A GENERAL EVALUATION OF LINEAR ELASTIC ASSESSMENT PROCEDURES IN SEISMIC CODES

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ABSTRACT

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Linear elastic procedures are extensively used in seismic codes either for design or assessment, however it does not necessarily mean that they are always accurate. In the first part of this study, performance limits of the force-based linear assessment procedure in the Turkish Earthquake Code are verified and calibrated with the displacement-based performance limits which are obtained from experimental data. Four case study buildings, all of which satisfy code design requirements are assessed with force based and displacement based linear procedures under their design spectrum. Force and displacement based results are compared to each other. Accordingly, necessary adjustments are suggested on the force-based performance limits of the Turkish Seismic Code.

In the second part, limitations of linear procedures are investigated through a comparative evaluation of linear and nonlinear procedures. Nine case study buildings are analyzed using linear response spectrum analysis and nonlinear response history analysis procedures under nine different ground motions. Plastic rotation demands at member ends are used as common deformation parameters, where the results from nonlinear analyses are accepted as benchmark. Demand to capacity ratios (DCR) at member ends calculated from linear procedure is used as the decision parameter. For different buildings with different systems and irregularities, applicability of linear procedures are investigated and limitations are proposed by considering the level of irregularities which are expressed in terms of DCR distributions.

Keywords: Seismic assessment, linear procedures, nonlinear procedures, performance limits, limitations of linear procedures.

ÖZ

DEPREM YÖNETMELİKLERİNDEKİ DOĞRUSAL ELASTİK DEĞERLENDİRME YÖNTEMLERİNİN İRDELENMESİ

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Deprem yönetmeliklerinde doğrusal elastik yöntemler hem tasarım hem de deprem güvenliği değerlendirmesinde sıklıkla kullanılmaktadır. Ancak bu durum onların her zaman tutarlı olduğu anlamına gelmez. Bu çalışmanın ilk kısmında Türkiye Deprem Yönetmeliği'nde kullanılan kuvvet esaslı performans sınırları test edilmiş ve deneysel verilere dayanarak elde edilmiş olan deplasman esaslı performans limitlerine bağlı olarak düzeltilmiştir. Dört adet yönetmelik uyumlu olarak tasarlanmış örnek binanın değerlendirmesi kuvvet ve deplasman esaslı doğrusal yöntemler kullanarak Yönetmelik tasarım spektrumu altında yapılmıştır. Kuvvet ve deplasman esaslı yöntemler ile elde edilen sonuçlar birbirleriyle karşılaştırılmıştır. Bunlara bağlı olarak, Türkiye Deprem Yönetmeliğindeki performans limitlerinde gerekli görülen değişiklikler önerilmiştir.

İkinci kısımda doğrusal ve doğrusal olmayan yöntemlerin karşılıklı değerlendirilmesi ile doğrusal yöntemlerin sınırları incelenmiştir. Dokuz adet örnek bina, dokuz farklı yer hareketi altında doğrusal mod birleştirme ve zaman tanım alanında doğrusal olmayan hesap yöntemleriyle analiz edilmiştir. Doğrusal olmayan analiz sonuçları ölçüt olarak kabul edilerek, eleman uçlarındaki plastik dönme talepleri genel deformasyon parametresi olarak kullanılmıştır. Doğrusal yöntem ile hesaplanan eleman uçlarındaki talep-kapasite oranları (DCR) karar parametresi olarak kullanılmıştır. Doğrusal yöntemlerin uygulanabilirliği değişik sistemlere ve düzensizliklere sahip farklı binalar için incelenmiş ve DCR dağılımına bağlı olarak belirlenen düzensizlik seviyelerine göre sınırlamalar önerilmiştir.

Anahtar kelimeler: Deprem güvenliği değerlendirmesi, doğrusal elastik yöntem, doğrusal olmayan yöntem, performans sınırları, doğrusal elastik yöntemlerin uygulama sınırları.

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CHAPTER 1

INTRODUCTION

1.1 Statement of the Problem

There are four main types of analysis in determining the seismic response of structures, for both design and assessment. They are linear static, linear dynamic, nonlinear static and nonlinear dynamic analysis procedures. Linear and nonlinear static procedures are conducted under a response or design spectrum whereas linear and nonlinear dynamic (response history) procedures are conducted under a ground motion excitation. Among these four analysis methods, the most accurate response is obtained by nonlinear dynamic analysis (nonlinear response history analysis). However, because of the difficulties in employing nonlinear dynamic analysis, it is almost never used in practical civil engineering applications including seismic design according to codes. Instead, linear static analysis is preferred by engineers for most of the cases. The reason behind this is the simplicity. Procedures followed in linear static analysis are easy to apply and follow, even for civil engineers who do not have deep knowledge on seismic behavior of structures. Therefore, seismic codes around the world are mainly based on linear static analysis for both design of new and assessment of existing structures. However, earthquake engineering cannot rely on completely linear static analysis for all kinds of structures because it may not correctly estimate the response of structures that undergo post-yielding (inelastic) deformations under strong earthquake ground motions. Thus, some limitations are introduced in seismic design codes for using linear elastic methods. For some structures, it is mandatory to use one of the nonlinear methods specified above since linear methods are not reliable for them.

Earthquake engineering is a fast developing field of applied science. Assumptions and theories that are accepted twenty years ago may not be valid today. This phenomenon is also true for seismic design codes. The methods used in earthquake engineering are improved each day to better estimate the complete effects of earthquakes on the structures using simpler approaches. For this reason, seismic design codes are updated from time to time in order to account for the recent developments in the field. One of the most critical updates is acceptability of employing nonlinear static analysis for different types of structures. By changing trends in the last decade from force-based approach to displacement-based approach, limitations are imposed on employing linear static analysis in seismic design codes.

In the first part of this study, performance limits of Turkish Earthquake Code (TEC2007) are investigated. In the second part, limitations on using the linear static analysis procedures for displacement-based assessment procedures in TEC2007 are investigated and possible improvements are proposed.

1.2 Review of Past Studies

Literature review is presented in two parts. In the first part, studies on the limitations in employing linear elastic procedures as well as the improvements on linear elastic procedures for obtaining better estimation of inelastic response of R/C structures are reviewed. In the second part, studies investigating the weaknesses of force reduction factors (R) employed in seismic codes are presented.

1.2.1 Limitations and Improvements of Linear Elastic Analysis Methods

Linear static analysis methods can be considered as approximations for estimating the true nonlinear dynamic response of structures under earthquake effects. Therefore, improvements are necessary for better estimations of the reality. These improvements also increase the applicability of linear elastic methods and decrease the amount of limitations for using them.

Moehle (1984) suggested that linear static analysis without any modification shall be used for preliminary design or for ‘standard’ structures only. In order to enlarge the structure types that linear static analysis can be used, several improvements on the linear elastic procedures are suggested. In that study, four 9 story R/C wall-frames are experimented with different wall heights. Their analytical models are prepared using three different approaches. The first one used gross cross-sectional properties of members, the second one used cracked cross-sectional properties of members and the third one used the substitute structure approach. The results indicate that the analysis based on gross cross-sectional properties of elements failed to estimate the response of the overall structure, whereas, other two models correlated well with the real response of the structures. If the building is regular, mid-height structure, linear static analysis with little improvement can be applied to estimate the inelastic response.

Moehle and Alarcon (1986) investigated the effect of stiffness and strength irregularity over the structure height. In this study, two 9 story R/C wall-frame structures are experimented, one without any irregularities and another with stiffness and strength irregularities over the height. Elastic and inelastic analyses are performed along with the experiments. The results showed that for the regular building, both elastic and inelastic analyses estimated the response well. However, elastic analyses underestimated the displacement response and could not assess the nonuniformity of force distribution.

Kosmopoulos and Fardis (2007) worked on four real structures with significant plan and elevation irregularities in their study. Nonlinear response history analyses are conducted as benchmark and linear analyses (modal response spectrum analysis and inverted triangular equivalent lateral force approach) are carried out to check whether they can estimate the nonlinear behavior of the structure. The structures analyzed had fundamental periods in the velocity-sensitive region. The ground motions chosen were consistent with the Eurocode8 spectrum. Although, the structures analyzed were highly irregular and therefore, linear elastic analysis procedures are not allowed by EC8, the results show that 5%-damped elastic analysis can estimate the response of these structures fairly accurate. It is advised that the criterion to use linear elastic methods in EC8 can be relaxed.

Another work related to the code provisions on seismic assessment of existing structures was published by Toprak, Gülay and Ruge (2008). In this study, a real case of damaged building from 1998 Adana-Ceyhan earthquake is observed with respect to the linear procedures defined in both Eurocode8 and TEC2007, which was recently published at the time. The actual 6 story R/C frame structure, which is highly regular in both plan and elevation was built according to the 1975 Turkish Seismic Code and it was reported as moderately damaged after the 1998 Adana-Ceyhan earthquake. This also supports the idea that seismic codes become robust and should be updated in certain time intervals. Although the performance limits are identified differently in EC8 and TEC2007, both of them produced similar results, which were slightly overestimating the observed performance level of the structure.

Chandler and Mendis (1998) proposed displacement-based approach instead of force-based in their study. Both approaches are tried on models that were prepared according to European earthquake code provisions. The results showed that displacement-based approach can be useful for many of the cases that force-based approach fails.

In the process of transition from force-based to displacement-based approach, equivalent systems phenomena became important to estimate the post-yielding response of structures. Günay (2008) worked on this subject and proposed a new procedure called equivalent linearization. This method uses response spectrum analysis, but the stiffnesses of structural members, which are expected to yield, are reduced accordingly. Lin and Lin (2009) proposed non-iterative equivalent linearization. In this study, the equivalent linear systems, which are based on the secant period, are presented in order to estimate the maximum acceleration and displacement responses of the existing structures. Instead of ductility ratio, equivalent period and damping values are defined by the strength ratio which is known for the existing structures, hence the iterations are omitted. As the result of this study, it is proposed that this procedure may be applicable to wide range of structures with different fundamental periods.

Sucuoğlu and Günay (2009) compared equivalent linearization method with other ‘widely used’ analysis methods, namely response spectrum, conventional pushover and nonlinear response history analysis, in their work. Two buildings are used in this study. The first one is a 12 story R/C frame with significant higher mode effects. The other one is a 6 story unsymmetrical structure with significant torsional effects. As a result of this study, it is revealed that equivalent linearization procedure can predict seismic response of the structures accurately even if torsional or higher mode effects are significant. Considering the simplicity of the procedure, it is proposed to be very effective to predict the inelastic response of structures.

Further study by Sucuoğlu and Günay (2010) proposed an improved linear-elastic response spectrum analysis procedure. According to this method, structural members, which are expected to yield, are determined in advance and their stiffnesses are reduced in one step, which eliminates the iterative procedure and simplifies the overall analysis. In this study, two different structures, one with higher mode effects and another one with significant torsional coupling were designed with the capacity design principles. The response obtained by improved linear-elastic procedure is compared with nonlinear static and nonlinear dynamic analysis results. The comparisons showed that the proposed method is as accurate as nonlinear static analysis for both of the structures. Considering the simplicity of this method over nonlinear static analysis, linear analysis can be chosen instead of nonlinear analysis. However, the seismic design codes are still preventing extended use of linear elastic methods. Considering all of the improvements in linear elastic methods with displacement-based approach, relaxation of seismic design codes in using linear elastic procedures can be expected accordingly.

1.2.2 Drawbacks of the Force Reduction Factors Defined in Seismic Design Codes

Force reduction factors (R) used in seismic design codes were introduced to apply a linear elastic approach, but obtain nonlinear response of a structure. R factors defined in seismic codes are highly dependent on the ductility level of the structural systems and materials. Using a single force reduction factor to estimate the nonlinear behavior of the structure by using linear elastic methods is the simplest possible solution to a very important problem and this approach has been accepted to be accurate enough for force-based analysis for many years. However, earthquake engineering is shifting through the displacement-based approaches more each day. As a result of this change, force reduction factors used in seismic design codes are further questioned.

Mondal, Ghosh and Reddy (2013) published their study on the acceptability of force reduction factors used in seismic design codes. The study is mainly focused on the Indian seismic design code IS1893. However, force reduction factors given also in EC8 and ASCE7 are compared to the calculated force reduction factors in element and structure level. Results of the study indicate that for low performance limits, R factors given in codes are smaller than reality, which may cause dangerous situations. Another important outcome of this work is that the actual value of R in designs will be lower than calculated values because of some errors which may occur during the construction.

Ashrafi (2013) discussed the insufficiency of using a single force reduction factor for the structures with significant higher mode effects. In this work, 150-meter tall R/C structure with a concrete shear wall core is modeled and analyzed using both linear elastic and nonlinear methods. In linear elastic methods, force reduction factors are used in order to see whether the nonlinear behavior can be estimated correctly or not. The results show that using a single R factor to reduce the forces eliminates the effects of higher modes on the structural response and this elimination does not necessarily resulting in safer results. Linear elastic methods may underestimate the responses, which may cause a dangerous situation. By using linear elastic methods with a force reduction factor, one cannot obtain the correct distribution of forces on elements apart from the overall structural response. As a result, code based linear elastic methods cannot be used for the seismic evaluation of buildings for which higher modes are effective in the seismic response.

1.3 Assessment Procedures in Seismic Codes

Assessment procedures in seismic codes are explained in two parts for three different seismic codes, namely Eurocode 8-3, ASCE 41 and TEC. In the first part, force-based assessment procedures; and in the second part, displacement-based assessment procedures that are defined in the aforementioned seismic design/assessment codes are investigated.

1.3.1 Force-based Assessment Procedures in Seismic Assessment Codes

1.3.1.1 Force-based Assessment in ASCE41

At first, knowledge level of an existing structure is determined. Knowledge level is expressed by factors that accounts for the amount of information an engineer has about the structure to be assessed. After this, analysis procedure to be used during assessment is determined. Two different analysis procedures for force-based assessment are introduced in ASCE41. Both of them are linear elastic procedures, namely linear static analysis and linear dynamic analyses.

a) Linear Static Analysis (equivalent lateral load method)

It is the simplest analysis procedure defined in ASCE41. In this method, the total force expected to act on a structure during an earthquake is distributed along the height of the building in a shape of inverted triangle. If the structure is expected to stay in the elastic range, then linear static analysis is an acceptable method. However, if the structure exceeds yield point, this method is expected to overestimate the response. There are some additional conditions that are preventing the linear static analysis to be used. These conditions that should be satisfied in order to use the linear static method according to ASCE41 are listed below:

- The building cannot have torsional stiffness irregularity at any floor. In order to check this, drift values can be calculated. If drift along any side of the building is greater than the 150% of the average drift, then the building has torsional stiffness irregularity.
- The building shall not have vertical stiffness irregularity. If average drift at any story is larger than 150% of the adjacent story, then the building has vertical stiffness irregularity.
- The building shall not have a non orthogonal lateral force resisting system.

- The ratio of the horizontal dimension at any story to the corresponding dimension at an adjacent story shall not exceed 1.4
- Fundamental period of the structure shall not be longer than 3.5 times T_s, where T_s is the transition period between constant acceleration and constant velocity regions.

b) Linear Dynamic Analyses

There are two different linear dynamic procedures in ASCE41. The first method is the response spectrum analysis. If response spectrum analysis method is used, total number of modes that should be taken into account should be decided correctly. The second method is linear response history analysis. For either one of linear dynamic analyses, the ground motions chosen should be consistent with the elastic response spectrum defined in ASCE41.

After completing the analysis, type of actions on elements are defined. There are two types of actions specified in ASCE41, namely force-controlled and deformation-controlled actions. They basically distinguish the ductile (deformation-controlled) or non-ductile (force-controlled) elements or mechanisms. It is important to decide whether an element or mechanism is behaving ductile or brittle under a ground excitation. For, deformation-controlled design actions, force on any member, Q_{UD}, is defined in Equation 1.1.

$$Q_{UD} = Q_G \pm Q_E \tag{1.1}$$

In Equation 1.1, Q_G is the action due to design gravity loads and Q_E is the action due to earthquake loads. If the element or mechanism is a non-ductile one, then force-controlled actions take place. In force-controlled design actions, Q_{UF}, is calculated according to Equation 1.2.

$$Q_{UF} = Q_G \pm \frac{Q_E}{C_1 * C_2 * J} \tag{1.2}$$

J is the force delivery reduction factor ($J \geq 1.0$) in Equation 1.2. J values can be taken as 2, 1.5 and 1 for high, moderate and low levels of seismicity, respectively. J should be chosen as 1 for immediate occupancy structure performance level. C_1 and C_2 coefficients are used to consider the effects of pinching, stiffness degradation and strength deterioration on maximum response of the structure.

Deformation-controlled actions in primary and secondary elements should satisfy the condition given in Equation 1.3.

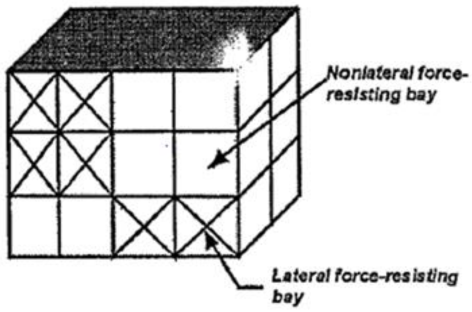
$$m * \kappa * Q_{CE} \geq Q_{UD} \tag{1.3}$$

In this equation, Q_{CE} is the expected strength of the component at the deformation level, κ is the knowledge factor and m is the component demand modification factor, which is highly dependent on the ductility level of the component and mechanism. “m” factors calculated for ductile elements at this point are checked with the tables provided in different sections of ASCE41 in order to decide the structure’s performance level according to the force-based assessment approach. For non-ductile components (force-based actions), the total force on the component should not exceed the expected strength, Q_{CE}. Therefore, m factors are not calculated for non-ductile elements.

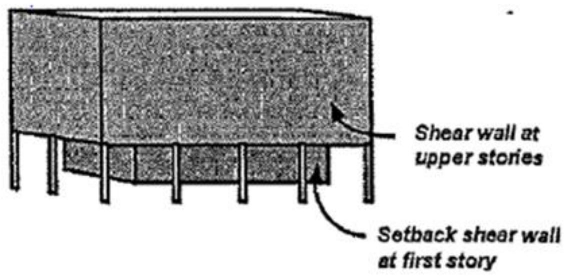
Although the linear elastic procedures for force-based assessment in ASCE41 are straight forward and easy to apply compared to the nonlinear approaches, they cannot be always used. There are certain limitations that do not allow using linear elastic analysis methods. According to ASCE41, main reason for linear elastic methods to estimate “unrealistic” response is the structural irregularities. If there is no irregularity in the building, linear elastic methods can be used if certain conditions are satisfied. Irregularity level of a structure becomes important at this point. Up to a certain irregularity level, errors caused by the linear analysis methods stay in the acceptable range. The irregularities that may prevent linear elastic analysis procedures are given below from *i* to *iv*.

i. In plane irregularity: The structure has in plane irregularity when the lateral force resisting elements are present in one story, but does not exist in the adjacent story.

ii. Out of plane irregularity: The structure has out of plane irregularity when out of plane offset of an element is not present. The Figure 1.1 explains these irregularities.



a) In-plane discontinuity in lateral system



b) Typical building with out-of-plane offset irregularity

Figure 1.1: Typical in-plane and out-of-plane irregularities in the structures

iii. Weak story irregularity: Weak story irregularity is determined by the average shear demand to capacity ratios (DCR) of stories. The structure has weak story irregularity if the ratio of average shear DCR of any story to that of an adjacent story in the same direction exceeds 125%. The average DCR, $\overline{DCR}$, of a story is calculated using the formula given in Equation 1.4.

$$\overline{DCR} = \frac{\sum_1^n DCR_i * V_i}{\sum_1^n V_i} \tag{1.4}$$

In Equation 1.4, DCR_i is the critical action DCR for element i, which is defined as the system, such as moment, shear or axial force, that creates the maximum demand to capacity ratio on that element, of the story, V_i is the total calculated lateral shear force in an element i due to earthquake response, assuming that the structure remains elastic, and n is total number of elements in the story.

iv. Torsional strength irregularity: The structure has torsional strength irregularity when the critical element DCR of one side at the story is larger than 1.5 times the critical element DCR of the other side with respect to stiffness center of that story.

The largest DCR for any element at a particular particular story is termed the critical element DCR at that story.

Linear elastic procedures in ASCE 41 can be used, provided that all member DCR’s are less than two for all structures, or the structure does not have any irregularities stated above in (i) to (iv). There are additional limitations for concrete systems. These limitations are discussd in detail in Chapter 3.

1.3.1.2 Force-based Assessment in Eurocode8

There is no force-based assessment procedure defined in EC8 for ductile elements or mechanisms. If the governing mechanism is brittle (e.g. shear in beam, column and wall members), force-based assessment can be done. As a result of any analysis, shear forces on the elements are acquired and these forces are compared with the shear strengths of the associated elements. Apart from this, assessment of existing structures is completely based on displacement-based procedures in EC8.

1.3.1.3 Force-based Assessment in TEC2007

Similar to ASCE41 provisions, as the first step of structural assessment, TEC2007 also requires determining the structure’s knowledge level. Then the analysis procedure is chosen. TEC2007 allows two analysis methods for force based approach, which are equivalent lateral force analysis and multi-modal response spectrum analysis, which are very similar to ASCE41 way.

a) Equivalent Lateral Force Analysis:

Although it is the simplest analysis procedure defined in TEC2007, similar to ASCE41, there are certain conditions that equivalent lateral force analysis cannot be used. These conditions are mainly based on the irregularities of the structure considered. Equivalent Lateral force analysis is not allowed if,

- A structure located in the first or second seismic zone has a height more than 25 meters, or the coefficient of torsional irregularity, η_{bi}, is larger than 1.4 for any floor. η_{bi} is calculated by the Equation 1.5.

$$\eta_{bi} = \frac{\Delta_{i,max}}{\Delta_{i,ave}} \tag{1.5}$$

In Equation 1.5, $\Delta_{i,max}$ is the maximum relative floor displacement of i^{th} floor and $\Delta_{i,ave}$ is the average floor displacement of the i^{th} floor, both of which are calculated according to equivalent lateral force analysis.

- A structure located in the first or second seismic zone has a height more than 40 meters, or the coefficient of torsional irregularity, η_{bi}, is larger than 2 for any floor, or soft story mechanism is present. The existence of soft story in a structure is checked by calculating the coefficient of vertical irregularity, η_{ki}. If η_{ki} is larger than 2, weak story mechanism is present in a structure. The formula to calculate coefficient of vertical irregularity is given by Equation 1.6.

$$\eta_{ki} = \frac{\left(\frac{\Delta_i}{h_i}\right)_{ave}}{\left(\frac{\Delta_{i+1}}{h_{i+1}}\right)_{ave}} \ or \ \eta_{ki} = \frac{\left(\frac{\Delta_i}{h_i}\right)_{ave}}{\left(\frac{\Delta_{i-1}}{h_{i-1}}\right)_{ave}} \tag{1.6}$$

In this equation, Δ_i, Δ_{i-1} and Δ_{i+1} are the lateral floor displacement of i^{th}, $i-1^{th}$ and $i+1^{th}$ floors, respectively; and h_i, h_{i-1} andh_{i+1} are the floor heights of i^{th}, $i-1^{th}$ and $i+1^{th}$ floors, respectively.

- a structure is located in the third or fourth seismic zone and higher than 40 meters.

b) Multi-Modal Response Spectrum Analysis:

It is also a linear elastic analysis method. However, since it includes the higher mode effects of structures, it can estimate the nonlinear response of structures more accurately. Another advantage of this analysis method is that it can be used for all structures. There are no specified limitations to use multi-modal response spectrum analysis in TEC2007, which allows engineers to use linear elastic analysis for all structures during seismic assessment.

In the force-based approach of TEC2007, after obtaining the response of a structure using linear elastic methods, performance level of the structure is determined using given tables for ductile elements, which are prepared considering the estimated ductility levels of elements. Demand to capacity ratios (DCR) are used in force-based assessment in TEC2007. DCRs are expressed as r factors in provided tables. r factor for each member is calculated according to Equation 1.7 and Equation 1.8.

$$r = \frac{M_E}{M_A} = \frac{N_E}{N_A} \tag{1.7}$$

$$M_A = M_K - M_D \; and \; N_A = N_K - N_D \tag{1.8}$$

In these equations, denoter '$_D$' represents the gravity loads and '$_E$' represents the earthquake loads. M_K and N_K are the moment and axial force capacity of a member depending on the material strength, respectively. M_A and N_A are the residual moment and axial load capacities of the members, respectively. For brittle elements or mechanisms, demand from the earthquake analysis should not exceed the capacity of the cross section (DCR<1).

1.3.2 Displacement-based Assessment Procedures in Seismic Assessment Codes

1.3.2.1 Displacement-based Assessment in ASCE41

If the displacement-based assessment procedure is selected, ASCE41 suggests two analysis methods. The first method is nonlinear static analysis. Nonlinear static analysis is a pushover analysis where the response of a structure in a post-yielding region can be observed. The second method suggested for the displacement-based assessment is nonlinear dynamic analysis. It is nonlinear response history analysis, which is the most accurate but most complicated analysis tool to observe the seismic response of any structure.

At the end of analysis, it is obligatory to check the acceptance criteria for both displacement-controlled and force-controlled actions. "For displacement-controlled actions, primary and secondary components should have expected deformation capacities not less than maximum deformation demands calculated at the target displacement. For force controlled actions, primary and secondary components shall have lower-bound strengths not less than the maximum design forces."(ASCE41 Chapter 3.4.3.2)

By using nonlinear methods, both forces and deformations can directly be calculated more accurately. Therefore, there are no additional coefficients to modify the calculated responses. Since displacement-based assessment is used after nonlinear analysis, response parameter to determine structure's performance level is no longer section forces, but section deformations. As the main parameter, plastic rotations at the member ends are used to determine the structure's performance level in ASCE41. Plastic rotations at member ends are directly calculated during nonlinear analyses. These rotation values are then compared to the "numerical acceptance criteria" tables, which are prepared for different structural systems, in different chapters of ASCE41. The performance level of a structure is determined according to this comparison.

1.3.2.2 Displacement-based Assessment in Eurocode 8

After determining the knowledge level of a structure, there are five possible analysis procedures that can be applied for the displacement-based assessment in EC8. The first one is the equivalent lateral force analysis. The second one is multi-modal response spectrum analysis. First two analysis procedures are linear elastic and similar to the ones used in force-based assessment procedures of ASCE41 and TEC2007. The third method is nonlinear static analysis, which is applied as the conventional pushover analysis, but with addition of higher mode effects if necessary. The fourth method in EC8 is nonlinear response history analysis and the last analysis method used is *q-factor approach*. In q-factor approach, regardless of structural type, a q factor of 1.5 and 2 is chosen for concrete and steel structures, respectively. Then, the seismic action is reduced by the q factor and the results are obtained accordingly.

