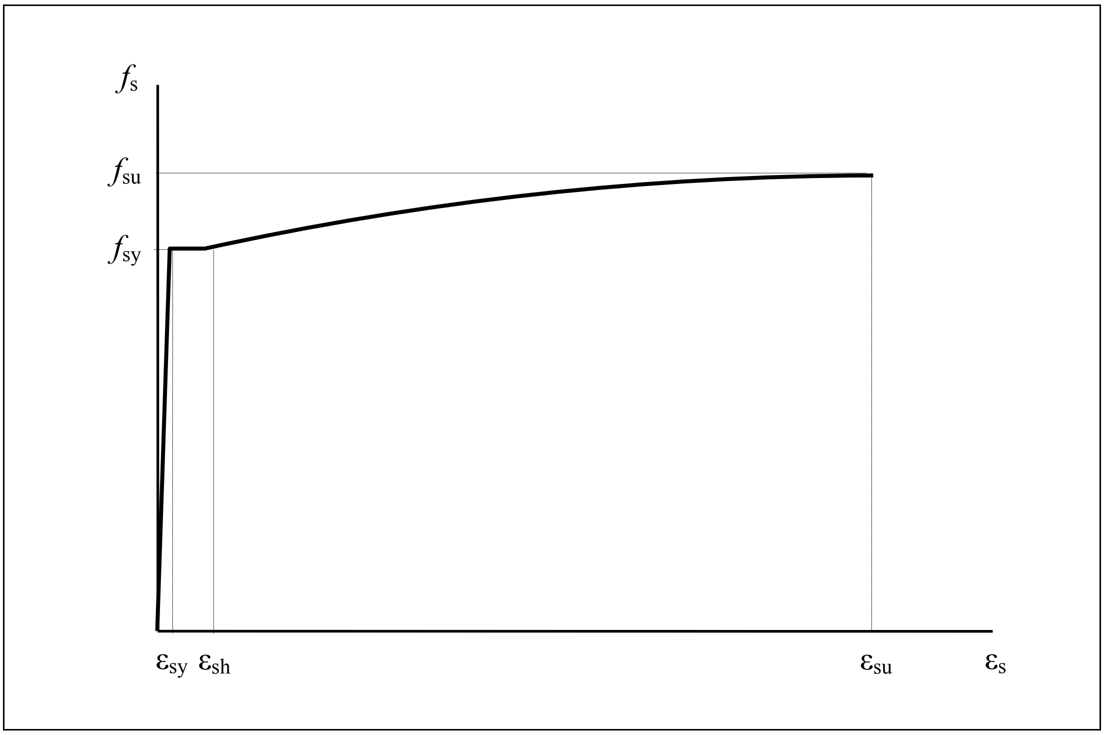


Figure A.2 Stress – Strain Model for Steel