Similar to the limitations on employing equivalent lateral force analysis in ASCE41 and TEC2007, EC8 also has some limitations that should be satisfied in order to apply this method. The main limitation is that lateral force analysis cannot be applied if the modes other than the fundamental mode are effective in the response of the structure, for each principal direction. In order to decide if the higher modes are effective, period and irregularities of a structure is investigated as explained below. Higher modes are effective for structures which

- has the fundamental period, T_1, that does not satisfy the limits given below:

$$T_1 \leq \begin{cases} 4 * T_C \\ 2\ sec. \end{cases}$$

  where T_C is the the upper limit of the period of the constant spectral acceleration branch,

- does not fullfill the requirements specified to account for the irregularities in elevation in EN1998-1: 2004, 4.2.3.3., which are listed below:

  (1) All lateral load resisting systems such as cores, shear walls, frames should run from the base to the top of the building without any interruption.

  (2) Lateral stiffness and the total mass values of stories should remain constant or reduce gradually from the base to the top of the building.

  (3) The ratio between actual story resistance to the resistance required by the analysis should not change disproportionately between adjacent stories in the framed structures.

  Additional limitations are specified if there are setbacks in the building. Figure 1.2 below explains these limitations.

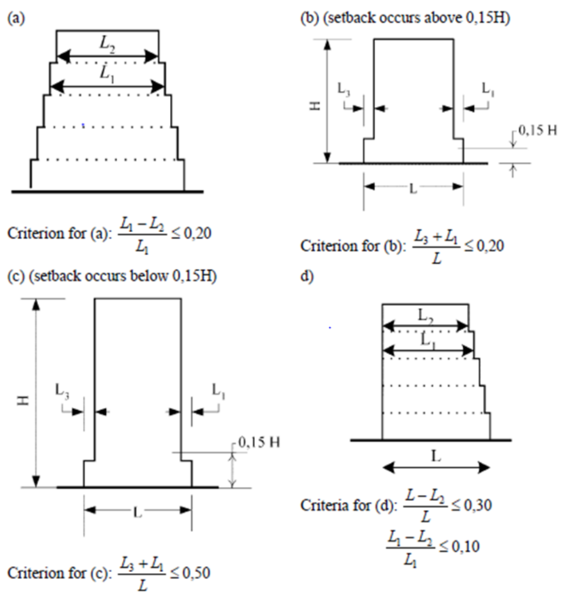


Figure 1.2: Criteria to satisfy irregularity in elevation specifications for different setbacks

As mentioned in Chapter 1.3.1.2 of this thesis, all of the analysis methods mentioned above can be used in displacement-based assessment procedure. Total chord rotation demands at the member ends are obtained for each member from the structural analysis. Then, by comparing these demands to the performance limits specified in EC8 in terms of total chord rotations, performance level of a structure is determined. Displacement-based assessment procedure of EC8 is similar to ASCE41 since they both use rotation demands on cross-sections; however, EC8 uses total rotation demands whereas ASCE41 uses plastic chord rotation demands. Another difference between EC8-3 and ASCE41 is that linear elastic analysis methods are also used in displacement-based assessment procedure in EC8-3.

Although EC8 allows using linear elastic procedures in displacement-based assessment, there are also limitations specified to use them. The reason behind this is that if the expected amount of nonlinearity is too high, then linear methods cannot estimate the accurate nonlinear response, so there should be a limit of linearity. There are two limitations that should be checked. The first one (1) is used for “ductile” whereas the second one (2) is used for “brittle” elements and mechanisms.

(1) DCR’s of all “ductile” primary elements are calculated. For DCR’s greater than 1, the ratio DCR_{max}/DCR_{min} is calculated where DCR_{max} is the maximum DCR and DCR_{min} is the minimum DCR. This DCR_{max}/DCR_{min} ratio does not exceed a maximum acceptable value in the range of 2 to 3.

(2) If the element or mechanism is brittle, demand on that element should be smaller than the capacity. In other words, DCR<1 should be satisfied for all “brittle” elements or mechanisms.

1.3.2.3 Displacement-based Assessment in TEC2007

After deciding on the structural performance level, the first step of displacement-based assessment approach in TEC2007 is to choose the analysis method to be used for assessment. There are two analysis methods for displacement-based assessment. The first method is nonlinear static analysis. The conventional (first mode) pushover analysis is carried out on the nonlinear model to obtain the deformation response. The second method defined in TEC2007 is nonlinear response history analysis, which is the most complex and accurate analysis tool to observe seismic response of any structure. Although nonlinear response history analysis is mentioned in TEC2007, pushover analysis is suggested used for assessment.

The parameters that are used to determine the performance level of a reinforced concrete structure are tension strains of steel and compression strains of cross-sections. Therefore, the response parameters obtained at the end of nonlinear analysis mentioned above are plastic rotations at the member ends and internal forces of the members. The forces are used to check the performance level and safety of brittle elements and/or mechanisms whereas plastic rotations are used for ductile elements and/or mechanisms. After obtaining the plastic rotation demands θ_p from nonlinear analysis, plastic curvature demand on cross-sections are calculated using Equation 1.9.

$$\phi_p = \frac{\theta_p}{L_p} \tag{1.9}$$

In Equation 1.9, L_p is the plastic hinge length of an element and ϕ_p is the plastic curvature demand on a cross-section. Then, total curvature demand, ϕ_t on cross-sections are calculated using Equation 1.10.

$$\phi_t = \phi_y + \phi_p \tag{1.10}$$

In Equation 1.10, ϕ_y is the yield curvature of cross-sections, which are known at the beginning of analysis. Finally, using the total curvature demand on cross-sections, the maximum strains of tension steel and compression concrete are calculated and performance level of a structure is determined.

All of the aforementioned performance limit tables used in seismic assessment codes are provided in Appendix A.

1.4 Objective and Scope

Verification and calibration of performance limits specified in TEC2007 are presented as well as the limitations of linear elastic procedures in displacement-based seismic assessment are discussed in this study. Nine different case study R/C structures are employed. These structures are a twelve story plane frame with full capacity design, a twelve story plane frame with relaxed capacity design, a twelve story plane wall-frame system, a twenty story plane wall-frame system, an eight story unsymmetrical-plan space frame, a six story unsymmetrical plane space frame with full capacity design, a four story retrofitted structure, a five story plane frame and a five story plane frame with reduced column capacity. In the verification and calibration of current seismic assessment code, three “capacity designed” structures and a four story “retrofitted” structure according to Turkish Seismic Code are used, Conventional pushover analysis and the response spectrum analysis are conducted under the linear elastic design spectrum defined in TEC2007, without reduction. Performance (r factor) limits, which are calculated according to TEC2007, and plastic rotation limits which are calculated using ASCE41, are used for calibration purposes. In the second part of this study, all of the nine structures are analyzed with response spectrum analysis, conventional pushover analysis and nonlinear response history analysis. Three different ground motions, each of them are further scaled up by 1.5 and 2 times, which makes a total of nine ground motions, are used in the analyses. Plastic rotation demands from linear and nonlinear analysis are compared to each other and to the performance limits suggested by ASCE41 provisions.

There are two main objectives of this study. The first objective is to verify and calibrate the performance limits specified by TEC2007. The second objective is to determine the limitations of linear elastic procedures in seismic assessment of existing structures.

CHAPTER 2

EVALUATION OF PERFORMANCE LIMITS IN SEISMIC ASSESSMENT CODES

2.1. Verification and Calibration Procedure

An evaluation of performance limits in seismic assessment codes is presented in this chapter. Four buildings are analyzed, all of which satisfy the design provisions specified in the Turkish concrete and seismic design codes, namely TS500 and TEC2007.

During the analysis, assessment procedures prescribed in TEC2007 are employed which are linear elastic and nonlinear static analysis procedures. Response spectrum analysis is used as the linear procedure because it considers the higher mode effects of the structures and therefore applicable to all buildings. Linear elastic design spectrum with 475 year return period, which is defined in TEC2007, is used in the analyses. No force reduction factor R is applied to the design spectrum. Design spectrum given in TEC2007 is shown in Figure 2.1 with the corner periods tabulated in Table 2.1.Conventional (first mode) pushover method suggested in TEC2007 is employed as nonlinear static analysis. Pushover method is applicable to some of the structures. For this reason, pushover analysis is implemented to the structures that have a dominant first mode effect. Elastic response spectrum explained above is also used in pushover analysis in order to obtain the target displacement demands of nonlinear models.

Since the four structures employed are designed under the code design spectrum and satisfy the seismic code requirements, it is expected that the assessment of these structures under the same design spectrum by using the performance limits in the seismic assessment codes should result in a satisfactory seismic performance. Accordingly, performance limits set forth for code designed structures can be evaluated by comparing the performance demands with the associated performance limits.

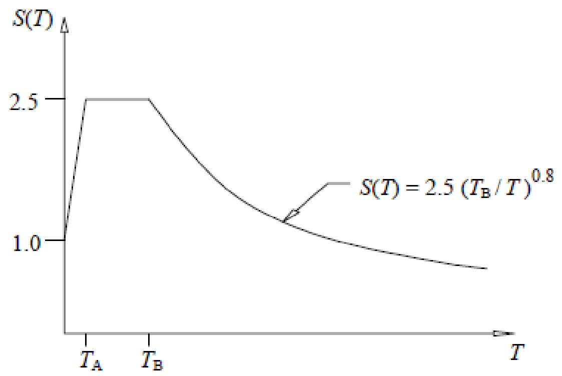


Figure 2.1: Elastic design spectrum with 475 year return period in TEC2007

Table 2.1: Corner periods according to soil types in TEC2007

Soil Class	**T_A (sec)**	**T_B (sec)**
Z1	0,10	0,30
Z2	0,15	0,40
Z3	0,15	0,60
Z4	0,20	0,90

After completing the analyses, two variables are calculated in order to determine the performance levels of the elements and the entire structure. The first variable calculated is demand to capacity ratio (DCR) of each element and the second one is the plastic rotation demands at each member end. DCR is used in force-based assessment whereas plastic rotation demands are used in displacement-based assessment.

2.1.1. Calculation of DCR

Moment demand to flexural capacity ratios at the element ends are calculated herein since flexural behavior is the dominant deformation mode in the structures analyzed. Equation 2.1 is used to calculate DCRs at member ends (TEC 2007).

$$DCR = \frac{M_E}{M_{rc}} \qquad (2.1)$$

In Equation (2.1), M_E is the earthquake moment demand on member ends obtained from the analysis and M_{rc} is the residual capacity moments of the member ends. Residual capacity moment at a member end is defined as the difference between capacity moment and the gravity moment. The bending directions should be taken into consideration in this calculation. The bending directions from both earthquake and gravity forces are shown in Figure 2.2.

Figure 2.2: Bending moment directions at the ends of a beam

2.1.2. Calculation of Plastic Rotations

Plastic rotation demands at member ends can be calculated differently, depending on the type of analysis. In nonlinear static pushover analysis, they are directly calculated by the nonlinear analysis procedure. If linear elastic response spectrum analysis is used for estimating the end rotations, then chord rotations at member ends are calculated first. Chord rotation is the angle between the chord that connects the two ends of a member, and the tangent to the deflected shape at the member end (Sucuoğlu & Günay, 2009). Figure 2.3 explains this definition. After calculating chord rotations, Equation 2.2 is used to obtain plastic rotations at the member ends.

$$\theta_p = \theta_c - \theta_y \qquad (2.2)$$

In Equation 2.2, θ_p is the plastic rotation demand at a member end, θ_c is the chord rotation calculated and θ_y is the yield rotation of the considered member end. Yield rotation is calculated by using the simple relationship between rotation and curvature, which is given in Equation 2.3. Yield curvature ϕ_y of a member depends on the cross-sectional properties and can be obtained from moment-curvature relationship for that cross-section.

$$\theta_y = \phi_y * L_p \qquad (2.3)$$

In Equation 2.3, L_p represents the plastic hinge length of an element. Plastic hinge lengths for beams and columns are calculated as half of the cross-section depth. For shear walls, different plastic hinge lengths L_p are calculated by using Equation 2.4 (Bohl & Adebar, 2011).

$$L_p = 0.43 * d + 0.077 * \frac{\sqrt{z}}{d} \qquad (2.4)$$

In equation 2.4, d is the effective flexural depth, which is approximately 0.8 times the wall length l_w , and z is the depth of shear span that can be taken as the total wall height, h_w.

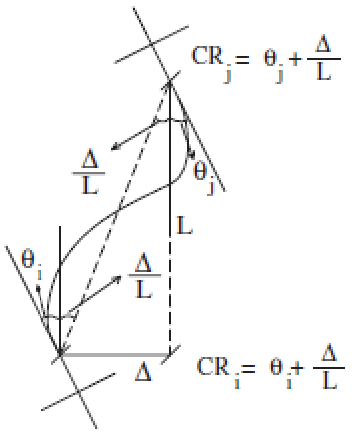


Figure 2.3: Definition of chord rotation at i and j ends of a frame member

DCR’s calculated are used in the force-based assessment procedure of TEC2007. The structures designed for certain performance levels according to

TEC2007 are analyzed by using the assessment procedures defined in the same code. Therefore, the verification of TEC2007 is done for different kinds of structures.

Different performance limits are used in seismic assessment codes. Performance limit of a structure represents its maximum available damage potential under seismic action. Three main performance limits are defined in seismic codes, which are immediate occupancy (IO), life safety (LS) and collapse prevention (CP). If a structure is designed according to life safety limit, it should not deform beyond that limit under seismic action. The performance limits in seismic codes are expressed in terms of either force or deformation quantities.

Plastic rotation demands calculated are used in displacement-based assessment. In this procedure, the limits specified in ASCE41 are used rather than the TEC2007 strain limits. The reason behind this is that calculation of plastic rotation is an easier procedure and plastic rotation is a more physical term than strains, which makes results more understandable. Both force-based and displacement-based assessment procedures are used for the same structures and the results from both procedures are compared to each other in order to further verify the credibility of suggested performance limits.

As explained above, force-based performance limits of TEC2007 and displacement-based performance limits of ASCE41 are used in order to determine the performance levels of structural elements. All of the tables provided in TEC2007 and ASCE41 are given in Appendix A. After determination of performance limits for elements of the structures using those tables, comparison between two seismic codes are carried out and possible changes to TEC2007 are proposed in the conclusion of this Chapter.

2.2. Code Designed Buildings

There are four buildings analyzed in this section. The first one is a six story 3D R/C frame with full capacity design. The second one is a twelve story R/C plane frame with full capacity design. The third one is a twenty story R/C plane wall-frame with full capacity design and the last one is a four story school building retrofitted according to TEC2007.

2.2.1. Six Story 3D R/C Frame with Capacity Design

The first building analyzed is a six story 3D R/C frame with unsymmetrical-plan structure. The story plan and 3D view of the building are given in Figure 2.4 and Figure 2.5, respectively. In order to account for torsion effects, center of mass is shifted away from the center of stiffness, which can be seen also in Figure 2.4. The structure is designed considering the provisions specified in TS500 and TEC2007 with capacity design (Günay, 2008). The column dimensions are 50x50 cm^2 for all columns. The beam dimensions are 55x30 cm^2 for all beams. Although the beam dimensions are the same, during design, different reinforcement amounts are used in order to account for torsion in the building. The cross-sections of beams and columns with the longitudinal reinforcement information and shear reinforcement detailing are shown in Figure 2.6. Information on shear reinforcement used in beams and columns is provided in Table 2.2. All of the stories are 3 meter high, except from the first story, which is 3.5 meter high. The total height of the structure is 18.5 meters. There is no basement, the structure starts from the ground level. In design, enhanced ductility level is chosen (R=8). The structure is in seismic zone 1 on Z3 type soil according to TEC2007. Characteristic strengths of concrete and steel are 25 MPa and 420 MPa, respectively.

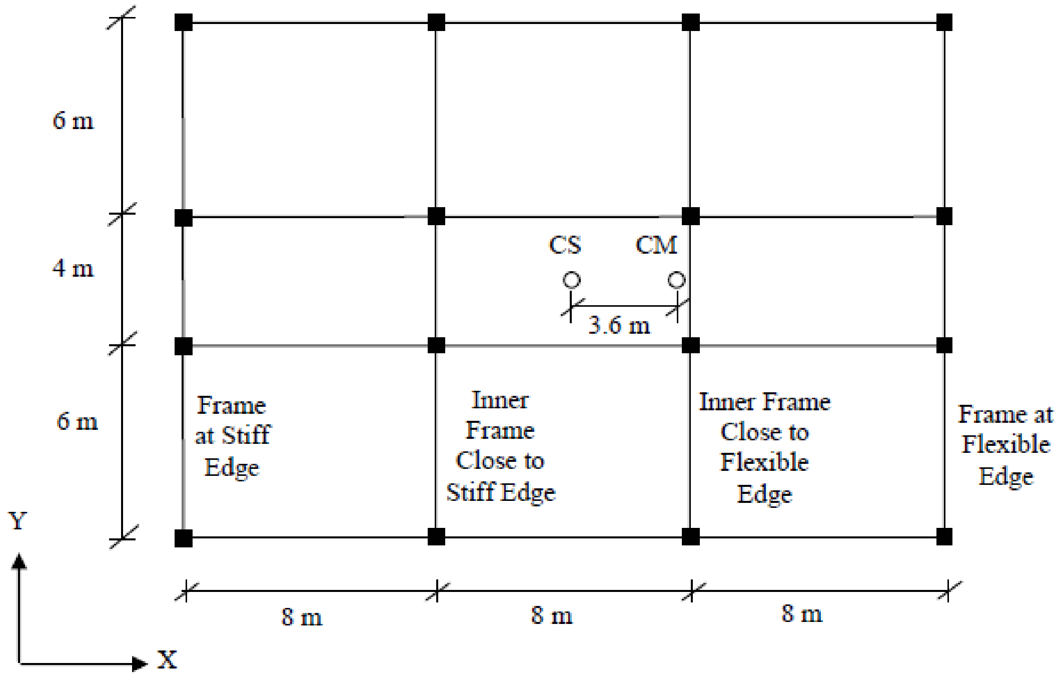


Figure 2.4: Story plan of the six story building with capacity design

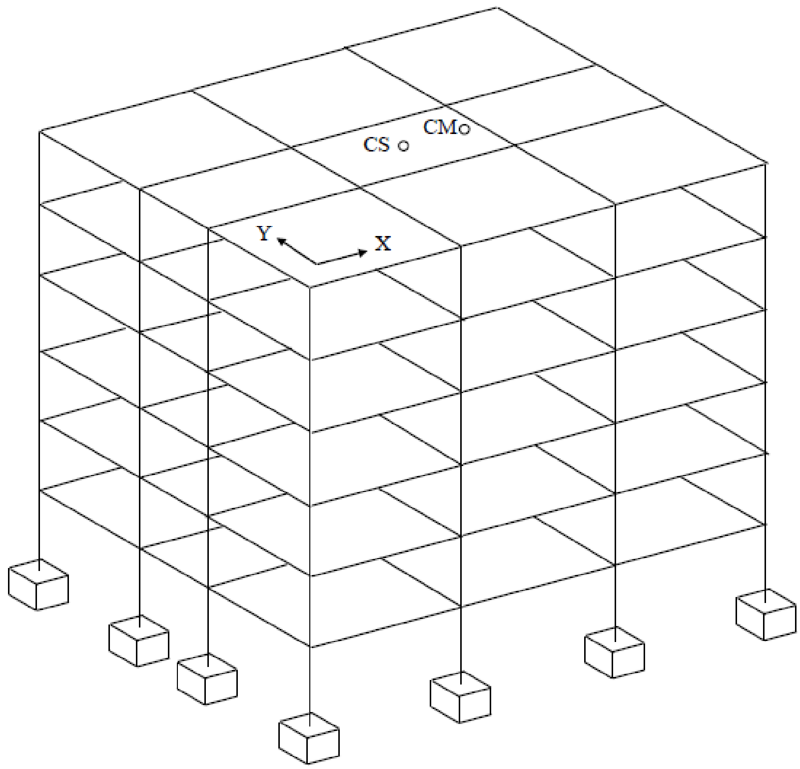
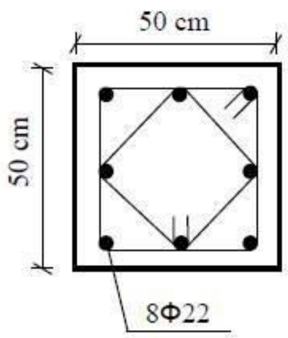
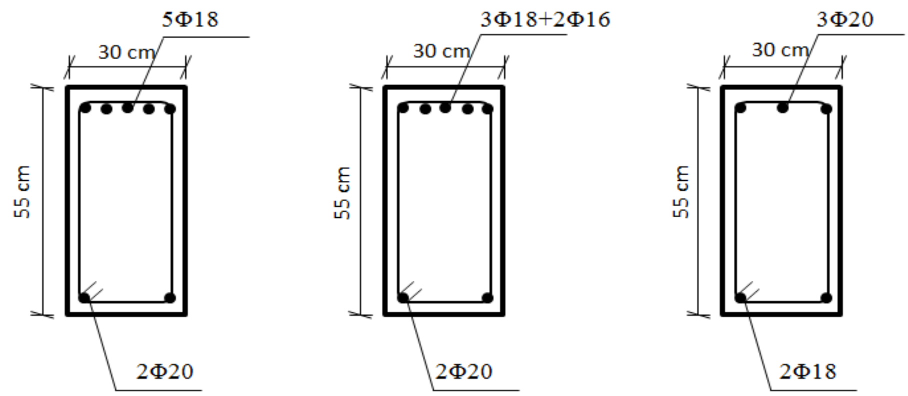


Figure 2.5: 3D view of the six story building with capacity design

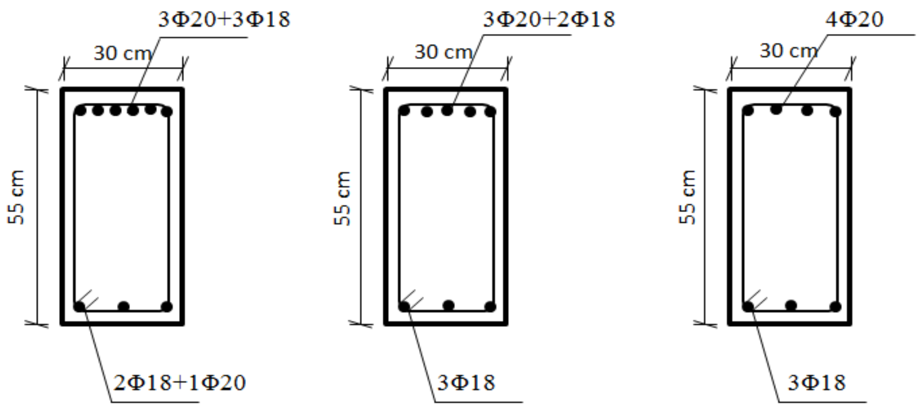


a) Column cross-sections of the building



i ii iii

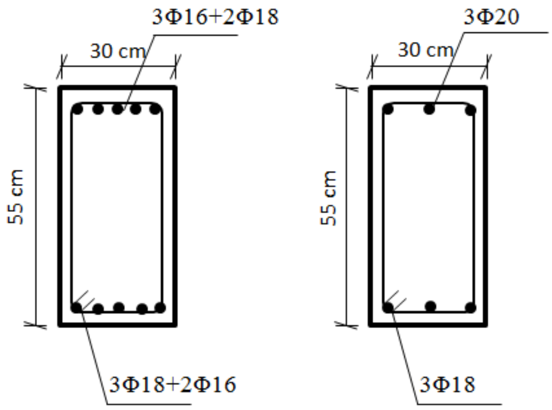
b) x-direction beams, outer frames for stories i.1-2, ii.3-4, iii.5-6



i ii iii

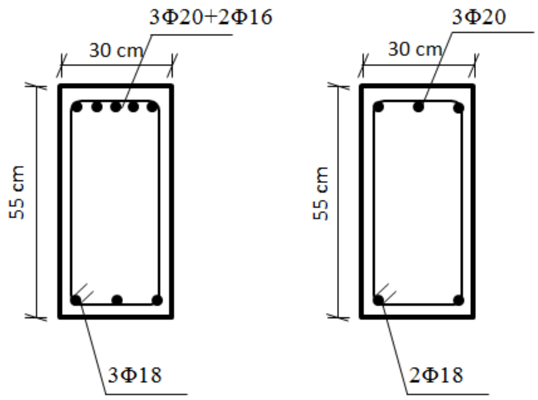
c) x-direction beams, inner frames for stories i.1-2, ii.3-4, iii.5-6

Figure 2.6: Cross-section details of columns and beams of six story building



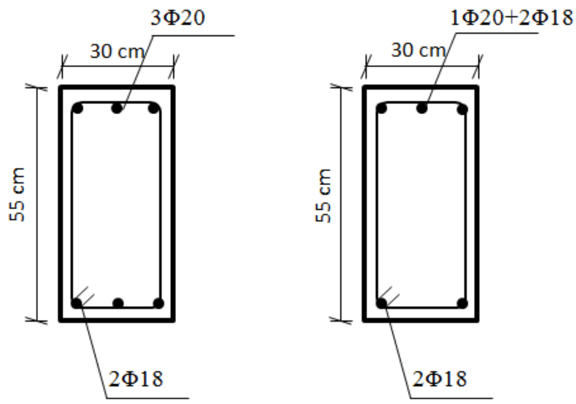
i ii

d) y-direction beams, frame at flexible edge, stories i.1-3, ii.4-6



i ii

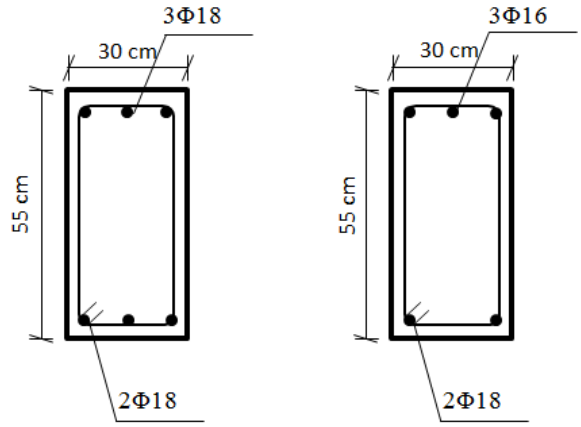
e) y-direction beams, inner frame close to flexible edge, stories i.1-3, ii.4-6



i ii

f) y-direction beams, inner frame close to stiff edge, stories i.1-3, ii.4-6

Figure 2.6 cont'd: Cross-section details of columns and beams of six story building



i ii

g) y-direction beams, frame at stiff edge, stories i.1-3, ii.4-6

Figure 2.6 cont'd: Cross-section details of columns and beams of six story building

Table 2.2: Shear design details of elements of the six story building

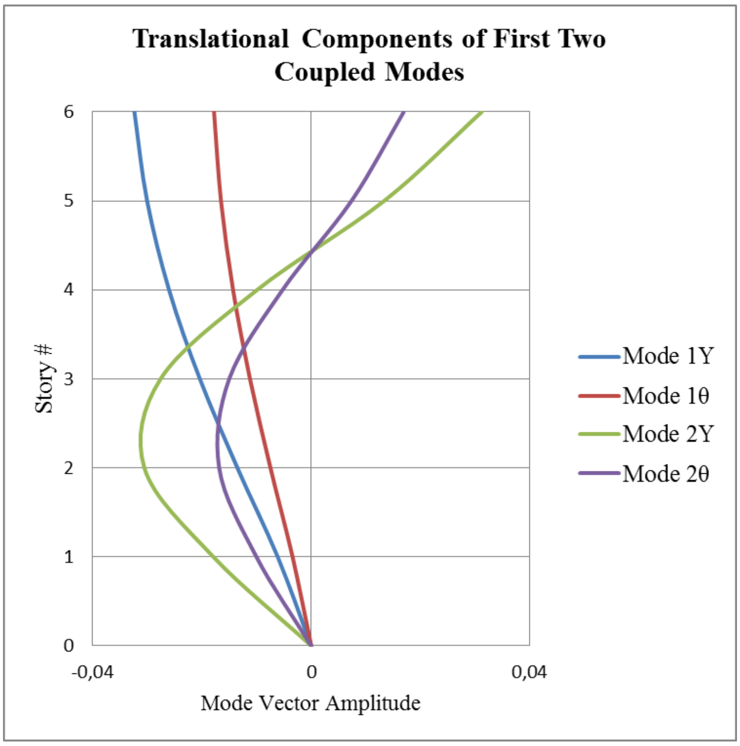
		Along End Region	Along Span Region
Columns	50x50 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
Beams	55x30 cm2	ϕ8 / 10 cm	ϕ8 / 18 cm

3D analytical model of the six story frame is prepared by using the OpenSees software (version 2.4.3). This software is used for both linear and nonlinear analysis. For linear analysis, elastic beam and column elements are defined using the cracked stiffness values, which are obtained by multiplying the gross moment of inertia with 0.4 and 0.6 for beams and columns, respectively. For nonlinear analysis, structural elements are modeled using beam with hinges definition of OpenSees. Plastic hinge lengths for all structural elements are calculated as the half of the cross-section depth. For beams, bi-linear force deformation behavior is used. In order to determine the yield and ultimate points of bi-linear behavior, Response2000 software is used. The moment-curvature results obtained from Response2000 are idealized as bi-linear curves and used as an input to OpenSees model. For columns, fiber sections are used along the plastic hinge length. The reason for using fiber sections in columns, but not in beams is that artificial axial forces occur in the beams. Confined and unconfined concrete are defined separately with the properties of specified reinforcement. The remaining parts of the elements, which contain the zone between plastic hinge regions at the element ends, are defined as elastic beam and column sections. Since the plastic hinges are expected to occur at the member ends, this assumption is valid. The same cracked section properties given above are also used for this part. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the model.

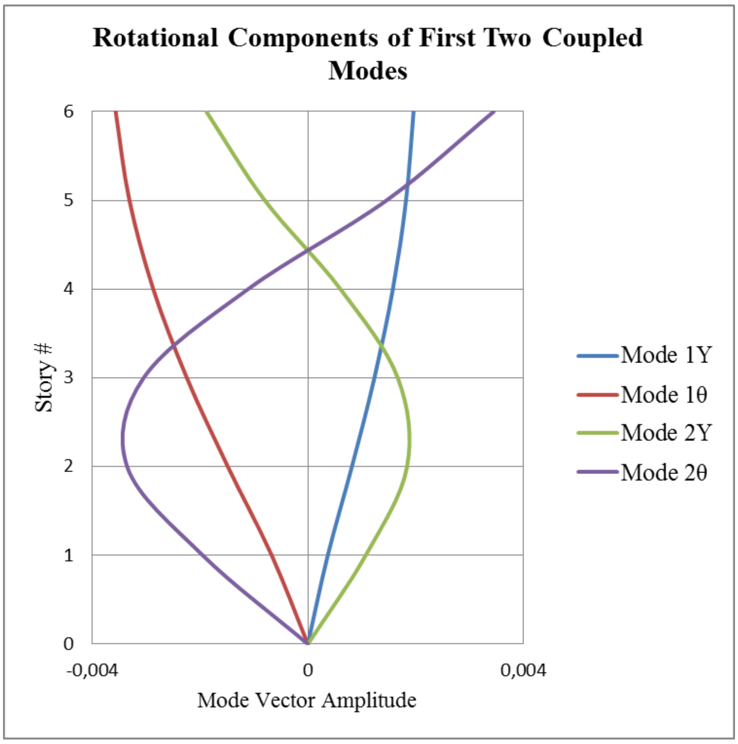
Free vibration properties of six story 3D R/C frame are calculated by eigenvalue analysis of the linear elastic model by using the cracked stiffness values. Modal information regarding the first three coupled modes is tabulated in Table 2.3 and the translational and rotational mode shapes of the first two coupled modes are shown in Figure 2.7.

Table 2.3: Free vibration properties of the first three coupled modes of six story R/C frame

Mode #	**T (sec)**	**Effective Modal Mass (tons)**	**Effective Modal Mass Ratio**
1Y	1,23	921,12	0,66
1θ	0,83	279,17	0,20
2Y	0,39	109,58	0,08
2θ	0,26	33,40	0,02
3Y	0,21	32,60	0,02
3θ	0,14	12,58	0,01



a



b

Figure 2.7: a. Translational and b. Rotational mode shapes of the first two coupled modes of six story R/C frame

2.2.2. Twelve Story R/C Frame with Capacity Design

The second building is a twelve story symmetrical-plan R/C structure. The story plan and 2D view of two adjacent frames are given in Figures 2.8 and 2.9, respectively. The structure is designed by considering the provisions specified in TS500 and TEC2007 for capacity design (Alıcı, 2012). The column dimensions are 50x50 cm^2, 45x45 cm^2 and 40x40 cm^2 for the first four, second four and the last four stories, respectively. Beam dimensions are 55x30 cm^2, 50x30 cm^2 and 45x30 cm^2 for the first four, second four and the last four stories, respectively. The cross-sections of beams and columns with the longitudinal reinforcement information and shear reinforcement detailing are shown in Figure 2.10. Information on shear reinforcement used in beams and columns is provided in Table 2.4. All of the stories are 3.2 meters high except the first story, which is 4 meters high. The total height of the structure is 39.2 meters. There is no basement level, she structure starts from the ground level. In design, enhanced ductility level is used (R=8). The structure is located in seismic zone 1 on Z3 type soil according to TEC2007. Characteristic strengths of concrete and steel are 25 MPa and 420 MPa, respectively.

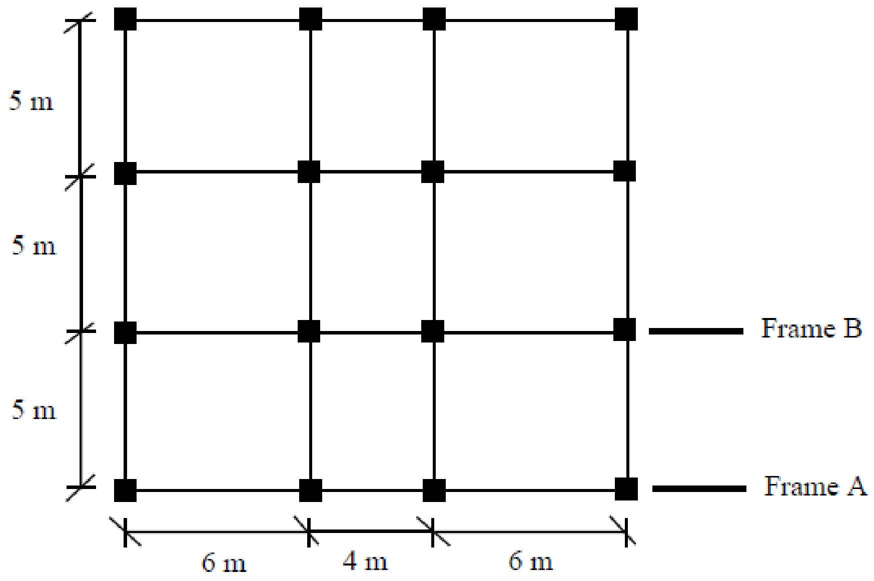


Figure 2.8: Story plan of the twelve story building with capacity design

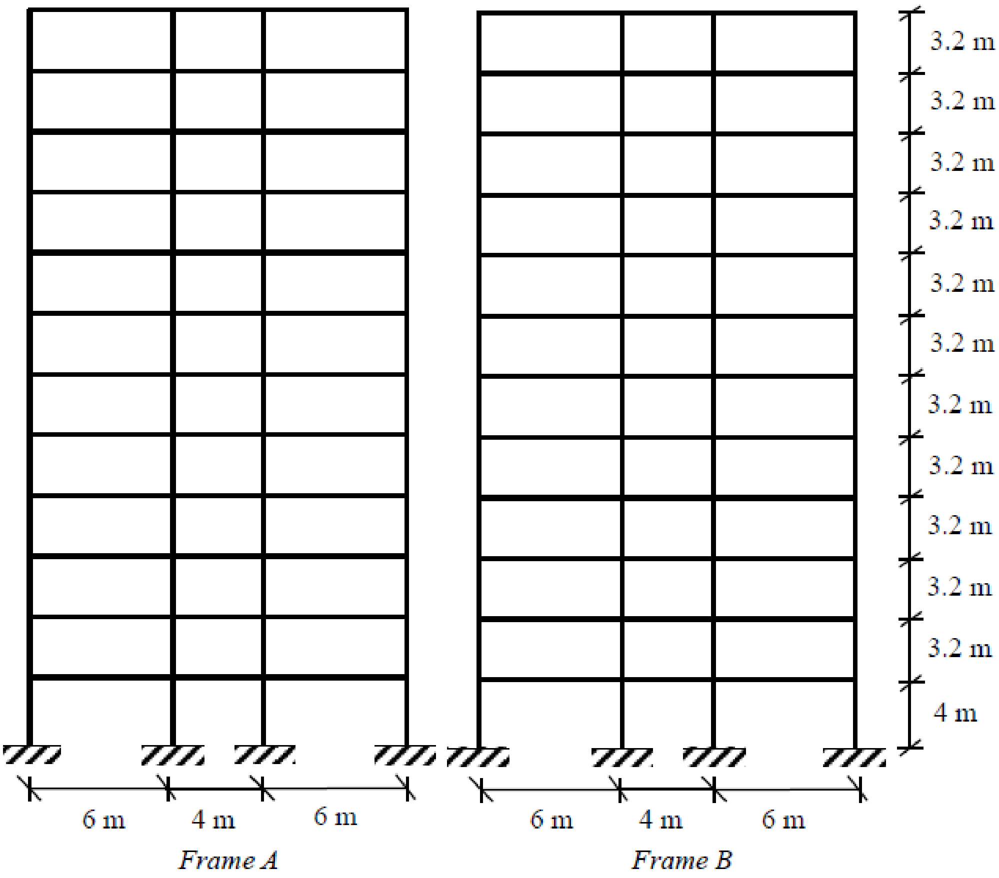
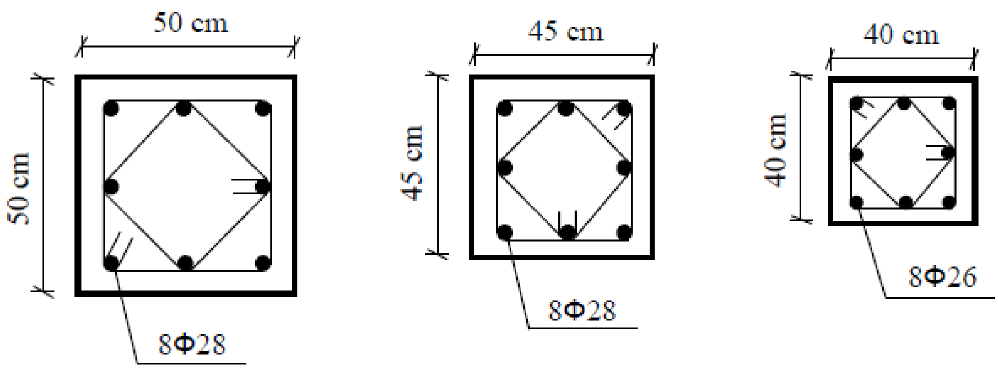
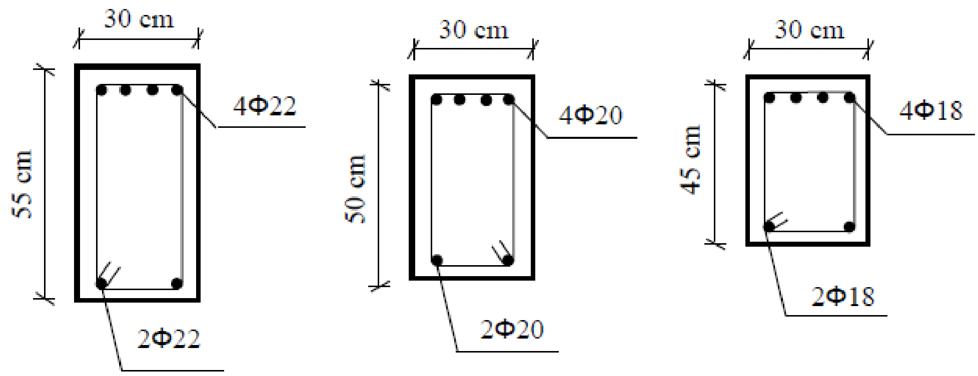


Figure 2.9: 2D model view of frames A and B of the twelve story building

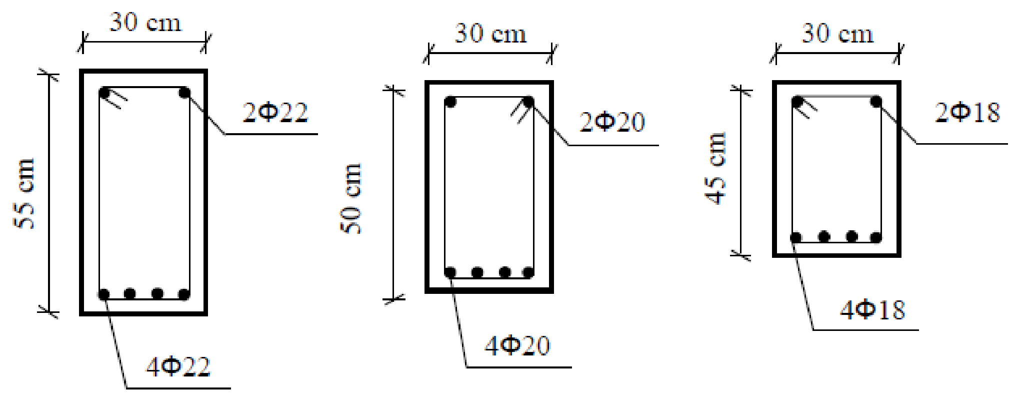


a) Column cross-sections of the building

Figure 2.10: Cross-section details of columns and beams of the twelve story building



b) Support region of beam cross-sections of the building



c) Span region of beam cross-sections of the building

Figure 2.10 Cont'd: Cross-section details of columns and beams of the twelve story building

Table 2.4: Shear design details of elements of twelve story building

		Along End Region	**Along Span Region**
Columns	50x50 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	45x45 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	40x40 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
Beams	55x30 cm2	ϕ8 / 12 cm	ϕ8 / 18 cm
	50x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	45x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm

2D analytical model of the twelve story frame is prepared by using OpenSees software (version 2.4.3). This software is used for both linear and nonlinear analysis. For linear analysis, elastic beam and column elements are defined by using the cracked stiffness values, which are obtained by multiplying the gross moment of inertia by 0.4 and 0.6 for beams and columns, respectively. For nonlinear analysis, structural elements are modeled by using "beam with hinges" definition of OpenSees. Plastic hinge lengths for all structural elements are calculated as half of the cross-section depth. For beams, bi-linear force deformation behavior is used. In order to determine the yield and ultimate points of bi-linear behavior, Response2000 software is used. For columns, fiber sections are used along the plastic hinge length. Confined and unconfined concrete are defined separately with the properties of specified reinforcement. The remaining parts of the elements, which contain the zone between plastic hinge regions at the element ends, are defined as elastic beam and column sections. Since the plastic hinges are expected to occur at the member ends, this assumption is valid. The same cracked section properties given above are also used for this part. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the model.

Free vibration properties of the twelve story R/C frame with full capacity design are calculated by eigenvalue analysis of the linear elastic model with the cracked stiffness values. Modal information regarding the first four modes is tabulated in Table 2.5 and the modes shapes of the first four modes are shown in Figure 2.11.

Table 2.5: Free vibration properties of the first four modes of twelve story R/C frame with full capacity design

Mode #	**T (sec)**	**Effective Modal Mass (tons)**	**Effective Modal Mass Ratio**
1	2,39	434,30	0,79
2	0,82	66,61	0,12
3	0,48	21,75	0,04
4	0,32	10,24	0,02

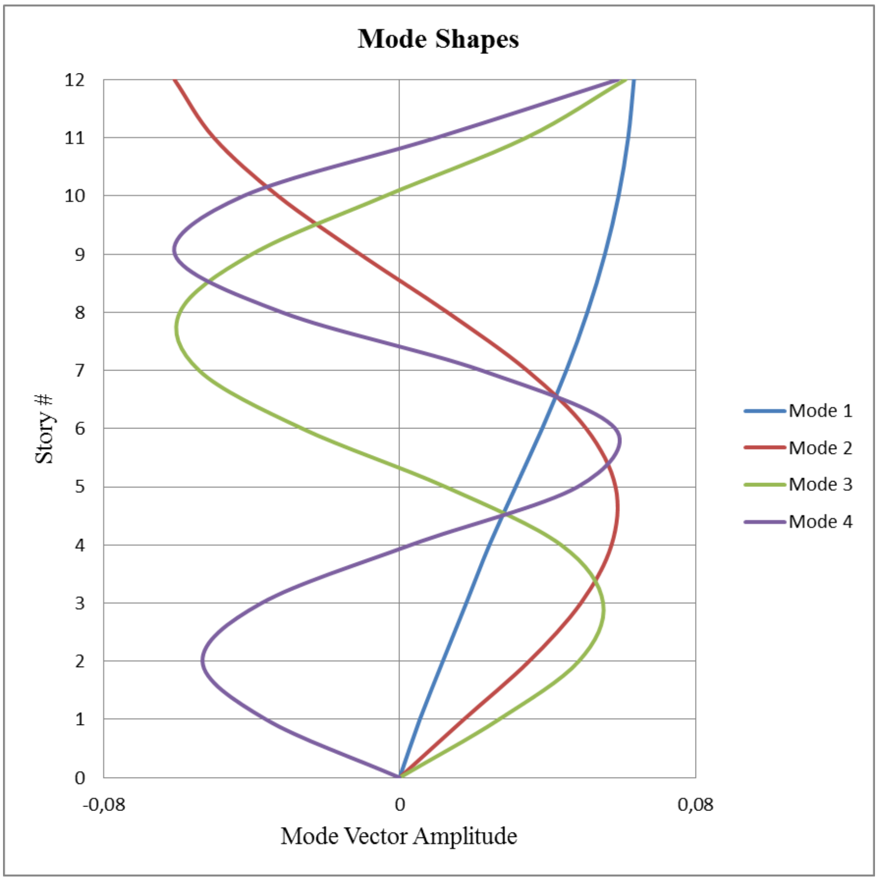


Figure 2.11: Mode shapes of the first four modes of twelve story R/C frame with capacity design

2.2.3. Twenty Story R/C Wall-Frame with Capacity Design

The third building to be analyzed is a twenty story symmetrical-plan R/C wall-frame structure. The story plan and 2D view of two adjacent frames are shown in Figures 2.12 and 2.13, respectively. Two 4 meter long and 0.3 meter thick shear walls are placed at the middle of interior frames in the direction of earthquake excitation. The structural members are designed considering the provisions specified in TS500 and TEC2007 (Alıcı, 2012). The column dimensions are 55x55 cm^2, 50x50 cm^2 and 45x45 cm^2 for the first six, second six and the last eight stories, respectively. Beam dimensions are 55x30 cm^2, 50x30 cm^2 and 45x30 cm^2 for the first six, second six and the last eight stories, respectively. The cross-sections of beams and columns with the longitudinal reinforcement information and shear reinforcement detailing are shown in Figure 2.14, and the cross-section details of shear walls are provided in Figure 2.15. Information on shear reinforcement used in beams and columns is provided in Table 2.6. All stories are 3.2 meter high, except the first story, which is 4 meters, similar to the twelve story building. The total height of the structure is 64.8 meters. There is no basement level, the structure starts from the ground level. In design, enhanced ductility level is chosen (R=7). The structure is in seismic zone 1 on Z3 type soil according to TEC2007. Characteristic strengths of concrete and steel are 35 MPa and 420 MPa, respectively.

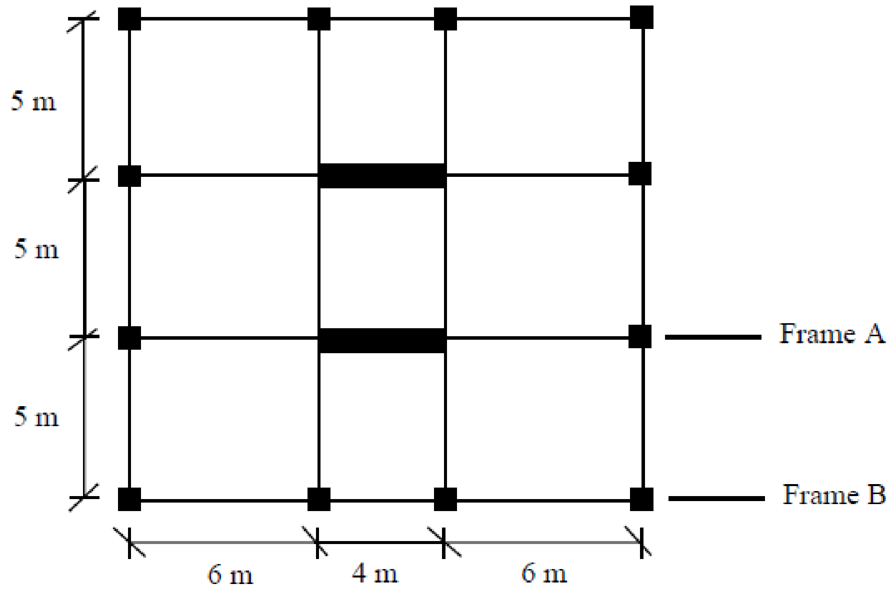


Figure 2.12: Story plan of the twenty story wall-frame building with full capacity design

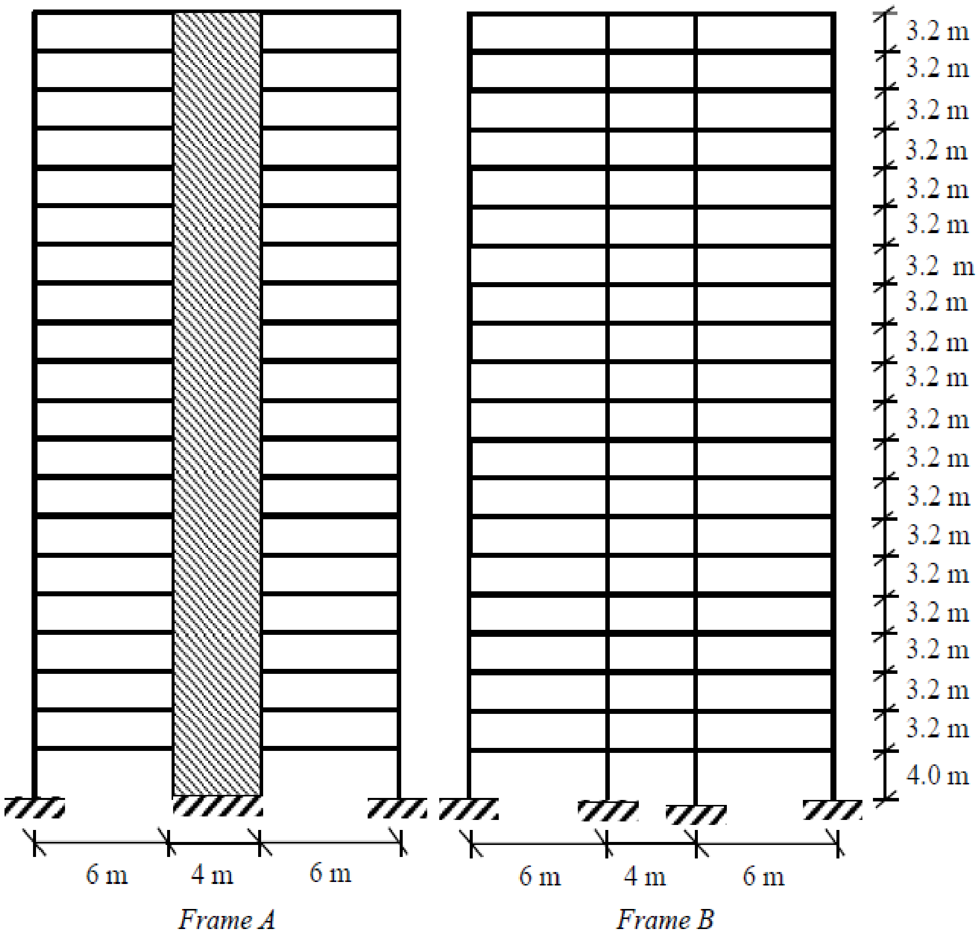
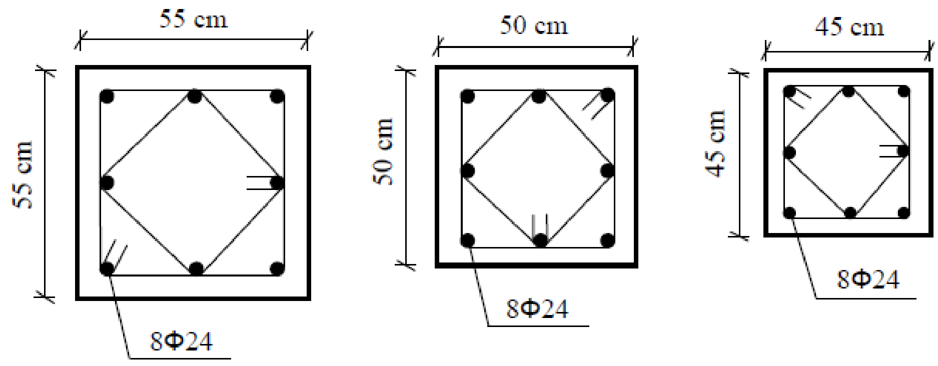
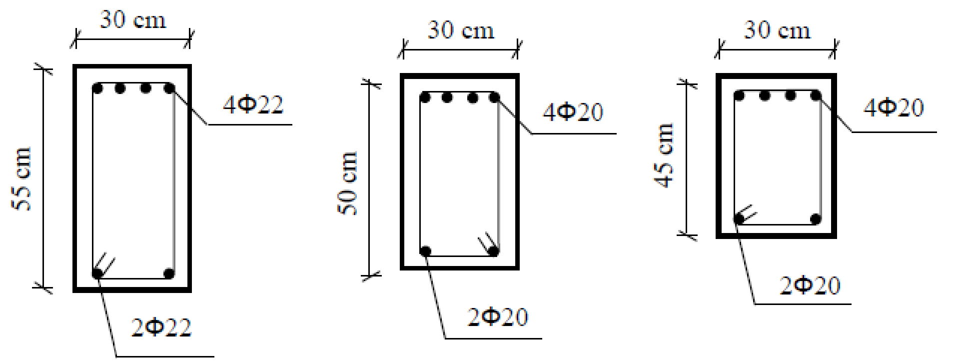


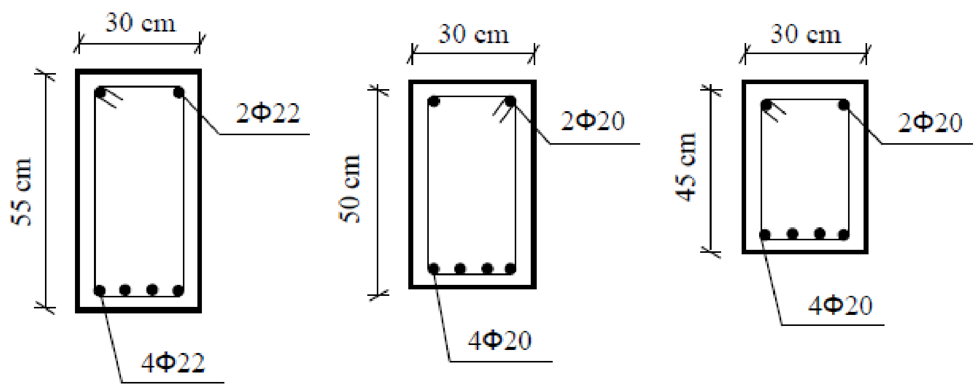
Figure 2.13: 2D model view of frames A and B of the twenty story wall-frame building



a) Column cross-sections

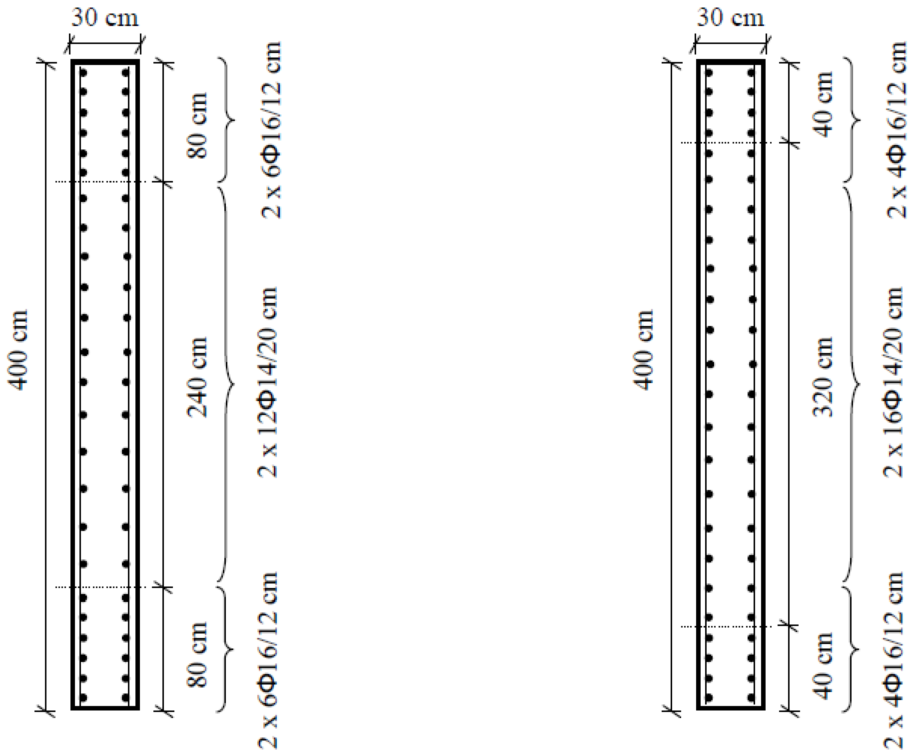


b) Support region of beam cross-sections of the building



c) Span region of beam cross-sections of the building

Figure 2.14 Cross-section details of columns and beams of the twenty story wall-frame building



a) For H < H_{cr} (H_{cr} =7.2 m) b) For H > H_{cr}

Figure 2.15: Cross-section details of shear walls of the twenty story wall-frame building

Table 2.6: Shear design details of columns and beams of twenty story wall-frame building

		Along End Region	**Along Span Region**
Columns	55x55 cm2	ϕ8 / 10 cm	ϕ8 / 20 cm
	50x50 cm2	ϕ8 / 10 cm	ϕ8 / 20 cm
	45x45 cm2	ϕ8 / 10 cm	ϕ8 / 20 cm
Beams	55x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	50x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	45x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm

2D analytical model of the twenty story wall-frame is prepared by using the OpenSees software. This software is used for both linear and nonlinear analysis. For linear analysis, elastic beam, column and shear wall elements are defined using the cracked stiffness values, which are obtained by multiplying the gross moment of inertia with 0.4 and 0.6 for beams and columns, respectively. For nonlinear analysis, structural elements are modeled using beam with hinges definition of OpenSees. Plastic hinge lengths for beams and columns are calculated as half of the cross-section depth. For shear wall, additional nodes are defined at the mid-heights of first and second stories to account for the plastic rotations that are expected to occur along the critical height, H_{cr}. Plastic hinges are defined along nodes within H_{cr} whereas for the rest of the structure, plastic hinges are defined for very small distances since plasticity of shear wall is expected to concentrate along the critical length. For beams, bi-linear force deformation behavior is used. In order to determine the yield and ultimate points of bi-linear behavior, Response2000 software is used. The moment-curvature results obtained from Response2000 are idealized as bi-linear curves and used as an input to OpenSees model. For columns, fiber sections are used along the plastic hinge length. For shear wall members, bi-linear moment-curvature relationship with shear aggregator is defined along the plastic hinge length. Confined and unconfined concrete are defined separately with the properties of specified reinforcement. The remaining parts of the elements, which contain the zone between plastic hinge regions at the element ends, are defined as elastic beam and column sections. The same cracked section properties given above are also used for this part. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the model.

Free vibration properties of the twenty story R/C wall-frame are calculated by eigenvalue analysis of the linear elastic model by using the cracked stiffness values. Modal information regarding the first four modes is tabulated in Table 2.7 and the modes shapes of the first four modes are shown in Figure 2.16.

Table 2.7: Free vibration properties of the first four modes of twenty story R/C wall-frame with full capacity design

Mode #	T (sec)	Effective Modal Mass (tons)	Effective Modal Mass Ratio
1	2,60	717,97	0,68
2	0,72	158,70	0,15
3	0,31	63,83	0,06
4	0,17	35,22	0,03

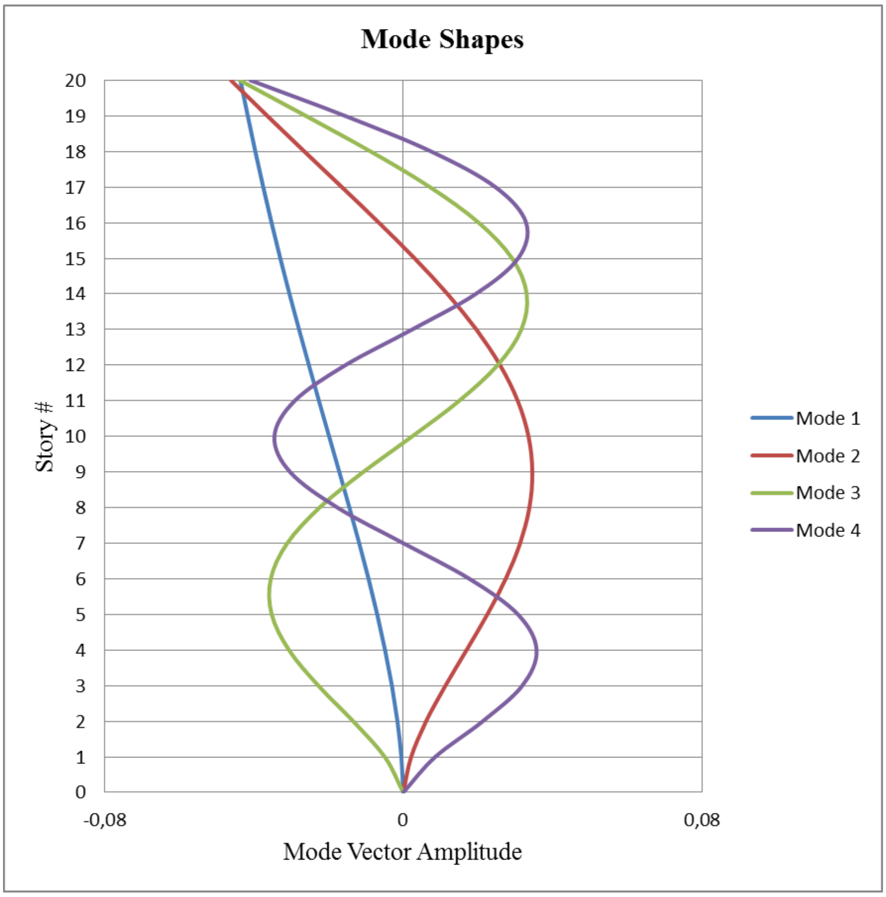


Figure 2.16: Mode shapes of the first four modes of twenty story R/C wall-frame with capacity design

2.2.4. Four Story Retrofitted R/C School Building

The last building analyzed in this section is a real case structure, which is a four story retrofitted R/C school building in Istanbul. The story plan and 3D view of the building are shown in Figures 2.17 and 2.18, respectively. Story plan does not change between stories; however member dimensions are different. 60x40 cm^2 and 60x30 cm^2 columns are present in the structure. Beam dimensions of the building are 80x35 cm^2 and 80x30 cm^2. There are existing shear walls and added shear walls during the retrofitting procedure. The existing shear walls have dimensions of 390x30 cm^2 and 360x30 cm^2. The general layout of the members can also be observed from Figure 2.17. Cross-section properties of beams and columns are given in Figure 2.20. Typical cross-section of a shear wall of the existing structure is shown in Figure 2.21. Information on shear reinforcement of beams and columns is provided in Table 2.8. The building has dimensions of 18.2, 45.2 and12.8 meters in x, y and z (height) directions, respectively. All of the stories are 3.2 meters high. There is no basement. The structure is located in seismic zone 1 on Z3 type soil. Characteristic strengths of concrete and steel are reported (Structure Evaluation Report of Kağıthane Ferit Aysan Çağdaş Yaşam Primary School, 2010) as 23 MPa and 420 MPa, respectively.

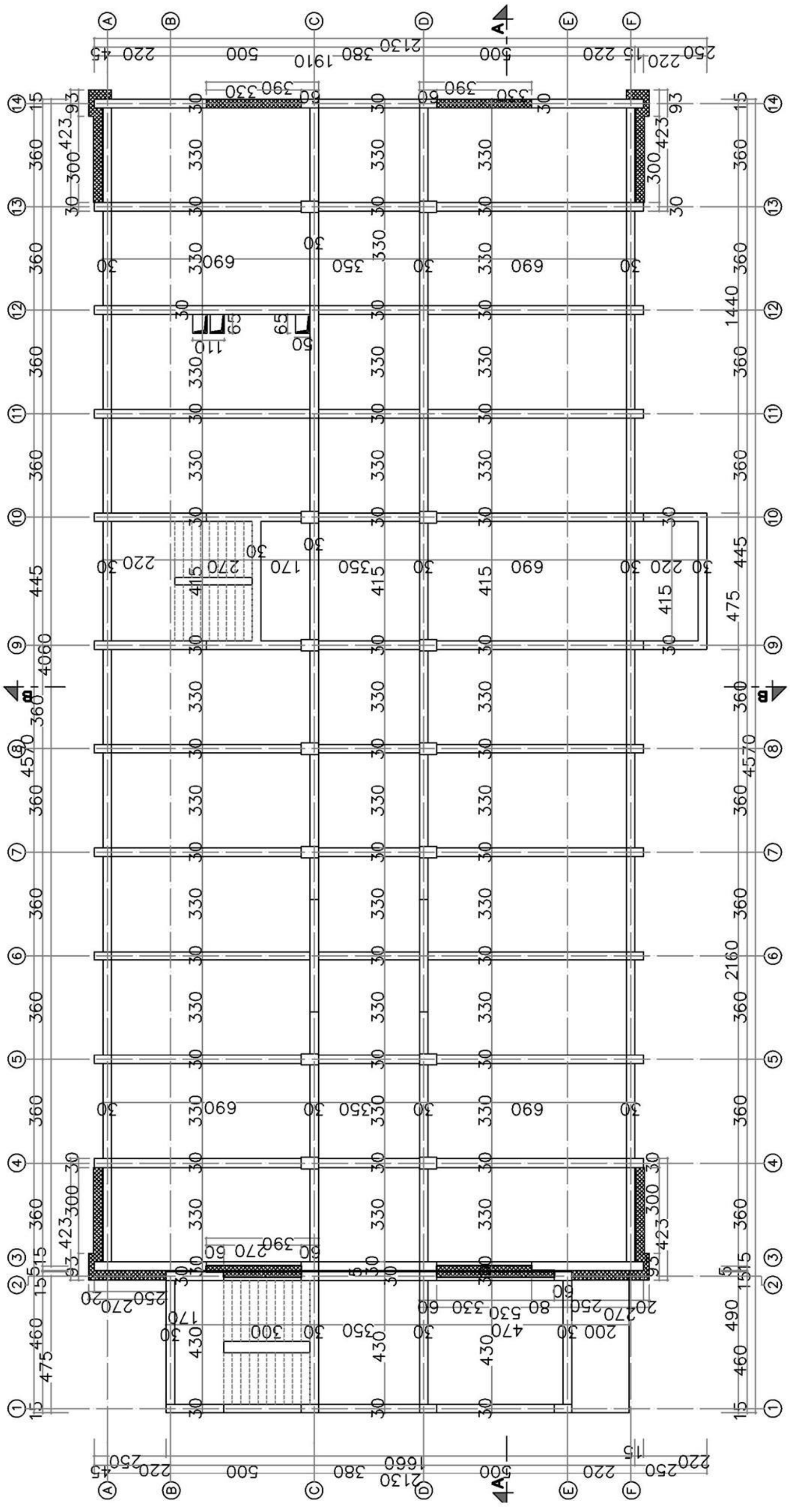


Figure 2.17: Story plan of the four story retrofitted school building

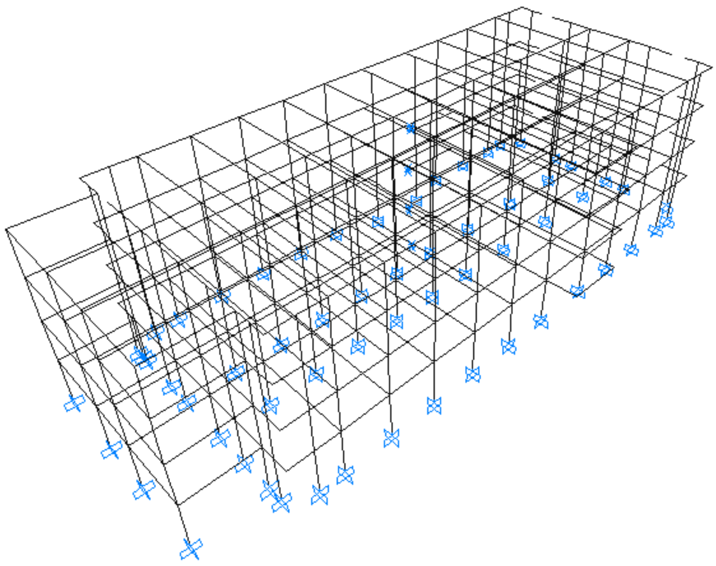
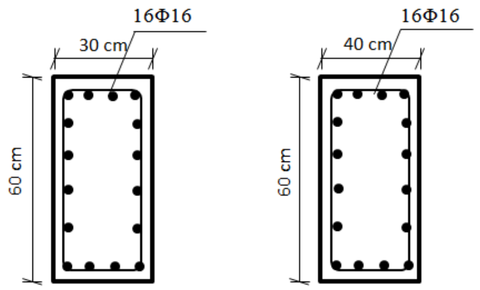
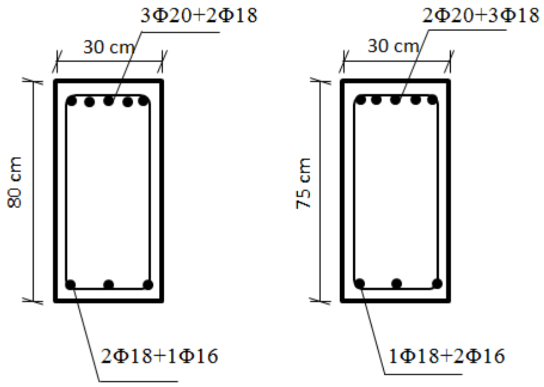


Figure 2.18: 3D view of the four story retrofitted school building



a) Typical column cross-sections of the building



b) Typical beam cross-sections of the building

Figure 2.19: Cross-section details of columns and beams of the four story retrofitted school building

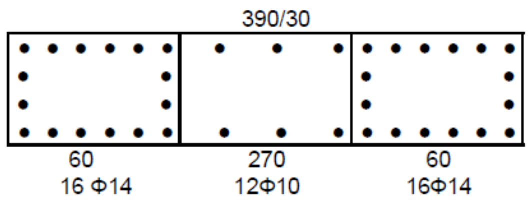


Figure 2.20: Typical cross-section of an existing shear wall of the four story retrofitted school building

Table 2.8: Shear design details of elements in the four story retrofitted building

		Along End Region	Along Span Region
Columns	All	ϕ8 / 20 cm	ϕ8 / 20 cm
Beams	All	ϕ8 / 25 cm	ϕ8 / 25 cm

3D analytical model of the four story retrofitted building is prepared by using the OpenSees software. This software is used for both linear and nonlinear analysis. For linear analysis, elastic beam, column and shear wall elements are defined with the previously mentioned cracked stiffness values. For nonlinear analysis, structural elements are modeled using beam with hinges elements of OpenSees. Plastic hinge lengths for beams and columns are calculated as half of the cross-section depth. For different shear walls, different plastic hinge lengths, L_p are calculated by using Equation 2.4. For the rest of the walls, plastic hinges are defined for very small distances since plasticity of shear wall is expected to concentrate inside the plastic hinge region. For beams, bi-linear force deformation behavior is used. In order to determine the yield and ultimate points of bi-linear behavior, Response2000 software is used. The moment-curvature results obtained from Response2000 are idealized as bi-linear curves and used as an input to OpenSees model. For columns and shear walls, fiber sections are used along the plastic hinge length considering the typical reinforcement information given in Figures 2.19 and 2.20. Since it is not possible to exactly know the details used in all structural elements of an existing structure, typical sections, which are obtained by several investigations and tests on the structure given above should be enough to model this building. Confined and unconfined concrete are defined separately with the properties of specified reinforcement. The remaining parts of the elements, which contain the zone between plastic hinge regions at the element ends, are defined as elastic beam and column sections. The same cracked section properties given above are also used for this part. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the analysis.

Free vibration properties of the four story R/C retrofitted building are calculated by eigenvalue analysis, with the cracked stiffness values. Modal information regarding the first four modes is tabulated in Table 2.9 and the shapes of the first four modes are shown in Figure 2.21. Torsion is not effective in this structure. After initial analysis, it is seen that y-direction is more critical for the structure; therefore, properties of x-direction modes are not included in the tables below.

Table 2.9: Free vibration properties of the first three translational modes of the four story retrofitted building

Mode #	**T (sec)**	**Effective Modal Mass (tons)**	**Effective Modal Mass Ratio**
1	0,31	4697,22	0,85
2	0,07	615,13	0,11
3	0,01	99,92	0,02

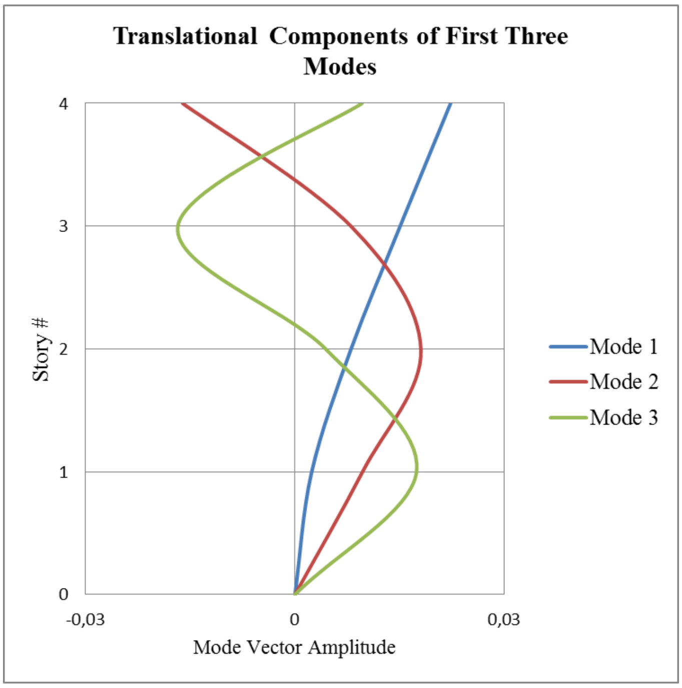


Figure 2.21: Mode shapes of the first three translational modes of four story retrofitted building

2.3. Presentation of Results

Results of analyses on the case study buildings defined above are presented separately for each structure.

2.3.1. 6 Story 3D R/C Frame with Capacity Design (LS; R=8)

Both of the analysis procedures, namely response spectrum and conventional pushover analysis, are applicable to this structure. Therefore, both force-based and displacement-based results are obtained.

The design of this structure is carried out according to the life safety performance level with a force reduction factor of R = 8. Therefore, the calculated performance quantities such as DCRs and plastic rotation demands at member ends are compared with the LS performance limits for both force-based and displacement-based procedures. For force-based procedure, the ratios of calculated DCR to the limit DCR for LS performance limit are plotted for each story. For displacement-based procedure, the ratios of plastic rotation demands to the plastic rotation limits for LS performance are plotted for each story. Rather than considering all elements individually, mean quantities of elements for each frame, namely flexible edge frame, inner frame to flexible edge, inner frame to stiff edge and stiff edge frame, are plotted separately for each story. The results are shown in Figure 2.22. The maximum torsional irregularity coefficient η_{bi}, according to TEC2007 is calculated as 1.50, which implies that the structure has significant torsional irregularity.

Maximum values of performance limits in TEC2007 and ASCE41 are given in Table 2.10. Only maximum values are put in that table because the calculated performance limits are also close to the maximum values for code-designed buildings. The *r factors* from TEC2007 for force-based assessment and θ_p values from ASCE41 for displacement-based assessment are inserted in that table.

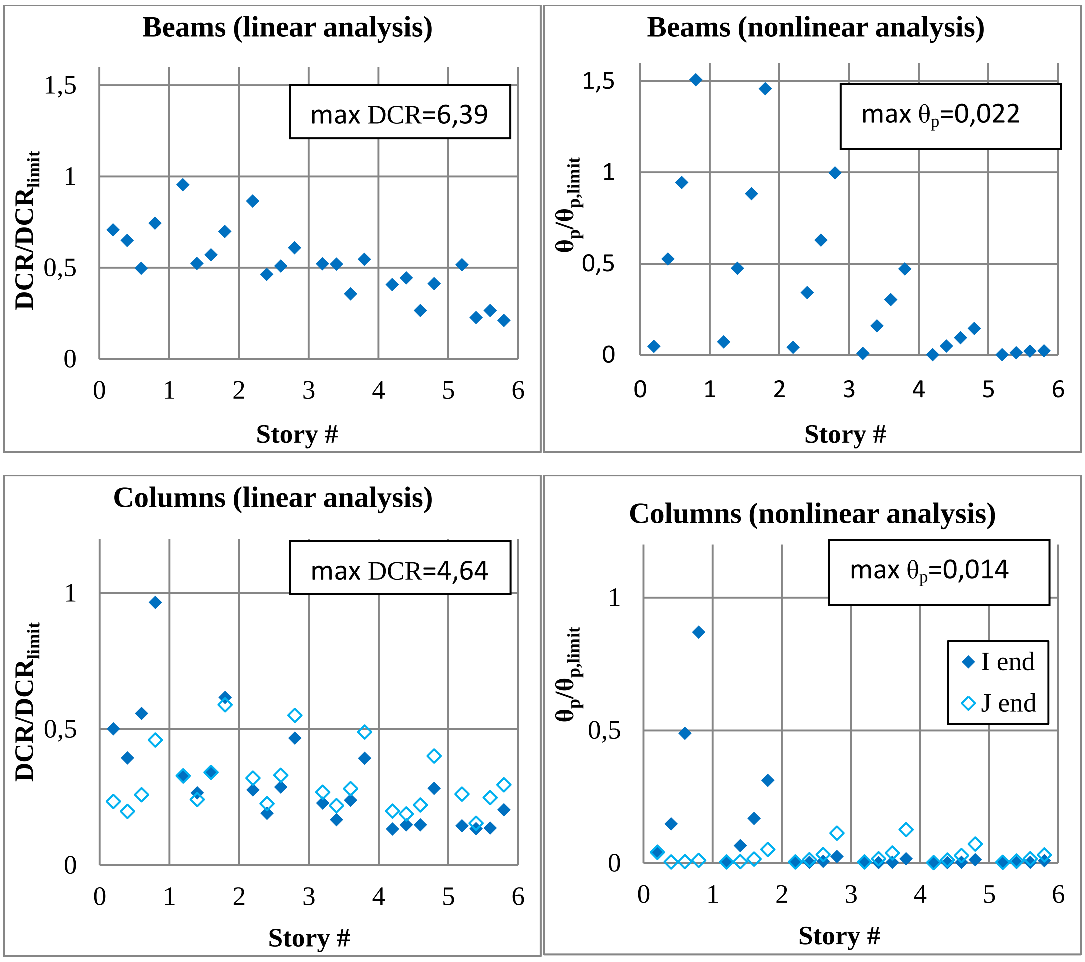


Figure 2.22: Comparative evaluation of performance parameters for the 6 Story 3D R/C Frame

Table 2.10: Maximum values of performance limits in TEC2007 and ASCE41 for beams and columns

		IO	LS	CP
Beams	TEC2007 (*r* factors-DCR)	3	7	10
	ASCE41 (θ_p)	0,010	0,020	0,025
Columns	TEC2007 (*r* factors-DCR)	3	6	8
	ASCE41 (θ_p)	0,005	0,026	0,035

The following observations are evident from the presented results.

- Since the building has significant torsional irregularity, linear analysis cannot capture the effects of nonlinear response accurately. Flexible edge beams and columns are much more affected than the stiff edge beams and columns, which is reasonable because the mass center of the building is 15% shifted from the stiffness center.
- Although it is a full capacity design, displacement-based (nonlinear) assessment of six story building indicates that the structure does not satisfy the LS performance limit. This is caused by the fact that design procedure is a force-based one with linear elastic analysis, and the structure has a significant torsional irregularity. Design forces obtained from linear elastic analysis seems not correct considering the post-yielding behavior of the structure since linear elastic analysis cannot predict the redistribution of forces after plastic hinge mechanisms occur at the flexible edge.
- If force-based assessment of TEC2007 is employed for this building, the structure is found safe. However, it is observed that the structure does not satisfy the pre-determined performance level according to displacement-based assessment. Therefore, it is necessary to check the irregularities of a structure, and if they exceed certain limits, nonlinear analysis methods should be used. This issue is further discussed in Chapter 3.

2.3.2. 12 Story R/C Frame with Capacity Design (LS; R=8)

Only response spectrum analysis is applicable to this structure because effective higher mode contribution prevents employing conventional pushover analysis. Therefore, results of response spectrum analysis are used for both force-based and displacement-based assessments for this building.

The design of this structure is for the life safety performance objective with the force reduction factor R = 8. Obtained performance quantities, which are DCRs and plastic rotation demands, are compared to LS performance limits for the force-based and displacement-based procedures. Similar to the previous structure, the ratios of calculated performance quantities to the limit quantities for LS performance limits are plotted for each story. Rather than considering all elements individually, mean quantities of elements for each story are plotted separately. The results are shown in Figure 2.23.

Maximum values of performance limits in TEC2007 and ASCE41 are given in Table 2.11. Only maximum values are given in the table because the other performance limits calculated are also close to the maximum values.

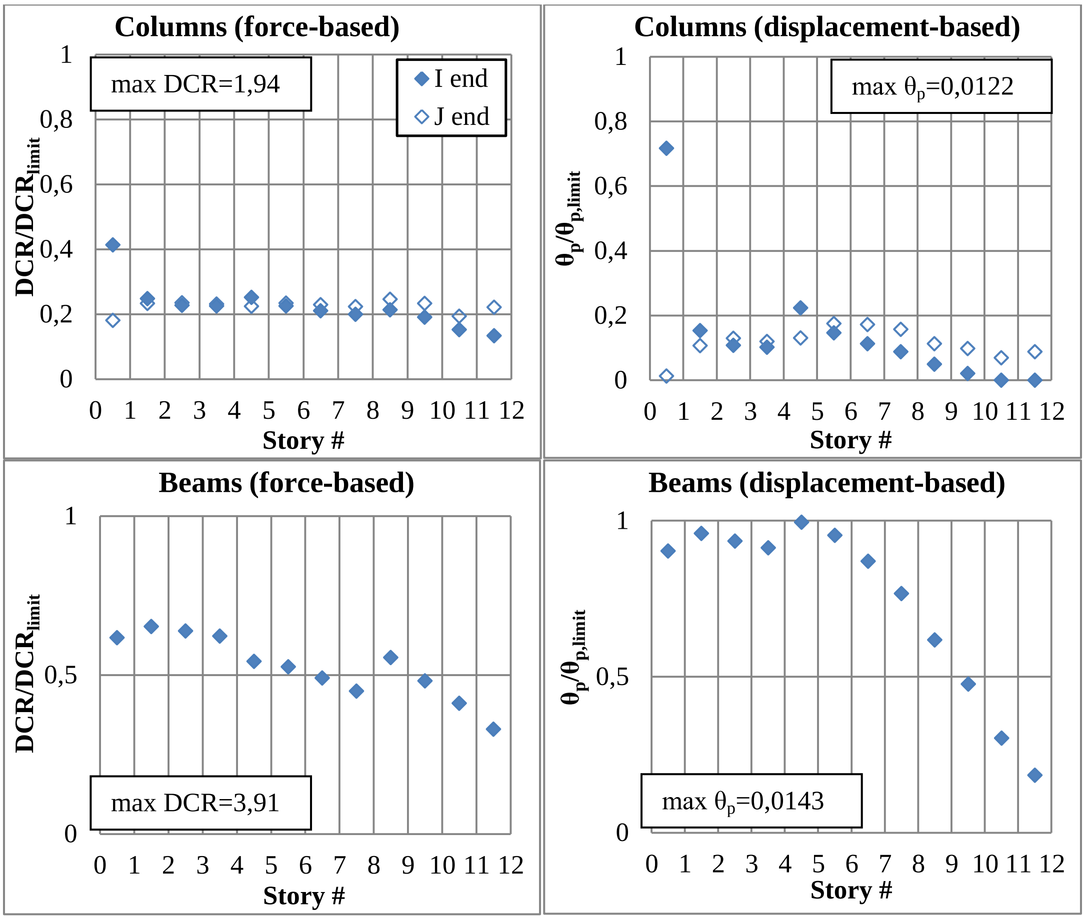


Figure 2.23: Comparative evaluation of performance parameters for the 12 Story R/C frame

Table 2.11: Maximum values of performance limits in TEC2007 and ASCE41 for beams and columns for twelve story building

		IO	LS	CP
Beams	TEC2007 (*r* factors-DCR)	3	7	10
	ASCE41 (θ_p)	0,010	0,020	0,025
Columns	TEC2007 (*r* factors-DCR)	3	6	8
	ASCE41 (θ_p)	0,005	0,026	0,035

The following conclusions can be derived from the results shown in Figure 2.23 and Table 2.11.

- Since this is a capacity designed structure and does not have any irregularities, it is expected that it should satisfy the aimed performance level. Considering DCR/DCR_{limit} values, they are expected to be less than 1 for column members, but close to 1 for beams. This is satisfied for the displacement based assessment, however, not for the force based assessment. This is an indication that the force based performance limits for beams are generous.

- Considering the DCR/DCR_{limit} values at column ends, they appear to be quite low, as a consequence of strong column-weak beam approach in design. However bottom ends of base columns are expected to develop plastic hinges. It is noteworthy that the displacements based procedure predicts the critical situation at the base of bottom story columns where a plastic hinge is expected. The force based procedure assigns a much higher performance. It may be considered to adjust the force based performance limits of TEC2007 for columns accordingly.
- Although the structure satisfies its performance level with respect to both procedures, displacement-based results are much more critical than the force based results for both beams and columns of the building. This fact also suggests that performance limits for force-based assessment of TEC2007 are too generous in general and needs to be adjusted.

2.3.3. 20 Story R/C Wall-Frame with Capacity Design (LS; R=7)

Similar to the twelve story R/C frame, only response spectrum analysis is applicable to this structure because of higher mode effects. Therefore, results of response spectrum analysis are used for both force-based and displacement-based assessments of this building, too.

This structure is designed according to the life safety performance with a force reduction factor R = 7. Calculated performance quantities, namely DCRs and plastic rotation demands, are compared to the LS performance limits for force-based and displacement-based procedures, respectively. Similar to the previous structure, the ratios of calculated performance quantities to the limit quantities for LS performance limits are plotted for each story. Rather than considering all elements individually, mean quantities of elements for each story are plotted separately. The results are shown in Figure 2.23. Maximum values of performance limits in TEC2007 and ASCE41 are given in Table 2.11.

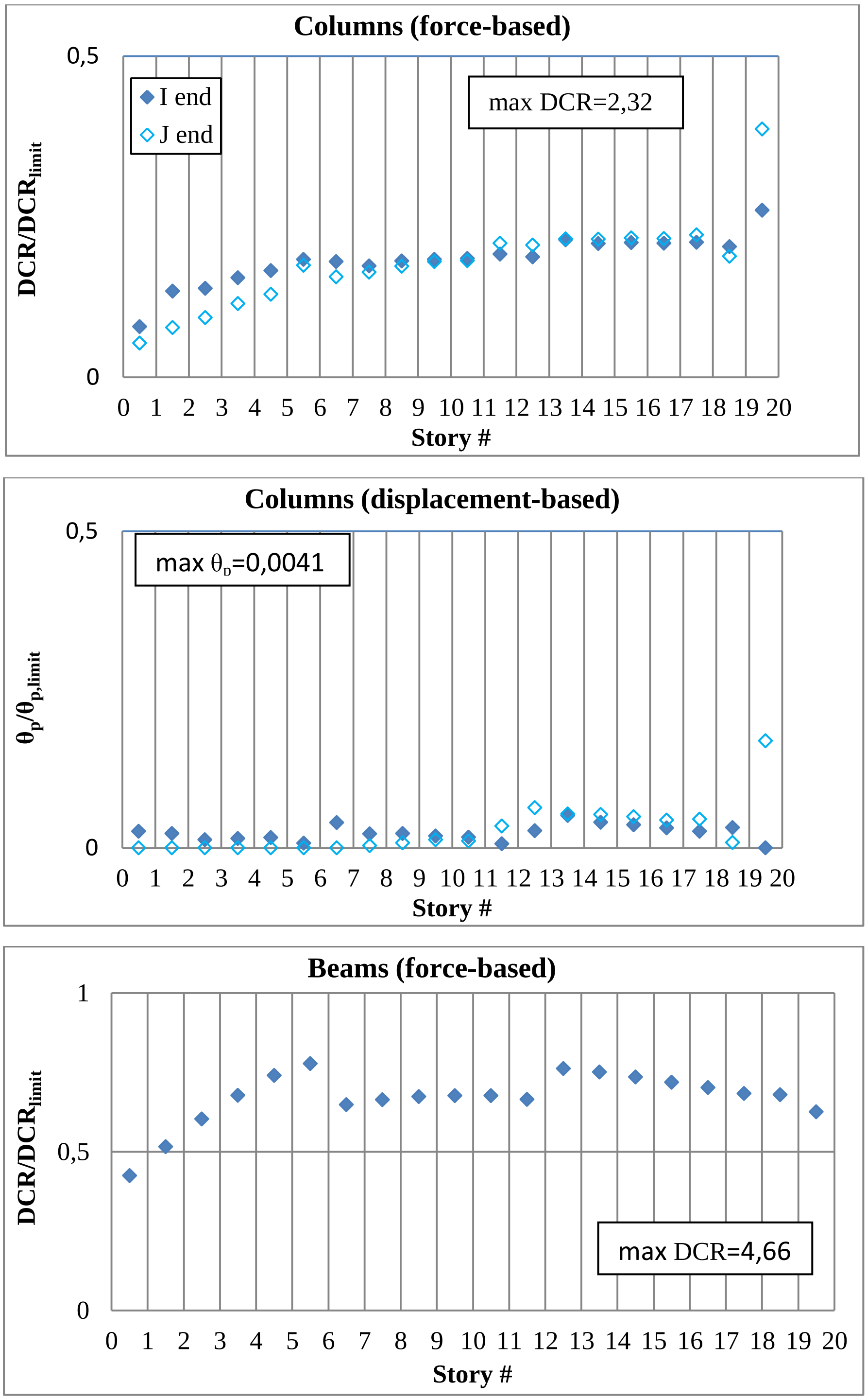


Figure 2.24: Comparative evaluation of performance parameters for the 20 Story R/C wall-frame

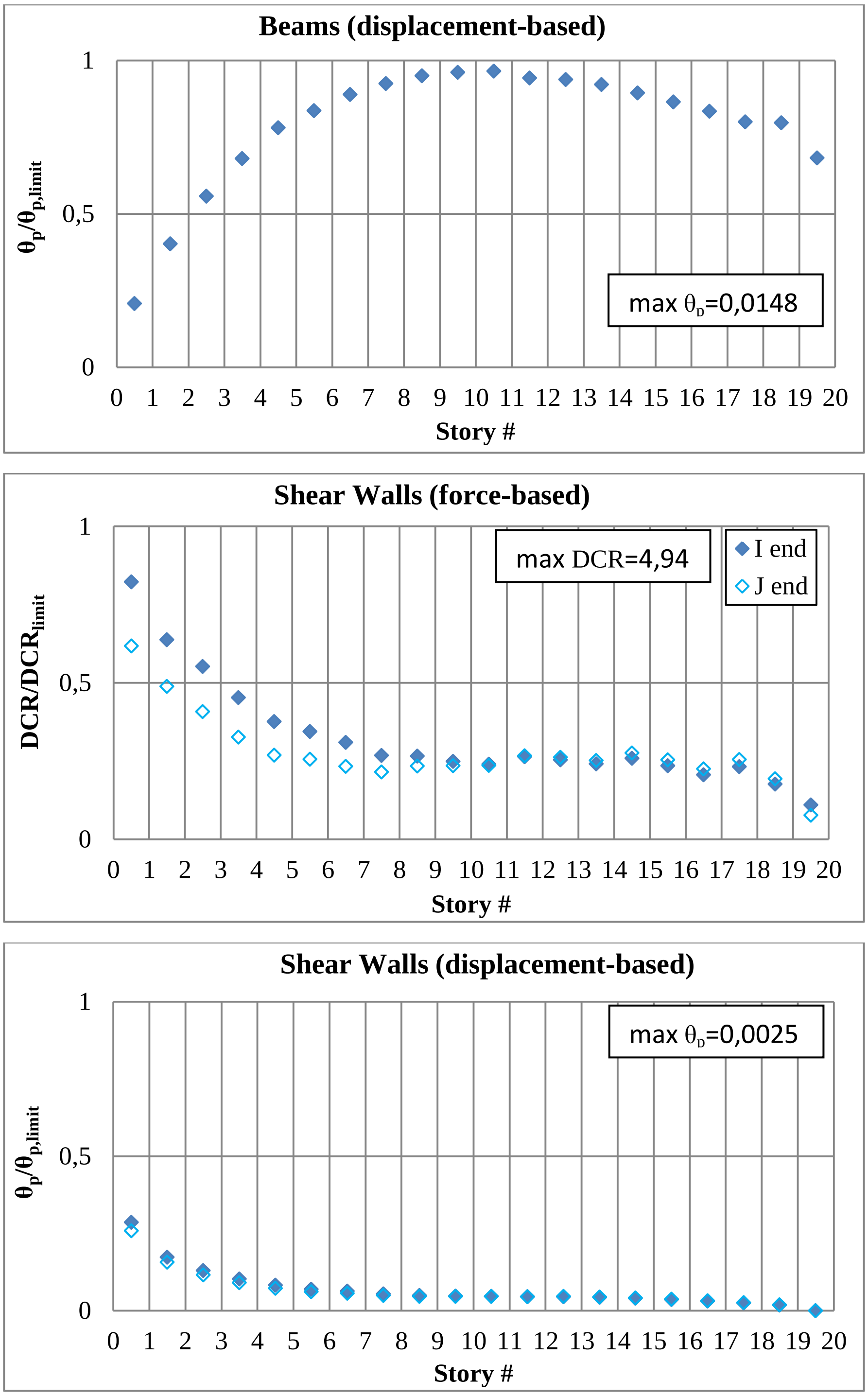


Figure 2.24 cont'd: Comparative evaluation of performance parameters for the 20 Story R/C wall-frame

Table 2.12: Maximum values of performance limits in TEC2007 and ASCE41 for beams, columns and shear walls

		IO	LS	CP
Beams	TEC2007 (*r* factors)	3	7	10
	ASCE41 (θ_p)	0,010	0,020	0,025
Columns	TEC2007 (*r* factors)	3	6	8
	ASCE41 (θ_p)	0,005	0,026	0,035
Shear Walls	TEC2007 (*r* factors)	3	6	8
	ASCE41 (θ_p)	0,005	0,010	0,015

Following results can be obtained from Figure 2.24 and Table 2.12.

- Demands on columns are very low due to the presence of a shear wall. This is expected in general. Both force based and displacement based assessment confirm this situation consistently.
- On the other hand, beam performances are closer to the LS limits because higher demands occur at beam ends, especially the ones connected to the shear wall.
- Beams are much closer to the LS limit when displacement-based approach is considered because larger plastic deformations are expected at the beam ends due to capacity design and strong column-weak beam criterion. Since these results are obtained from the same analysis method, the performance levels obtained should be close to each other. However, because of high performance limits allowed by TEC2007, displacement-based results are much critical for beams. Apparently, the limiting *r* factors for beams need some adjustment.
- There is a contrary situation for shear walls, where the force based procedure yields larger ratios at the lower stories. This is a problem related to defining the capacity moments of shear wall sections, which is discussed in detail for the following building.

2.3.4. 4 Story Retrofitted R/C Building (IO; R=4)

Both analysis procedures, namely response spectrum and pushover analyses are applicable to this structure. Therefore, both force-based and displacement-based results are obtained. The structure is analyzed in the more critical short direction.

The design of this structure satisfies the immediate occupancy performance limit with a force reduction factor R = 4. Since this is a retrofitted school building, this structure is expected to perform at the IO level. Calculated performance quantities, such as DCRs and plastic rotation demands at member ends are compared with the LS performance limits for both force-based and displacement-based procedures. For the force-based procedure, the ratios of calculated DCR to the limit DCR for IO performance limit are plotted for each story. For displacement-based procedure, the ratios of plastic rotation demands to the limit plastic rotations for IO performance limits are plotted for each story. For this structure, all elements are considered individually since the structure is not perfectly regular in plan. Results are presented considering each story separately, but all elements in those stories are shown. The results are presented in Figure 2.25. The maximum value of torsional irregularity constant η_{bi} according to TEC2007 is calculated as 1.04, which implies that the structure does not have notable torsional irregularity.

Maximum performance limit values for elements that have nonconforming transverse reinforcement are given in Table 2.13 according to TEC2007 and ASCE41. Maximum performance limit values for conforming transverse reinforcement (C) are added to Table 2.13 only for shear walls, because new shear walls added during retrofitting has adequate conformity of transverse reinforcement whereas beams and columns of the structure have remained the same, as non-conforming.

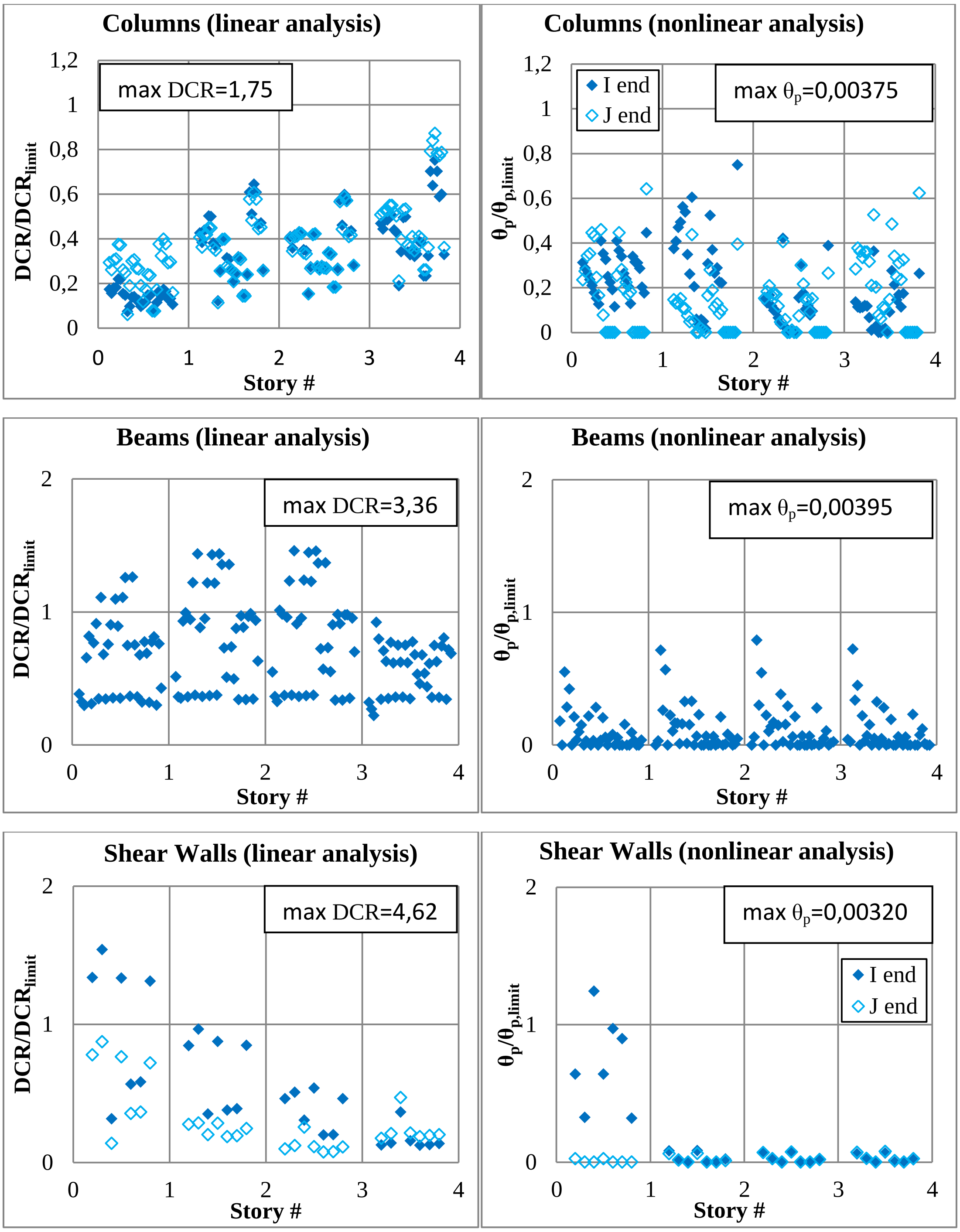


Figure 2.25: Comparative evaluation of performance parameters for the 4 Story R/C retrofitted building

Table 2.13: Maximum values of performance limits in TEC2007 and ASCE41 for retrofitted structure

		IO	LS	CP
Beams	TEC2007 (*r* factors-NC)	2,5	4	6
	ASCE41 (θ_p-NC)	0,005	0,010	0,020
Columns	TEC2007 (*r* factors-NC)	2	3,5	5
	ASCE41 (θ_p-NC)	0,005	0,009	0,010
Shear Walls	TEC2007 (*r* factors-NC)	2	4	6
	TEC2007 (*r* factors-C)	3	6	8
	ASCE41 (θ_p-NC)	0,002	0,004	0,008
	ASCE41 (θ_p-C)	0,005	0,010	0,015

The following conclusions can be reached from the results shown in Figure 2.25 and Table 2.13.

- For both force-based and displacement-based procedures, demands at column ends are low. This is expected because the structure has a number of massive shear walls. Therefore, the critical components are shear wall bases and beam ends that are connected to shear walls.
- Five percent of NC beams exceed the IO performance limits in force based procedure whereas none of those beam ends exceed the limits in displacement based procedure. The main reason behind it is the difference in the method of analysis, i.e. linear versus nonlinear. Linear methods cannot estimate the force distributions correctly in complex frame-wall structures. The forces on beams might be overestimated by the linear elastic analysis where redistribution after first yielding is not accounted for.
- For shear walls, both force-based and displacement-based assessment revealed that they do not satisfy the IO performance limits. For the displacement-based results, only one shear wall member fails to satisfy this limit. When checked, it is observed that the failing wall is not a new wall that is added during retrofitting procedure, but an existing one without confined boundaries. Therefore, the performance limits for that wall is smaller compared to the new walls. For the force-based results, there are four walls that exceeds IO limit. All four of them are new built walls during retrofitting, so the problem is not related to the small

performance limits, but high DCRs. The main reason of getting high DCR is choosing yield moment as the capacity moment. Figure 2.26 shows the moment-curvature relationship of a shear wall that has the highest DCR, which is 4,62. The yield moment and curvature values for the wall as well as the curvature equivalent of limit state plastic rotations are also marked on Figure 2.26. The yield moment of the shear wall is chosen as 10,270 kN.m for the most critical member, which is the first yielding point and the safest choice. However for shear walls, it is not possible to decide on an exact yield point, which can also be seen in Figure 2.26. Yield moment for this shear wall can vary between the first yield moment of 10270 kN.m and the capacity moment 17,050 kN.m which is a wide range. If 17050 kN.m is chosen as the capacity moment for this shear wall, then this member will satisfy the IO performance limit. Since the minimum and safest yield moment is used during linear elastic analysis and DCR calculations, some shear walls did not satisfy the performance limit, but this does not mean that the structure is not safe. This means that there is an uncertainty in deciding on the capacity moments of shear wall members.

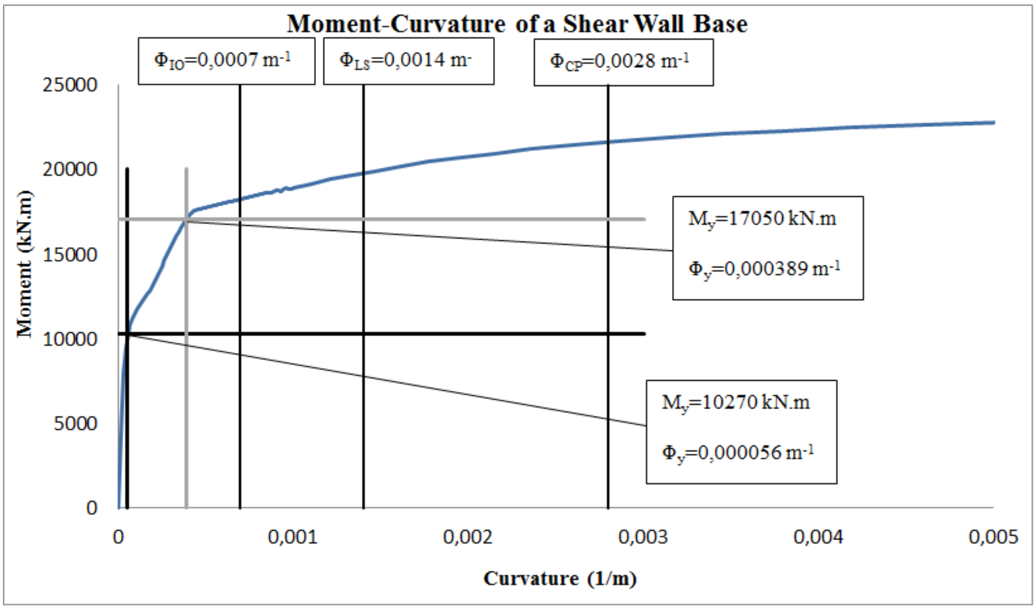


Figure 2.26: Moment-curvature relationship of the most critical shear wall base of the retrofitted building

2.4. Proposed Modifications on Performance Limits

Performance evaluation results obtained from the four code conforming buildings by using both force based (linear elastic analysis) and displacement based (linear elastic and nonlinear analysis) revealed that the performance limits employed in the force based procedure are more generous than those employed in the displacement based procedures. In fact, displacement based performance limits (plastic rotations) are more reliable since they can be, and are verified by test results whereas force based limits (r factors) are totally judgmental, and cannot be verified by physical testing. Therefore the force based performance limits needs some adjustment to lower values in order to provide consistency with the displacement based performance limits.

The suggested values are marked on the TEC 2007 tables below, along with the *m* factors given in ASCE 41 for the force based linear elastic procedure.

Table 2.14: Proposed demand to capacity ratios that define performance limits for R/C beams of TEC2007 (r_s)

Ductile Beams			Performance Limit		
$\frac{\rho-\rho'}{\rho_b}$	Confined Boundary	$\frac{V_e}{b_w d\ f_{ctm}}$	IO	LS	CP
≤ 0.0	Yes	≤ 0.65	3 (3)	~~7~~ 6 (6)	~~10~~ 8 (7)
≤ 0.0	Yes	≥ 1.30	~~2.5~~ 2 (2)	~~5~~ 4 (3)	~~8~~ 6 (4)
≥ 0.5	Yes	≤ 0.65	~~3~~ 2 (2)	~~5~~ 4 (3)	~~7~~ 6 (4)
≥ 0.5	Yes	≥ 1.30	~~2.5~~ 2 (2)	~~4~~ 3.5 (2)	5 (3)
≤ 0.0	No	≤ 0.65	~~2.5~~ 2 (2)	~~4~~ 3.5 (3)	~~6~~ 5 (4)
≤ 0.0	No	≥ 1.30	~~2~~ 1.5 (1.25)	3 (2)	~~5~~ 4 (3)
≥ 0.5	No	≤ 0.65	2 (2)	3 (3)	~~5~~ 4 (3)
≥ 0.5	No	≥ 1.30	1.5 (1.25)	2.5 (2)	~~4~~ 3 (2)

~~3~~: Existing limit; 3: No change; 2: Suggested limit; (2) ASCE 41 *m* factor

Table 2.15: Proposed demand to capacity ratios that define performance limits for R/C columns of TEC2007 (r_s)

Ductile Columns			Performance Limit		
$\frac{N_K}{A_c f_{cm}}$	Confined Boundary	$\frac{V_e}{b_w d f_{ctm}}$	**IO**	**LS**	**CP**
≤ 0.1	Yes	≤ 0.65	~~3~~ 2 (2)	~~6~~ 5 (3)	~~8~~ 7 (4)
≤ 0.1	Yes	≥ 1.30	~~2.5~~ 2 (2)	~~5~~ 4 (2.4)	~~6~~ 5 (3.2)
≥ 0.4 ve ≤ 0.7	Yes	≤ 0.65	~~2~~ 1.5 (1.25)	~~4~~ 3 (2)	~~6~~ 5 (3)
≥ 0.4 ve ≤ 0.7	Yes	≥ 1.30	1.5 (1.25)	2.5 (1.6)	3.5 (2.4)
≤ 0.1	No	≤ 0.65	2 (2)	~~3.5~~ 3 (2)	~~5~~ 4.5 (3)
≤ 0.1	No	≥ 1.30	1.5 (2)	2.5 (2)	~~3.5~~ 3 (2.4)
≥ 0.4 ve ≤ 0.7	No	≤ 0.65	1.5 (1.25)	2 (1.5)	3 (2)
≥ 0.4 ve ≤ 0.7	No	≥ 1.30	1 (1.25)	1.5 (1.5)	2 (1.75)
≥ 0.7	–	–	1	1	1

~~3~~: Existing limit; 3: No change; 2: Suggested limit; (2) ASCE 41 *m* factor

Table 2.15: Proposed demand to capacity ratios that define performance limits for R/C shear walls of TEC2007 (r_s)

Ductile Shear Walls	Performance Limit		
Confined Boundary	**IO**	**LS**	**CP**
Yes	~~3~~ 2.5 (2)	~~6~~ 5 (4)	~~8~~ 7 (6)
No	2 (2)	~~4~~ 3 (2.5)	~~6~~ 5 (4)

~~3~~: Existing limit; 3: No change; 2: Suggested limit; (2) ASCE 41 *m* factor

CHAPTER 3

LIMITATIONS ON LINEAR ELASTIC PROCEDURES FOR SEISMIC ASSESSMENT

Force and deformation demands calculated by using linear elastic procedures deviate from those calculated by inelastic procedures as the level of inelastic deformations increase. But more importantly, as inelastic deformations localize, it leads to entirely different deformation patterns in the linear elastic and inelastic structures. The objective of this chapter is to determine, as much as possible, the limitations that should be imposed on linear elastic procedures. The methodology followed is based on conducting response history analysis on several structures with different irregularities by using both linear elastic and nonlinear models in parallel. Several ground motions with different intensities are employed. Plastic rotations at member ends are the response parameters used for comparison. Plastic rotations for linear elastic models are determined from chord rotations, as described in the previous Chapter.

3.1. Ground Motions Employed in Case Studies

Three ground motions are selected from the PEER strong motion database to conduct linear elastic and nonlinear response history analysis in this chapter. Acceleration time histories of these ground motions are given in Figure 3.1. PGA, PGV and PGD of the selected ground motions are shown in Table 3.1.

Selected ground motions are further scaled up by the ratios of 1.5 and 2. This procedure is adopted to observe the effects of stronger intensity ground motions which create higher nonlinearities in the structures during the analysis. Pseudo acceleration response spectra of nine ground motions along with the TEC2007 design spectrum are shown in Figure 3.2.

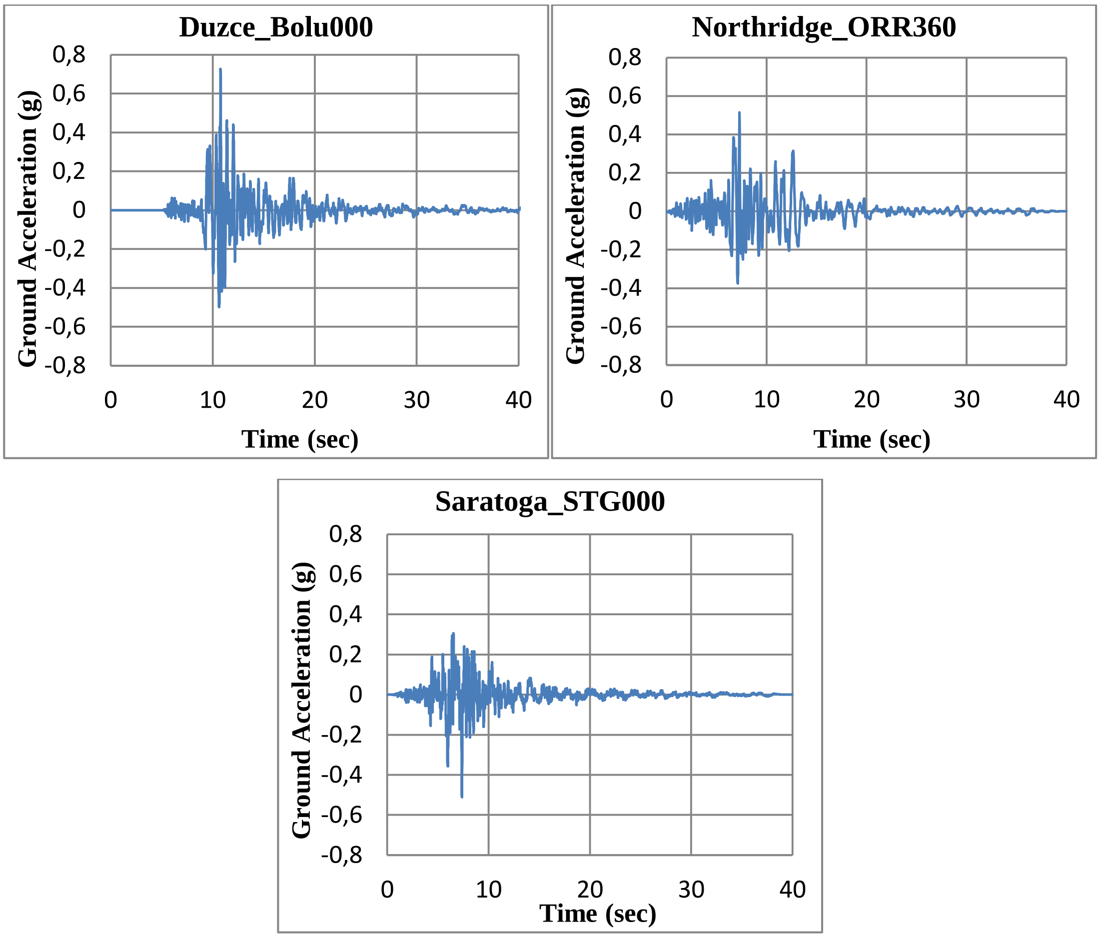


Figure 3.1: Acceleration time histories of selected ground motions

Table 3.1: Properties of selected ground motions

Station	PEER Code	Earthquake	M_w	PGA(g)	Site Geology
Bolu	BOL000	Düzce, 1999	7,1	0,73	D
Castaic - Old Ridge Route	ORR360	Northridge, 1994	6,7	0,51	B
Saratoga – Aloha Ave	STG000	Loma Prieta, 1989	6,9	0,51	D

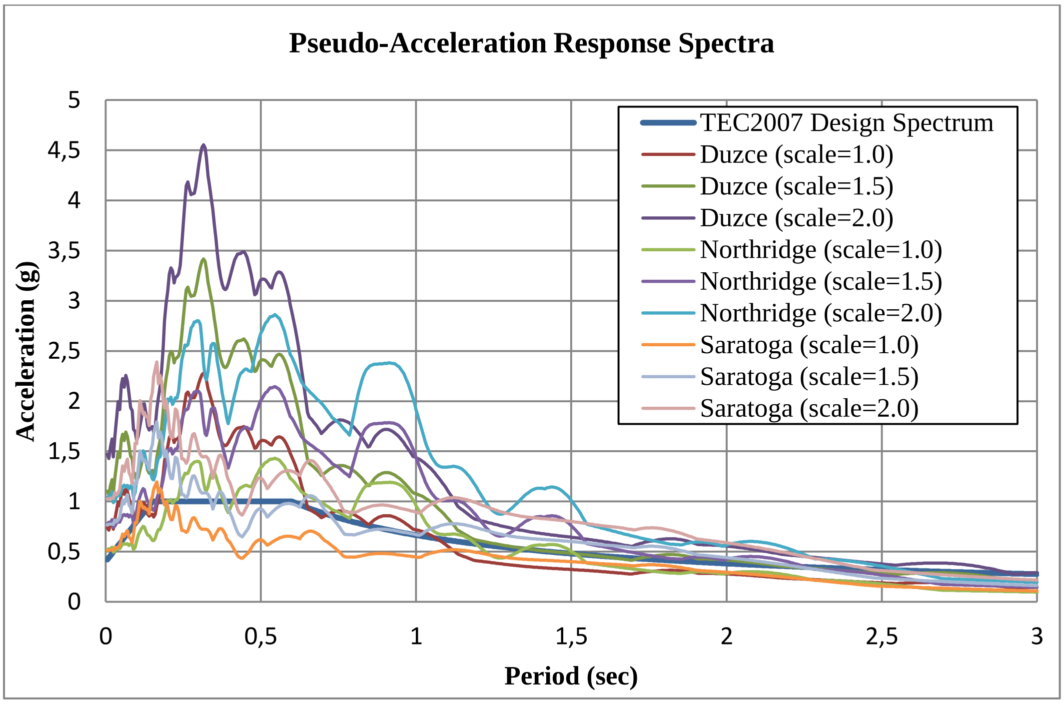


Figure 3.2: Response spectra of ground motions and TEC2007 design spectrum

3.2. Case Study Buildings

Building structures presented in Chapter 2 are re analyzed in this chapter. In addition to these buildings, five new structures are modeled for this chapter, which makes a total of nine buildings to be investigated. These additional structures are a twelve story R/C frame with relaxed capacity design, a twelve story R/C wall-frame, an eight story 3D R/C frame, a five story R/C plane frame with appropriate column capacity for strong column-weak beam principle, and a similar five story R/C frame with reduced column capacity.

3.2.1. Twelve Story R/C Frame with Relaxed Capacity Design

The first additional building is a twelve story symmetrical-plan R/C structure. The story plan and 2D elevation view of two adjacent frames are given in Figures 3.3 and 3.4, respectively. The structure has the same properties with the one defined in Section 2.2.2 except the longitudinal reinforcement of the columns. In order to account for the effects of non-seismic design, column capacities of the twelve story frame with full capacity design are reduced by 25%. By doing that, the rule of column end capacities should be at least 20% more than the beam end capacities at any joint is violated for some of the joints. Member dimensions, beam cross-sections and shear reinforcements of the members are the same as the twelve story with full capacity design building. The cross-sections of beams and columns are shown in Figure 3.5. Information on shear reinforcement used in beams and columns is provided in Table 3.2. All of the stories are 3.2 meter high except the first story, which is 4 meters high. The total height of the structure is 39.2 meters. The structure is in seismic zone 1 on Z3 type soil according to TEC2007. Characteristic strengths of concrete and steel are 25 MPa and 420 MPa, respectively.

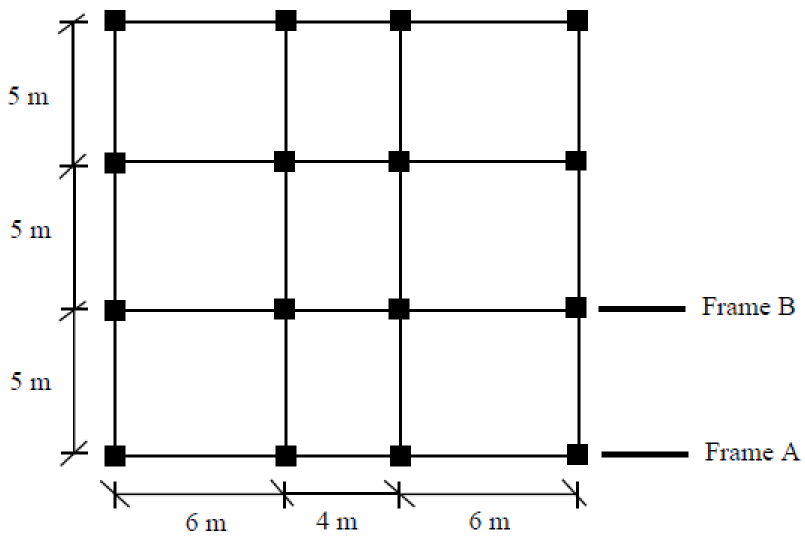


Figure 3.3: Story plan of the twelve story building with relaxed capacity design

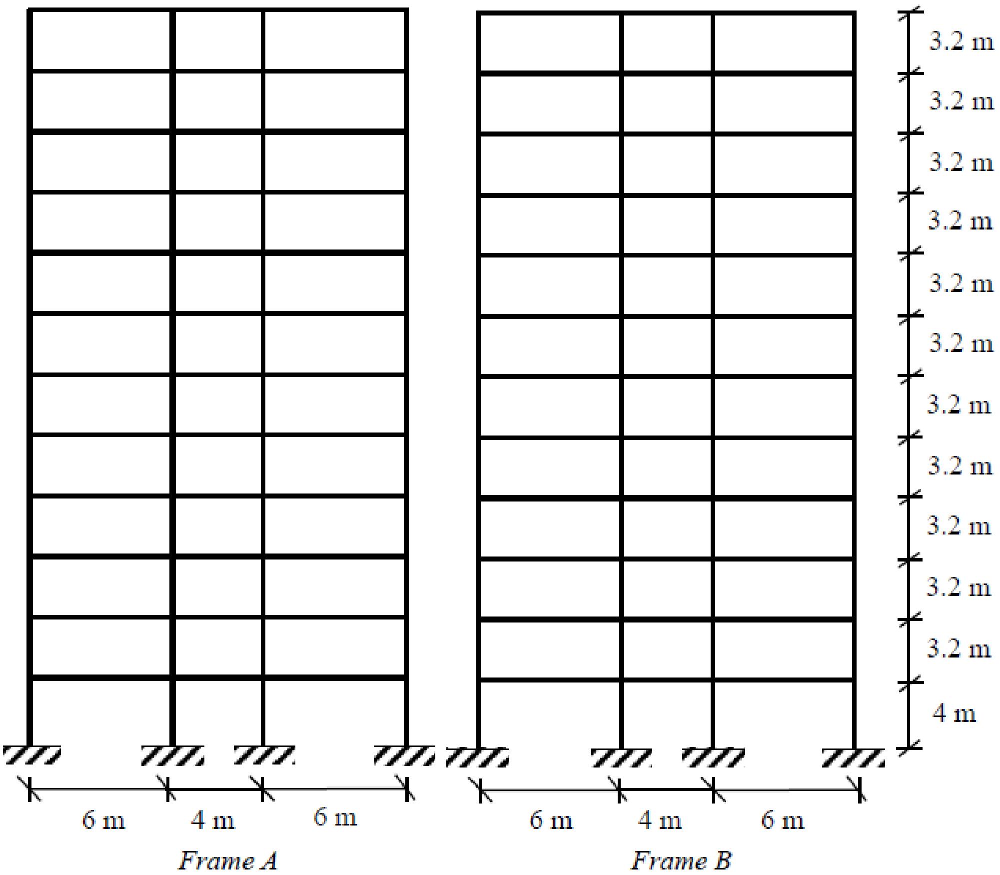
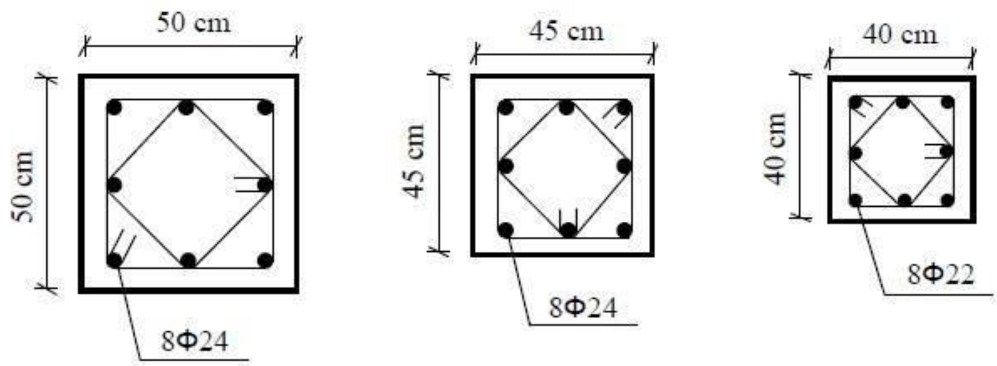
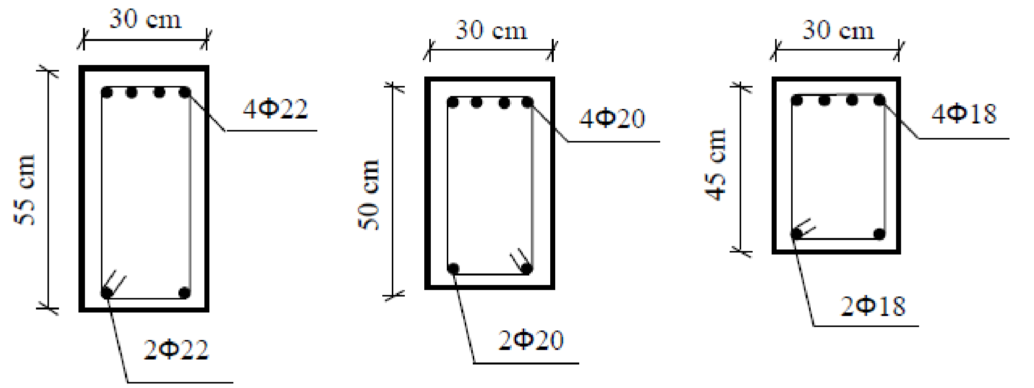


Figure 3.4: Elevation view of frames A and B of the twelve story building with relaxed capacity design

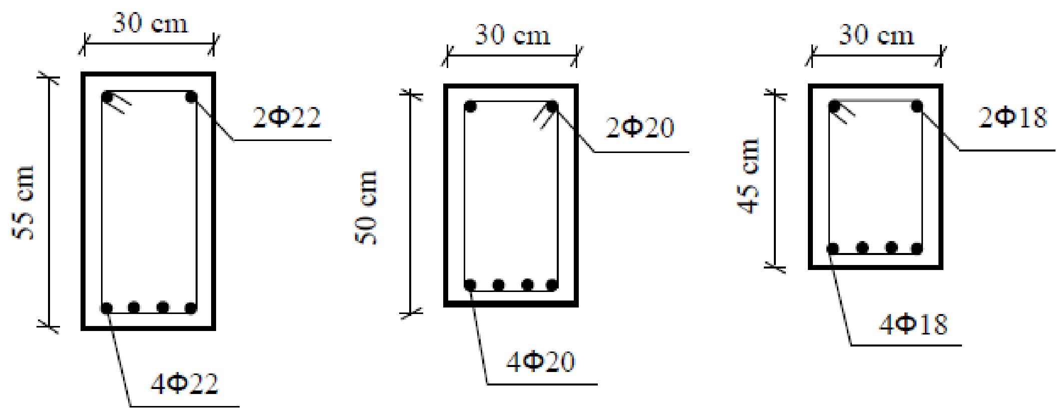


a) Column cross-sections of the building

Figure 3.5: Cross-section details of columns and beams of twelve story building with relaxed capacity design



b) Support region of beam cross-sections of the building



c) Span region of beam cross-sections of the building

Figure 3.5 cont'd: Cross-section details of columns and beams of twelve story building with relaxed capacity design

Table 3.2: Shear design details of elements of twelve story building with relaxed capacity design

		Along End Region	**Along Span Region**
Columns	50x50 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	45x45 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	40x40 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
Beams	55x30 cm2	ϕ8 / 12 cm	ϕ8 / 18 cm
	50x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	45x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm

2D analytical model of this frame is prepared by using the OpenSees software (version 2.4.3). This software is used for both linear and nonlinear response history analysis. For linear analysis, elastic beam and column elements are defined by using the cracked stiffness values, which are obtained by multiplying the gross moment of inertia by 0.4 and 0.5 for beams and columns, respectively. The cracked stiffnesses are reduced further in order to consider capacity reduction of columns. For nonlinear analysis, structural elements are modeled by using "beam with hinges" definition of OpenSees. Plastic hinge lengths for all structural elements are calculated as half of the cross-section depth. For beams, bi-linear force deformation behavior is used. In order to determine the yield and ultimate points of bi-linear behavior, Response2000 software is used. For columns, fiber sections are used along the plastic hinge length. Confined and unconfined concrete are defined separately with the properties of specified reinforcement. The remaining parts of the elements, which contain the zone between plastic hinge regions at the element ends, are defined as elastic beam and column sections. The same cracked section properties given above are also used for this part. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the model.

Free vibration properties of the twelve story R/C frame with relaxed capacity design are the same as the building defined in Section 2.2.2.

3.2.2. Twelve Story R/C Wall-Frame

The second additional case study building is a twelve story symmetrical-plan R/C structure with shear walls placed along the earthquake direction. The story plan and 2D view of two adjacent frames are shown in Figures 3.6 and 3.7, respectively. This structure is also very similar to the twelve story R/C frame with full capacity design. Beam and column properties are the same in both structures, but shear walls are added into the two interior frames of the building in order to observe the comparative effects of shear walls. The cross-sections of beams and columns are shown in Figure 3.8. The cross-section properties of the shear walls are given in Figure 3.9. Information on shear reinforcement used in beams and columns is provided in Table 3.3. All of the stories are 3.2 meter high, except the first story, which is 4 meters high. The total height of the structure is 39.2 meters. Characteristic strengths of concrete and steel are 25 MPa and 420 MPa, respectively.

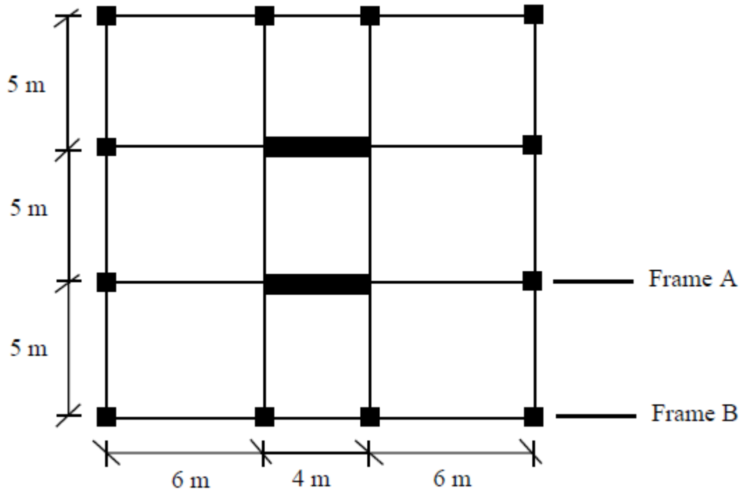


Figure 3.6: Story plan of the twelve story R/C wall-frame building

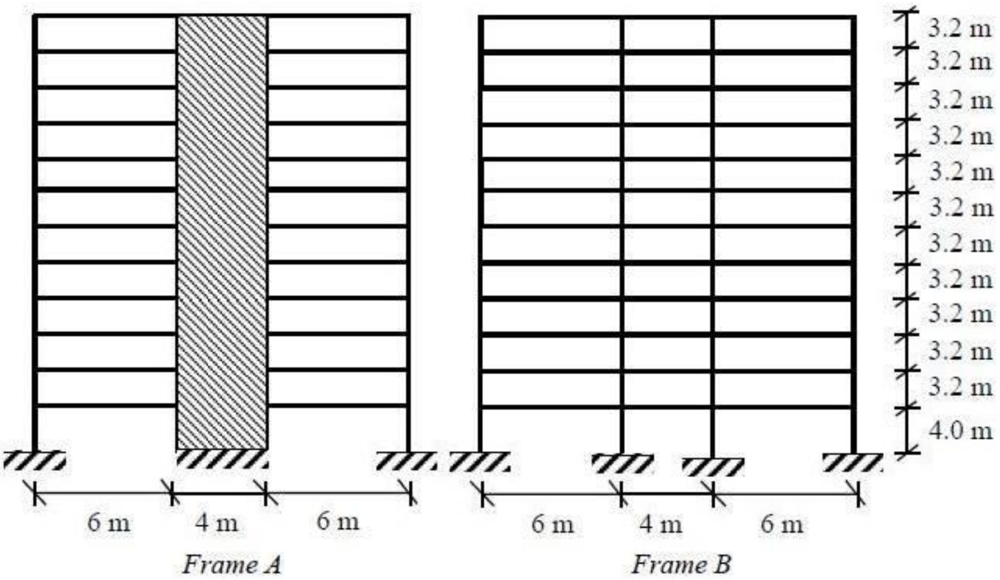
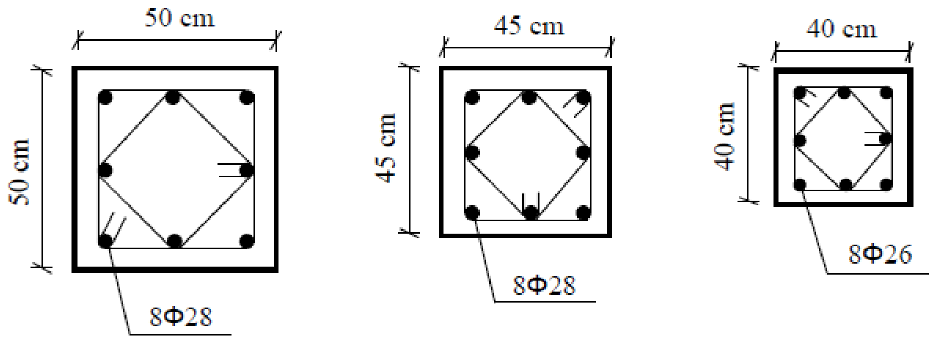
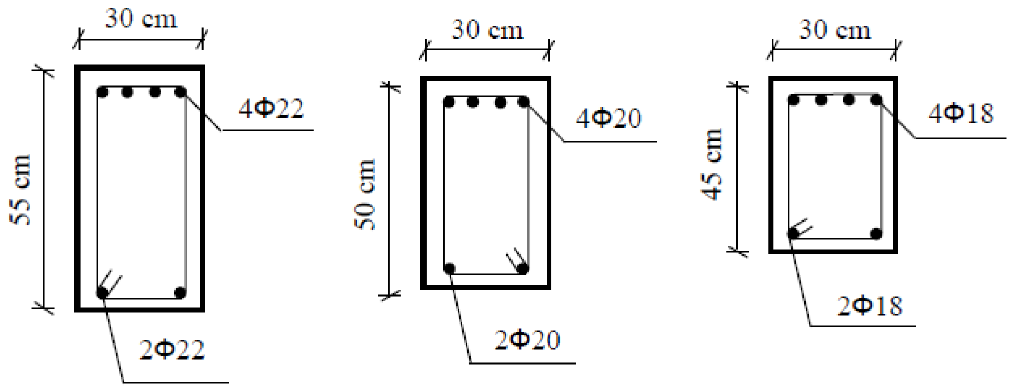


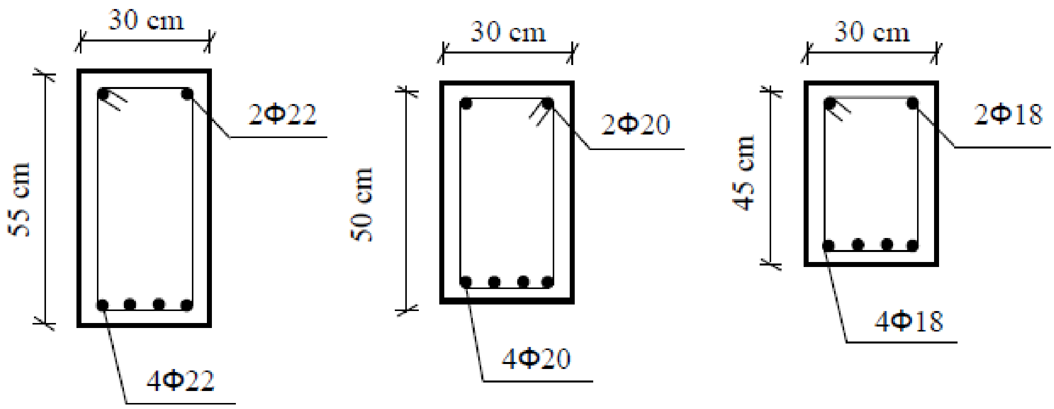
Figure 3.7: 2D model view of frames A and B of the twelve story R/C wall-frame building



a) Column cross-sections of the building

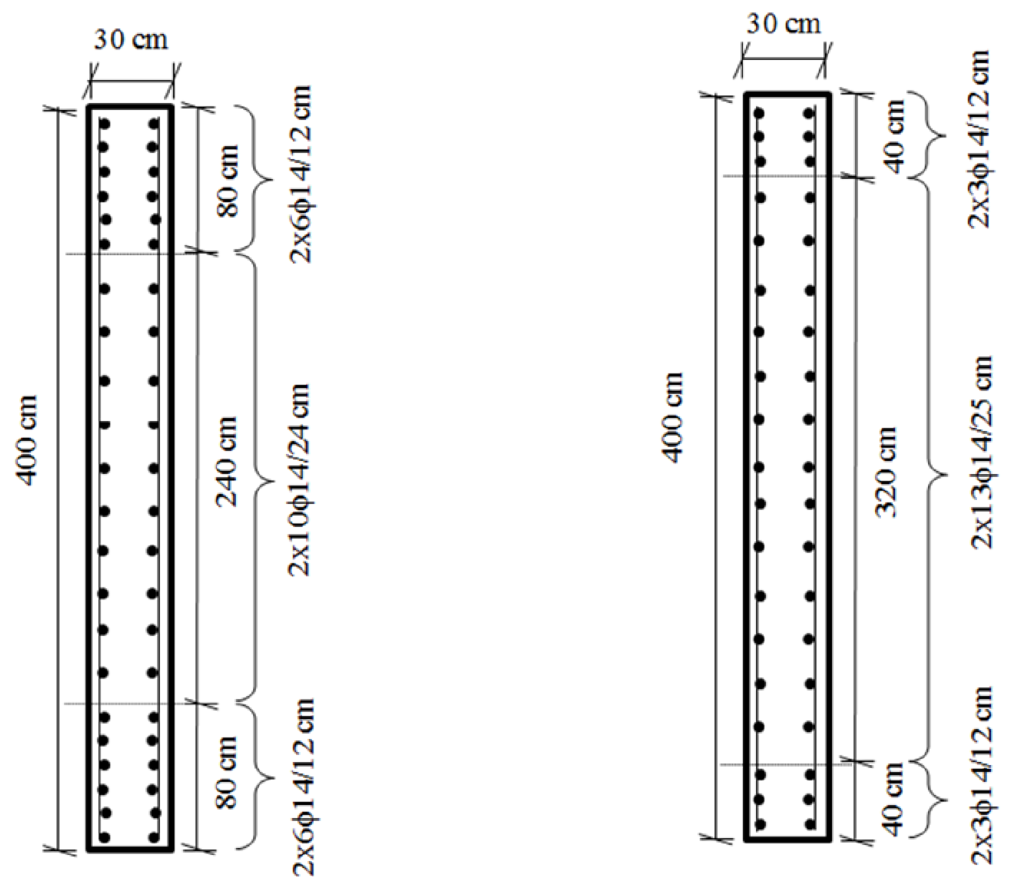


b) Support region of beam cross-sections of the building



c) Span region of beam cross-sections of the building

Figure 3.8: Cross-section details of columns and beams of twelve story wall-frame building



a) For $H<H_{cr}$ (H_{cr}=4 m) b) For $H>H_{cr}$

Figure 3.9: Cross-section details of shear walls of twelve story wall-frame building

Table 3.3: Shear design details of elements of twelve story wall-frame building

		Along End Region	Along Span Region
Columns	50x50 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	45x45 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	40x40 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
Beams	55x30 cm2	ϕ8 / 12 cm	ϕ8 / 18 cm
	50x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm
	45x30 cm2	ϕ8 / 10 cm	ϕ8 / 15 cm

2D analytical model of the twelve story wall-frame is prepared by using the OpenSees software (version 2.4.3). This software is used for both linear and nonlinear analysis. For linear analysis, elastic beam, column and shear wall elements are defined using the cracked stiffness values. For nonlinear analysis, structural elements are modeled using beam with hinges definition of OpenSees. Plastic hinge lengths for beams and columns are calculated as the half of the cross-section depth. For shear wall, an additional node is defined at the mid-height of first story to account for the plastic rotations that are expected to occur along the critical height, H_{cr}. Plastic hinges are defined along nodes within H_{cr} whereas for the rest of the structure, plastic hinges are defined for very small distances since plasticity of shear wall is expected to concentrate along the critical length. For beams, bi-linear force deformation behavior is used. In order to determine the yield and ultimate points of bi-linear behavior, Response2000 software is used. The moment-curvature results obtained from Response2000 are idealized as bi-linear curves and used as an input to OpenSees model. For columns, fiber sections are used along the plastic hinge length. For shear wall members, bi-linear moment-curvature relationship with shear aggregator is defined along the plastic hinge length. Confined and unconfined concrete are defined separately with the properties of specified reinforcement. The remaining parts of the elements, which contain the zone between plastic hinge regions at the element ends, are defined as elastic beam and column sections. The same cracked section properties given above are also used for this part. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the model.

Free vibration properties of the twelve story R/C wall-frame are calculated by eigenvalue analysis of the linear elastic model using the cracked stiffness values. Modal information regarding the first four modes is tabulated in Table 3.4 and the modes shapes of the first four modes are shown in Figure 3.10.

Table 3.4: Free vibration properties of the first four modes of twelve story R/C wall-frame

Mode #	T (sec)	Effective Modal Mass (tons)	Effective Modal Mass Ratio
1	1,31	366,45	0,67
2	0,28	100,25	0,18
3	0,11	37,81	0,07
4	0,06	18,85	0,03

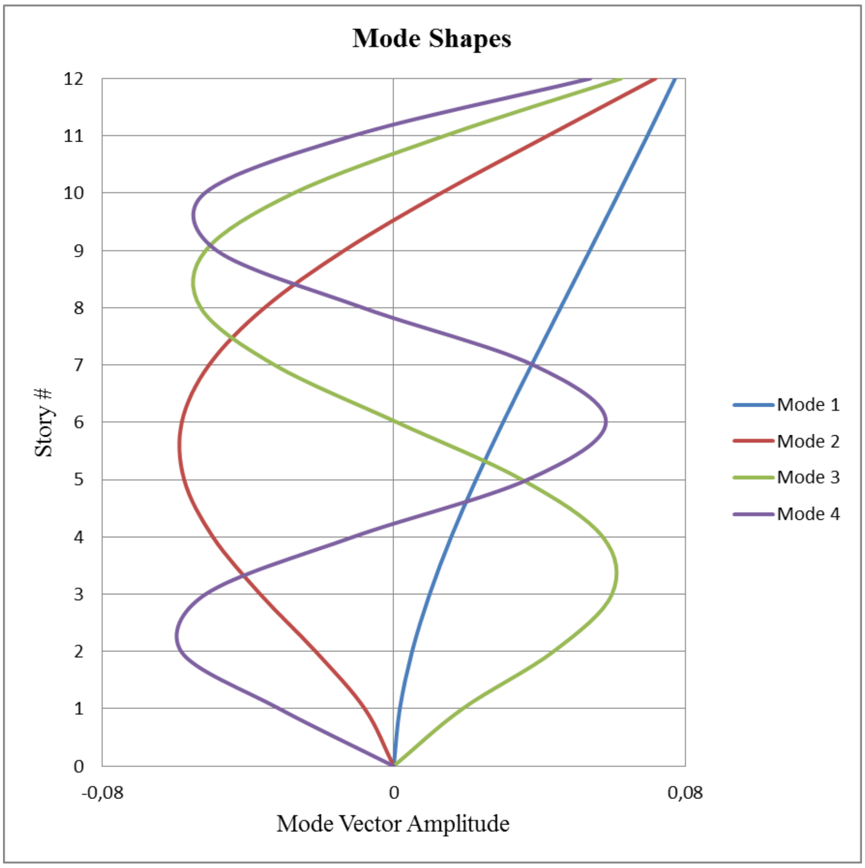


Figure 3.10: Mode shapes of the first four modes of twelve story R/C wall-frame

3.2.3. Eight Story 3D R/C Frame

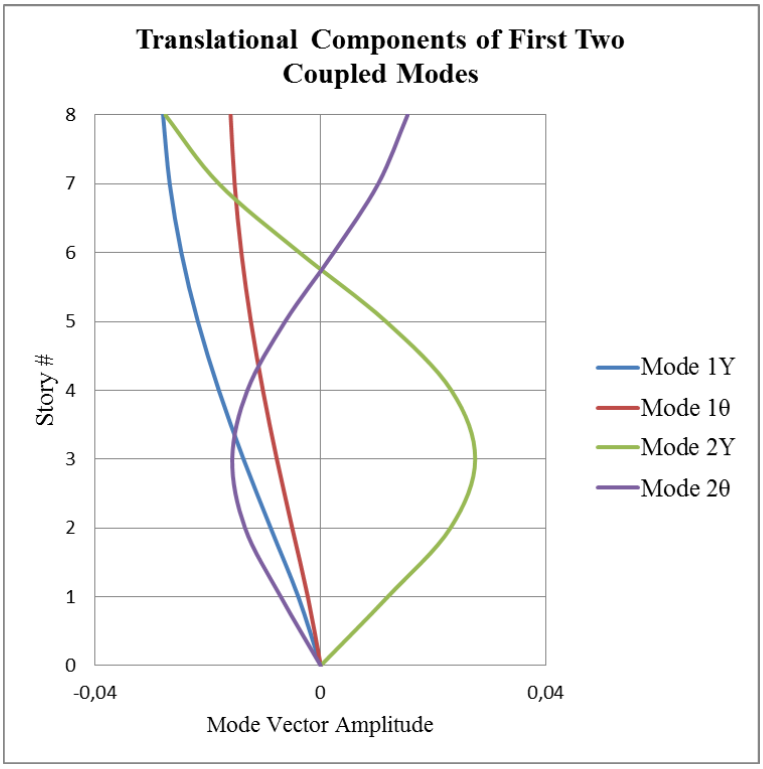
The third additional case study building to be analyzed is an eight story 3D R/C frame with unsymmetrical-plan. This structure is the same as the six story 3D R/C frame with full capacity design, except its story number. Without changing any property, two stories are added to the six story building (Kaatsız, 2012). In the added two stories, beams and columns have the same properties with the beams and columns of the fifth and sixth stories. The information on general layout and cross-sectional properties of the elements is given in Section 2.2.1. Considering the fact that six story building is a full capacity design structure, adding two stories without any extra design considerations is expected to create a deficiency in the building. The effect of this deficiency on the response is investigated.

3D analytical model of the eight story building is prepared by using the OpenSees software (version 2.4.3). This software is used for both linear and nonlinear analysis. For linear analysis, elastic beam and column elements are defined using the cracked stiffness values, which are obtained by multiplying the gross moment of inertia with 0.4 and 0.6 for beams and columns, respectively. For nonlinear analysis, structural elements are modeled by using beam with hinges definition of OpenSees. Plastic hinge lengths for all structural elements are calculated as the half of the cross-section depth. For beams, bi-linear force deformation behavior is used. In order to determine the yield and ultimate points of bi-linear behavior, Response2000 software is used. The moment-curvature results obtained from Response2000 are idealized as bi-linear curves and used as an input to OpenSees model. For columns, fiber sections are used along the plastic hinge length. The remaining parts of the elements, which contain the zone between plastic hinge regions at the element ends, are defined as elastic beam and column sections. The same cracked section properties given above are also used for this part. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the model.

Free vibration properties of the eight story 3D R/C frame are calculated by eigenvalue analysis of the linear elastic model by using the cracked stiffness values. Modal information regarding the first three coupled modes is tabulated in Table 3.5 and the translational and rotational mode shapes of the first two coupled modes are shown in in Figure 3.11.

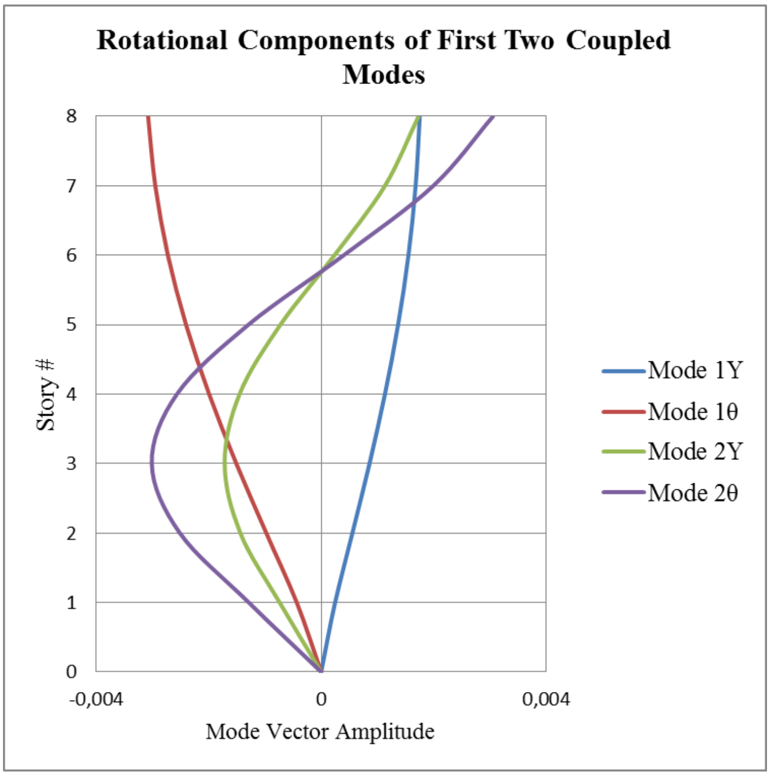
Table 3.5: Free vibration properties of the first three coupled modes of eight story R/C frame

Mode #	T (sec)	Effective Modal Mass (tons)	Effective Modal Mass Ratio
1Y	1,74	1196,39	0,63
1θ	1,05	387,68	0,20
2Y	0,56	142,11	0,07
2θ	0,34	47,21	0,02
3Y	0,31	46,48	0,02
3θ	0,21	26,15	0,01



a

Figure 3.11: a. Translational, and b. Rotational mode shapes of the first two coupled modes of the eight story R/C building



b

Figure 3.11 con'd: a. Translational, and b. Rotational mode shapes of the first two coupled modes of the eight story R/C building

3.2.4. Five Story R/C Frame

The fourth additional case study building is a five story symmetrical-plan R/C structure. The story plan and 2D view of two adjacent frames are given in Figures 3.12 and 3.13, respectively. The column dimensions are 50x40 cm^2 and beam dimensions are 50x30 cm^2 for the entire structure. The cross-sections of beams and columns with the longitudinal reinforcement information and shear reinforcement detailing are shown in Figure 3.14. Information on shear reinforcement used in beams and columns is provided in Table 3.6. All stories are 3 meter high. The total height of the structure is 15 meters. There is no basement level, she structure starts from the ground level. The structure is located in seismic zone 1 on Z3 type soil according to TEC2007. Characteristic strengths of concrete and steel are 25 MPa and 420 MPa, respectively.

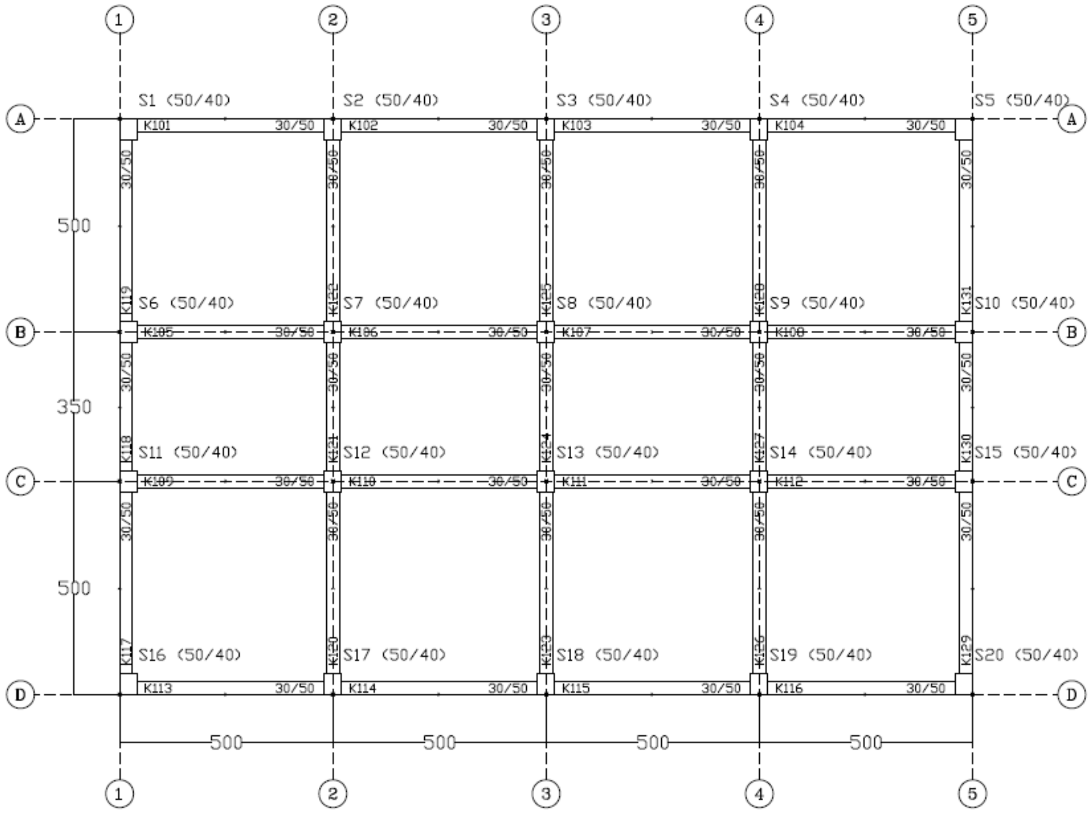


Figure 3.12: Story plan of five story building

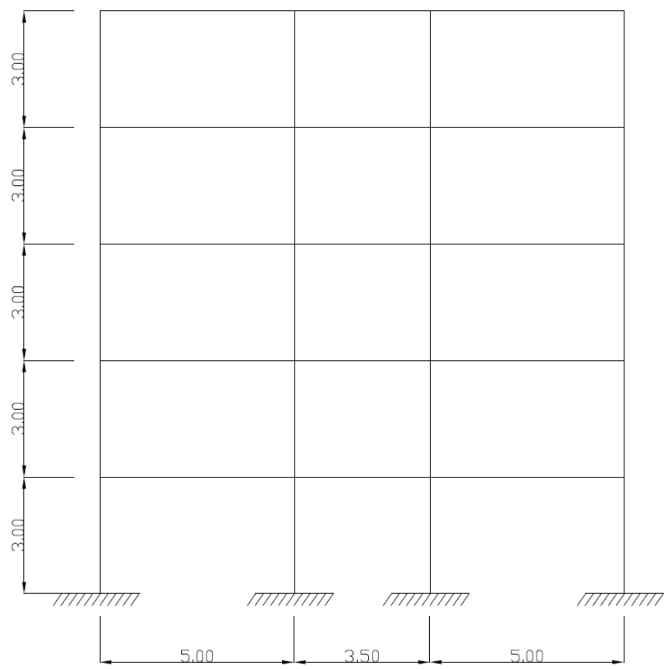
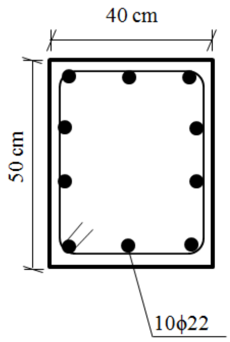
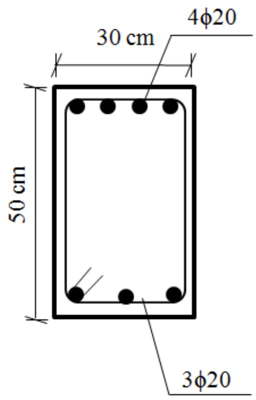


Figure 3.13: 2D elevation view of one frame of five story building



a) Column cross-sections of the building

b)



c) Beam cross-sections of the building

Figure 3.14: Cross-section details of columns and beams of five story building

Table 3.6: Shear design details of elements of five story frame

		Along End Region	Along Span Region
Columns	50x50 cm2	ϕ8 / 19 cm	ϕ8 / 19 cm
Beams	55x30 cm2	ϕ8 / 19 cm	ϕ8 / 19 cm

2D analytical model of the five story frame is prepared by using the OpenSees software (version 2.4.3). This model is used for both linear and nonlinear analysis. For linear analysis, elastic beam and column elements are defined by using the cracked stiffness values, which are obtained by multiplying the gross moment of inertia by 0.4 and 0.6 for beams and columns, respectively. For nonlinear analysis, structural elements are modeled by using “nonlinear beam and column” definition of OpenSees. For all elements, fiber sections are used along the member. Since it is the simple model, defining fiber sections throughout the entire length of elements did not create convergence problems. Confined and unconfined concrete are defined separately with the properties of specified reinforcement. Rigid diaphragms are assigned to each story and P-Δ effects are considered in the model.

Free vibration properties of the five story R/C frame are calculated by eigenvalue analysis of the linear elastic model with the cracked stiffness values. Modal information regarding the first three modes is tabulated in Table 3.7 and the modes shapes of the first four modes are shown in Figure 3.15.

Table 3.7: Free vibration properties of the first three modes of five story R/C frame

Mode #	T (sec)	Effective Modal Mass (tons)	Effective Modal Mass Ratio
1	0,92	210,22	0,82
2	0,28	28,13	0,11
3	0,15	11,60	0,05

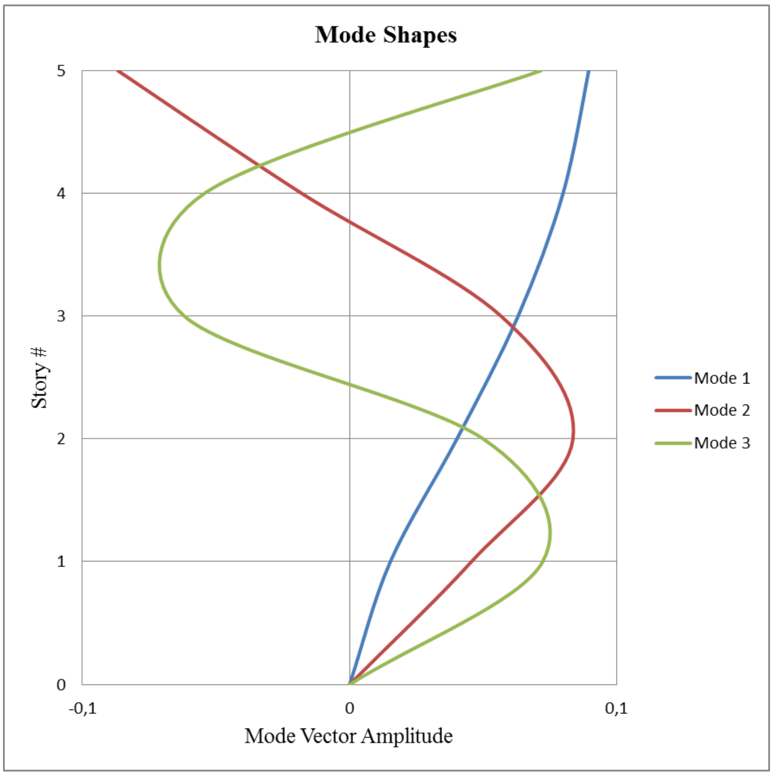


Figure 3.15: Mode shapes of the first three modes of five story R/C frame

3.2.5. Five Story R/C Frame with Reduced Column Capacity

The last additional case study building is a five story symmetrical-plan R/C structure with a reduced column capacity. The structure is identical to the previous one. The only difference is that flexural capacities of columns are reduced by 25%. By doing that, capacity ratios of column ends to beam ends at intersecting joints are reduced below 1, which is a significant deficiency. Plan, modeling considerations and free vibration properties of this structure is identical to the previous one.

3.3. Analysis Procedures Employed

Three different analysis procedures are employed in this chapter. The first one is the response spectrum analysis (RSA), which is a linear elastic procedure. Response spectrum analysis is implemented by using the response spectra of the selected and scaled ground motions rather than the design spectrum of TEC2007, which was the case in Chapter 2. Contributions of all modes are considered in the analysis in order to get the most accurate estimation. The second analysis procedure employed is the nonlinear response history analysis (NRHA). NRHA is the most advanced and the best analysis tool to observe the behavior of structures under ground excitation. Time history of ground motion is applied to the base nodes of the nonlinear model of the structure and the response of the structure at each time step is calculated. The third analysis procedure, namely conventional pushover analysis, is employed for simpler nonlinear models. Different from Chapter 2, in this chapter, target displacement demand at the top of the structure is determined by using the response spectra of selected ground motions for the fundamental period of the structure. Pushover analysis is employed only for three buildings. They are the four story retrofitted school building, five story 2D R/C frame and five story 2D R/C frame with reduced column capacity. Retrofitted building has many shear walls and a complex structure, hence there are convergence problems in NRHA. For the five story structures, ground motions are not used. Instead, design spectrum with 475 year return period in TEC2007 is used in both pushover and response spectrum analysis.

To observe the effects of earthquake intensity, design spectrum is scaled with 0.25, 0.50, 0.75 and 1.00 for comparative analysis.

From linear elastic analysis, DCRs and plastic rotation demands at the member ends are obtained from the chord rotations. Plastic rotation demands are also obtained from nonlinear analysis. The main goal in this chapter is the comparison between linear elastic and nonlinear analyses. DCRs are used to estimate the behavior of structures in the post-yielding zone by using linear elastic analysis.

In order to simplify the investigation of results and setting up a parameter for spectral intensity, the SI values of each case study for each ground motion data are calculated. In order to calculate SI, firstly the force reduction factors R for the structures are determined or estimated. For the structures that are designed with full capacity design criteria, R factor is the maximum one given in TEC2007. For other buildings, they are determined separately. Secondly, design spectrum with 475 year return period given in TEC2007 is reduced according to these R factors of the structures. Equation 3.1 is used to obtain the spectrum reduction function, $R_a(T)$ and Equation 3.2 is used to obtain the reduced design spectrum, $S_{aR}(T)$.

$$R_a(T) = \begin{cases} 1.5 + (R - 1.5) * \dfrac{T}{T_A} & (0 \leq T \leq T_A) \\ R & (T_A \leq T) \end{cases} \tag{3.1}$$

$$S_{aR}(T_1) = \frac{S_{ae}(T_1)}{R_a(T_1)} \tag{3.2}$$

In Equation 3.1, T_A is the first characteristic period of spectrum, which depends on the soil type chosen. In Equation 3.2, $S_{ae}(T_1)$ represents the design spectral acceleration with 475 year return period of TEC2007. Calculation of spectral intensity, SI, is done using Equation 3.3.

$$SI = \frac{S_{aGM}(T_1)}{S_{aR}(T_1)} \tag{3.3}$$

In Equation 3.3, S_{aGM} is the acceleration value obtained from the response spectrum of a ground motion. This value is obtained considering the first natural vibration of a structure. Figure 3.16 illustrates the calculation procedure of SI and the calculated R and SI values are tabulated in Table 3.8.

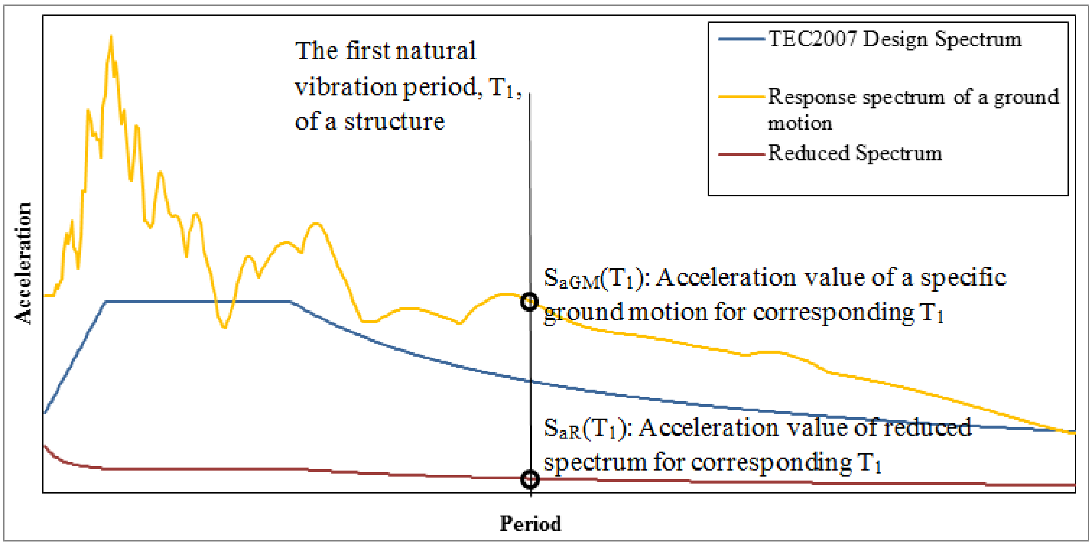


Figure 3.16: Illustration of calculation of acceleration values for SI calculation

The spectral intensities calculated are used in comparison between results of linear and nonlinear analysis. The level of nonlinearity on a structure is highly dependent on its spectral intensity, by definition. Therefore, it is used to distinguish the ground motions with respect to expected nonlinearities on the structure.

Reliability of linear elastic analysis methods for different type of structures is investigated at the end of this chapter using the obtained plastic rotation demands, DCRs as well as the spectral intensities and R factors.

Table 3.8: Force reduction factors and spectral intensities of buildings for different ground motions

			SI								
Building	Tn (sec)	R	Duzce (scale=1,0)	Duzce (scale=1,5)	Duzce (scale=2,0)	Northridge (scale=1,0)	Northridge (scale=1,5)	Northridge (scale=2,0)	Saratoga (scale=1,0)	Saratoga (scale=1,5)	Saratoga (scale=2,0)
6 Story	1,23	8	5,62	8,43	11,25	6,86	10,29	13,72	6,70	10,05	13,40
12 story with full capacity design	2,39	8	4,96	7,44	9,92	4,94	7,40	9,87	4,44	6,66	8,88
20 story wall-frame	2,60	7	4,29	6,43	8,58	3,32	4,98	6,65	3,28	4,92	6,56
4 story retrofitted	0,31	4	9,06	13,58	18,11	5,19	7,78	10,37	2,89	4,34	5,78
12 story with relaxed capacity design	2,50	8	4,72	7,08	9,44	4,46	6,69	8,92	3,90	5,86	7,81
12 story wall-frame	1,31	7	4,78	7,17	9,56	6,13	9,19	12,25	5,68	8,52	11,36
8 story	1,74	8	5,58	8,37	11,16	5,86	8,78	11,71	6,90	10,35	13,80

3.4. Presentation of Results

The level of nonlinearity in an existing structure that is analyzed by a linear elastic procedure can only be traced approximately, by using the DCR ratios. DCR's do not exactly indicate where inelastic actions develop, but give an approximate feeling where the capacities can be exceeded. Determining whether the results of linear elastic analysis is acceptable or not can only be decided through employing the DCR's since the basic objective is not to conduct nonlinear analysis. There is no other meaningful parameter which can be a basis of this decision from the results of linear elastic analysis. Therefore the decision on the acceptability of a linear elastic procedure has to be made by proposing DCR limits for different type of structures.

The deviations between the results of linear elastic and nonlinear response analysis mainly depend on the level of nonlinearity in the system, which is in turn related to the intensity of ground excitation. Lower excitation intensity leads to less nonlinearity, also lower values of DCR's in parallel. When the investigated system is regular, deviations may start at higher intensities (higher DCR's) and vice versa. Presence of irregularities has a strong influence on the values of DCR's that indicate the limitations to linear elastic procedures.

Basic irregularities concerning the localization of inelastic actions are the presence of weak stories, formation of column mechanism, and severe torsional eccentricity. Another factor, which is not an irregularity, is the presence of a dominant shear wall in the system. Yielding of shear wall at the base is a significant local nonlinearity, which causes significant change in the overall deformation pattern as compared to linear elastic response. On the other hand, if the system is regular and conforms to capacity design principles with strong columns and weak beams throughout the system, linear elastic deformation patterns do not deviate from the inelastic deformation patterns appreciably until the columns yield significantly at the base. Accordingly, higher DCR limits can be allowed for these systems.

Regular systems with capacity design and systems with different irregularities are investigated separately in the following sections, and different DCR limits are proposed for each system type. The results obtained are compared with the DCR limits suggested by ASCE 41 and Eurocode 8 and their validity is discussed.

3.5. Regular Frames with Capacity Design

A five story and two twelve story frames with capacity design are investigated as the buildings with no irregularities. The second twelve story building is obtained from the first one by reducing the column capacities by 25%. However the revised building still satisfies capacity design principles, with less margin however. The results are presented separately for each case study building with the related discussions. Different graphics are provided for each element type and ground motion. In these graphics, plastic rotation demands from linear and nonlinear procedures are provided along with the performance limits and DCR's of the buildings. In addition to these graphics, average column DCRs and ratio between average column and average beam DCRs at each story are tabulated for each ground motion. Average DCR of columns at each story is determined using Equation 1.4.

A regular frame which fully satisfies capacity design principles with columns stronger than the beams is expected to develop an inelastic deformation pattern schematized in Figure 3.17, on the left. When the flexural strength of columns are not significantly larger than the beams, a mixed column-beam mechanism develops, as also shown in Figure 3.17, on the right.

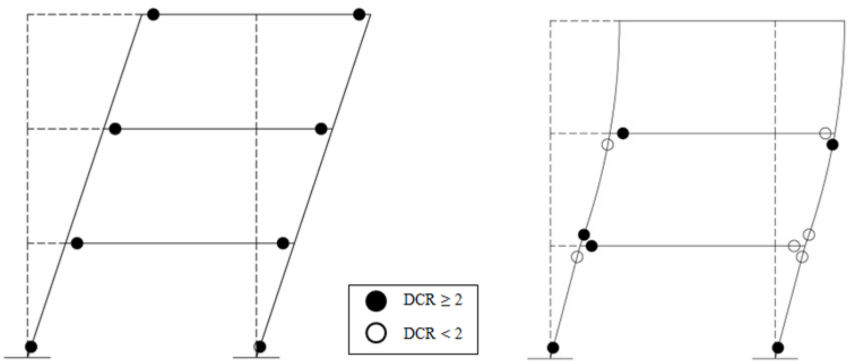


Figure 3.17: Beam and mixed beam-column mechanisms of regular frames

3.5.1. Five Story R/C Frame (R=8)

Pushover and response spectrum analysis is applied to this building. Both procedures are implemented under the 25, 50, 75 and 100 percent of design spectrum. In order to control the irregularities of this building, the average DCR of columns per story and the ratio of average column DCR to average beam DCR, namely r_{DCR}, are calculated in Tables 3.9 and 3.10, respectively.

Table 3.9: Average column DCRs per story for different scaled design spectrum for 5 story frame

	Design Spectrum Scale (%)			
story	25	50	75	100
1	0,72	**1,43**	**2,14**	**2,86**
2	0,57	1,04	1,54	2,03
3	0,46	0,82	1,19	1,57
4	0,38	0,60	0,85	1,10
5	0,33	0,42	0,52	0,63

Table 3.10: r_{DCR} per story for different scaled design spectrum for 5 story frame

	Design Spectrum Scale (%)			
story	25	50	75	100
1	0,83	**0,99**	**1,02**	**1,03**
2	0,62	0,67	0,68	0,68
3	0,55	0,61	0,63	0,63
4	0,51	0,60	0,63	0,64
5	0,64	0,68	0,69	0,69

For the first story, average column DCRs are very close to average beam DCRs. The main reason is that strong column weak beam principle is barely satisfied for this structure and this can cause some problems for higher intensities in linear elastic analysis. Plastic rotation demands from RSA and PO on each column and each beam are given in Figures 3.18 and 3.19, respectively.

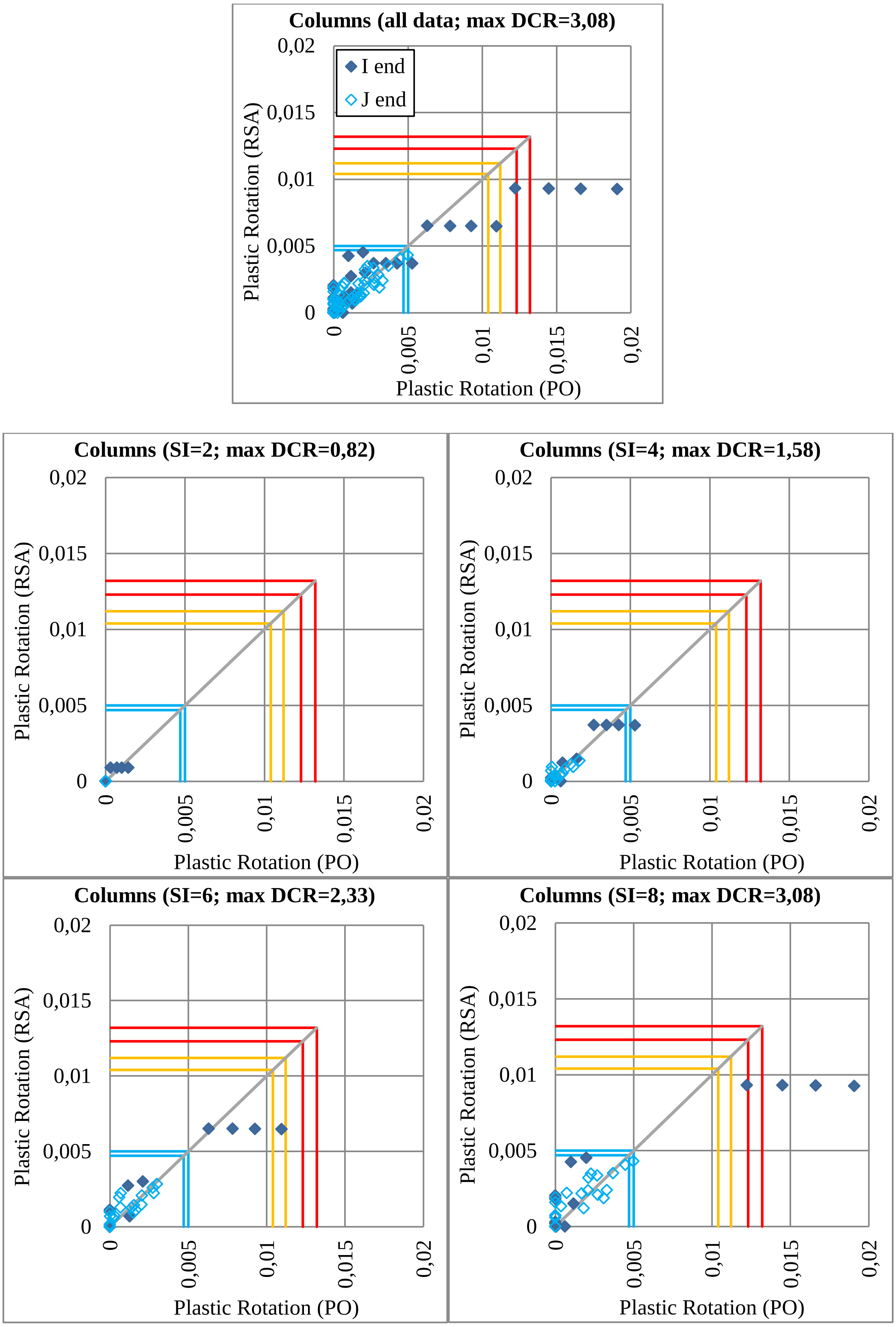


Figure 3.18: Plastic rotation demands from RSA and PO for columns of 5 story R/C frame

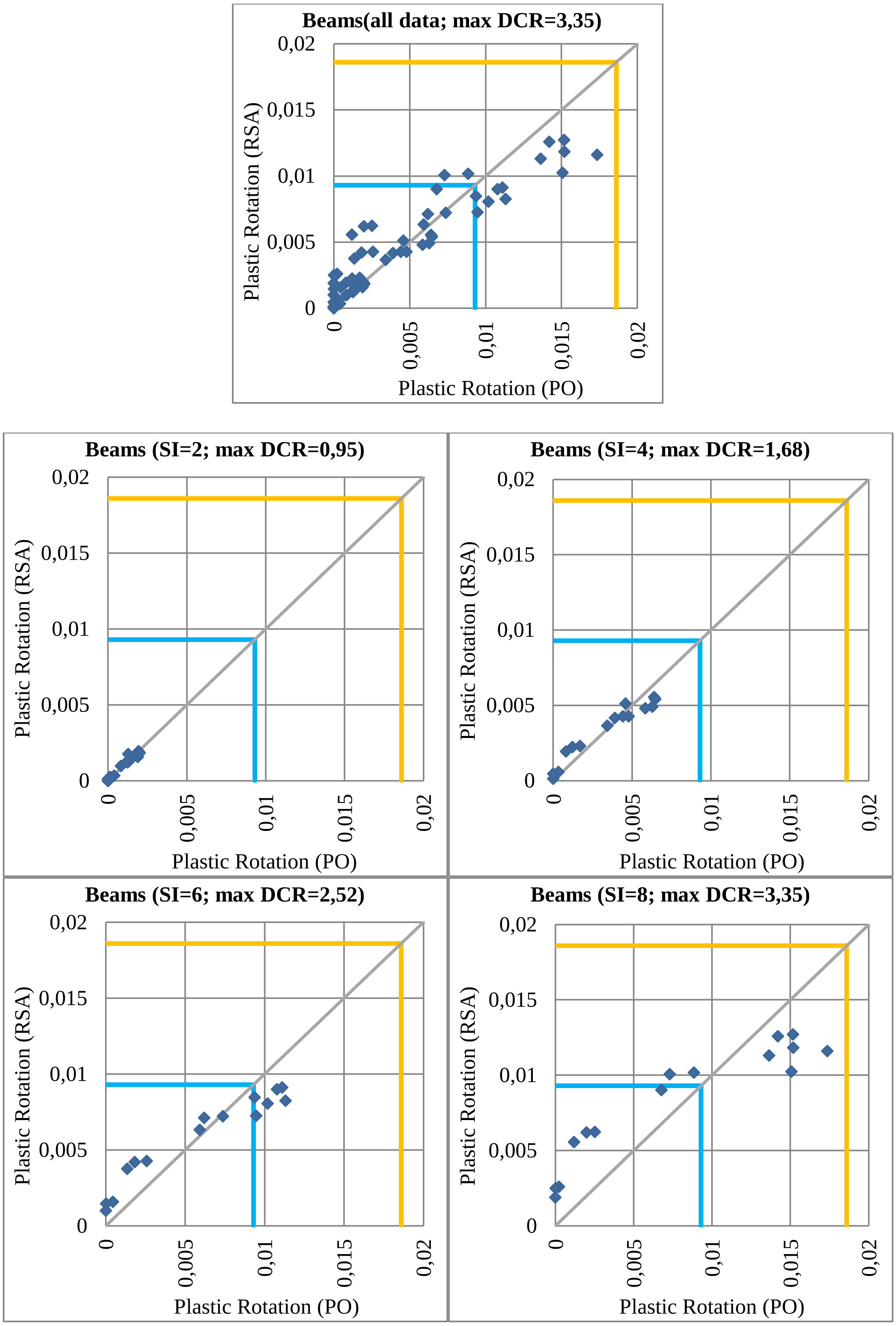


Figure 3.19: Plastic rotation demands from RSA and PO for beams of 5 story R/C frame

The following observations can be made from Figures 3.18 and 3.19 along with Tables 3.9 and 3.10.

- For small intensities, linear and nonlinear methods resulted very similar plastic rotation demands at all column ends. When the intensity is increased, the results from linear analysis start to deviate from inelastic analysis and become unsafe. From the last two graphics of Figure 3.18, in which the ground motion intensity is higher, it can be observed that even J ends of first story columns and I ends of second story columns undergo significant plastic rotation demands according to pushover analysis. This means that this structure develops weak story and hence linear response spectrum analysis cannot estimate the response of the structure when plastic hinge mechanisms occur at the first story columns. Therefore, as the intensity increases, the validity of linear analysis becomes questionable.
- It is observed that linear methods are not reliable when average DCR of columns are greater than average DCR of beams at a story. However, for the ratio of 0.99 in Table 3.10, linear methods could estimate the behavior of the structure well. Therefore, it cannot solely rely on average column DCR to average beam DCR at a story. Another possible criterion to add is the value of average DCR of columns at a story. In Table 3.9, it is seen that when the average column DCR exceeds 2, linear methods are not reliable. However, this conclusion is again not enough. As a result, combining two conclusions above, it can be said that if r_{DCR} value of any story is between 0.75 and 1.00, and any column DCR exceeds 2, linear methods are not safe to employ. This conclusion is partly consistent with ASCE 41.
- Unlike columns, beam responses are estimated considerably well by linear analysis. For small intensities, almost perfect correlation between linear and nonlinear methods is achieved. The divergence increases with the increasing intensity and the maximum error of linear method can be observed from the last graphic. At that intensity, because of the effect of weak story, the estimation of linear methods is not accurate for beams, too. Therefore the maximum allowable DCR can be set as 3 for beams.

3.5.2. Twelve Story R/C Frame with Capacity Design (R=8)

In order to control irregularities of this building, the average DCR of columns per story and r_{DCR} values are given in Tables 3.11 and 3.12.

Table 3.11: Average column DCRs per story for different ground motions for 12 story frame with capacity design

	GM name (scale)								
story	Düzce (1,0)	Düzce (1,5)	Düzce (2,0)	Northridge (1,0)	Northridge (1,5)	Northridge (2,0)	Saratoga (1,0)	Saratoga (1,5)	Saratoga (2,0)
1	1,42	2,13	2,84	1,46	2,18	2,91	1,09	1,64	2,18
2	0,83	1,19	1,55	0,86	1,23	1,61	0,71	1,00	1,29
3	0,74	1,07	1,40	0,74	1,07	1,40	0,64	0,91	1,19
4	0,76	1,07	1,40	0,72	1,02	1,33	0,65	0,90	1,17
5	0,91	1,31	1,72	0,87	1,25	1,64	0,77	1,09	1,43
6	0,85	1,20	1,56	0,84	1,18	1,53	0,72	0,99	1,27
7	0,83	1,17	1,52	0,85	1,20	1,56	0,68	0,93	1,19
8	0,84	1,17	1,52	0,88	1,24	1,61	0,66	0,89	1,13
9	1,07	1,48	1,90	1,14	1,59	2,05	0,82	1,07	1,34
10	1,05	1,44	1,85	1,08	1,48	1,89	0,77	0,98	1,20
11	0,93	1,25	1,58	0,89	1,17	1,46	0,66	0,80	0,96
12	0,82	0,96	1,12	0,78	0,89	1,01	0,70	0,75	0,82

Table 3.12: r_{DCR} per story for different ground motions for 12 story frame with capacity design

	GM name (scale)								
story	Düzce (1,0)	Düzce (1,5)	Düzce (2,0)	Northridge (1,0)	Northridge (1,5)	Northridge (2,0)	Saratoga (1,0)	Saratoga (1,5)	Saratoga (2,0)
1	0,52	0,55	0,56	0,52	0,55	0,56	0,47	0,51	0,53
2	0,30	0,30	0,30	0,30	0,30	0,30	0,29	0,30	0,30
3	0,27	0,28	0,29	0,27	0,28	0,28	0,26	0,28	0,28
4	0,28	0,29	0,29	0,28	0,28	0,29	0,27	0,28	0,28
5	0,36	0,39	0,40	0,35	0,38	0,39	0,34	0,37	0,39
6	0,34	0,36	0,37	0,34	0,36	0,36	0,33	0,35	0,36
7	0,35	0,37	0,37	0,34	0,36	0,37	0,32	0,35	0,36
8	0,36	0,38	0,39	0,36	0,38	0,39	0,33	0,36	0,37
9	0,34	0,37	0,38	0,34	0,36	0,37	0,31	0,34	0,35
10	0,36	0,39	0,41	0,36	0,38	0,40	0,32	0,35	0,37
11	0,35	0,39	0,42	0,34	0,37	0,39	0,29	0,32	0,35
12	0,38	0,41	0,43	0,37	0,39	0,40	0,35	0,37	0,38

Strong column-weak beam principle is well satisfied for this building. However, according to ASCE41, this structure has weak story irregularity at the first story under all ground motions.

Average plastic rotation demands per story from RSA and NRHA on columns and beams are shown in Figures 3.20 and 3.21, respectively.

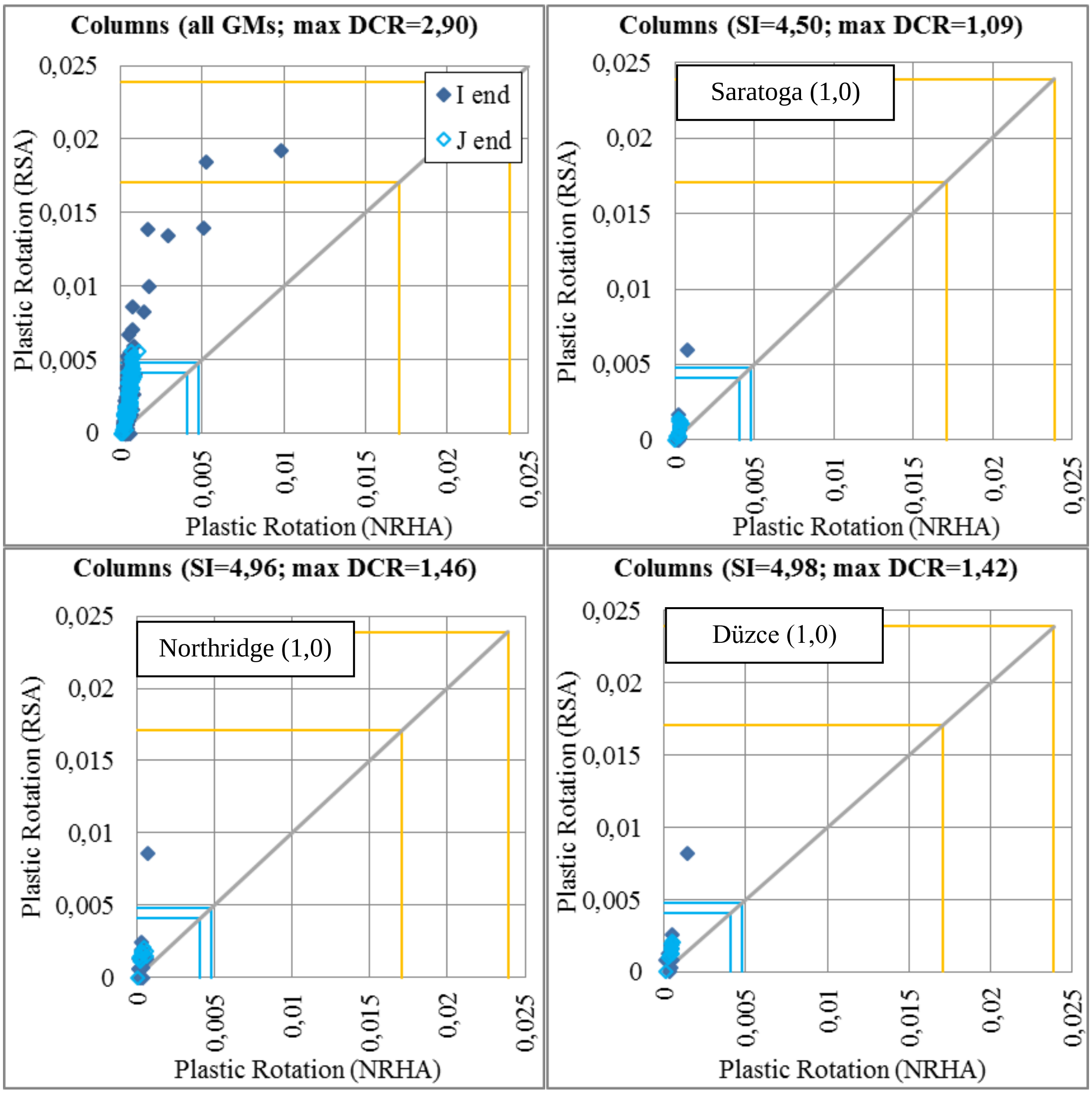


Figure 3.20: Average plastic rotation demands from RSA and NRHA for columns of the 12 story R/C frame with capacity design

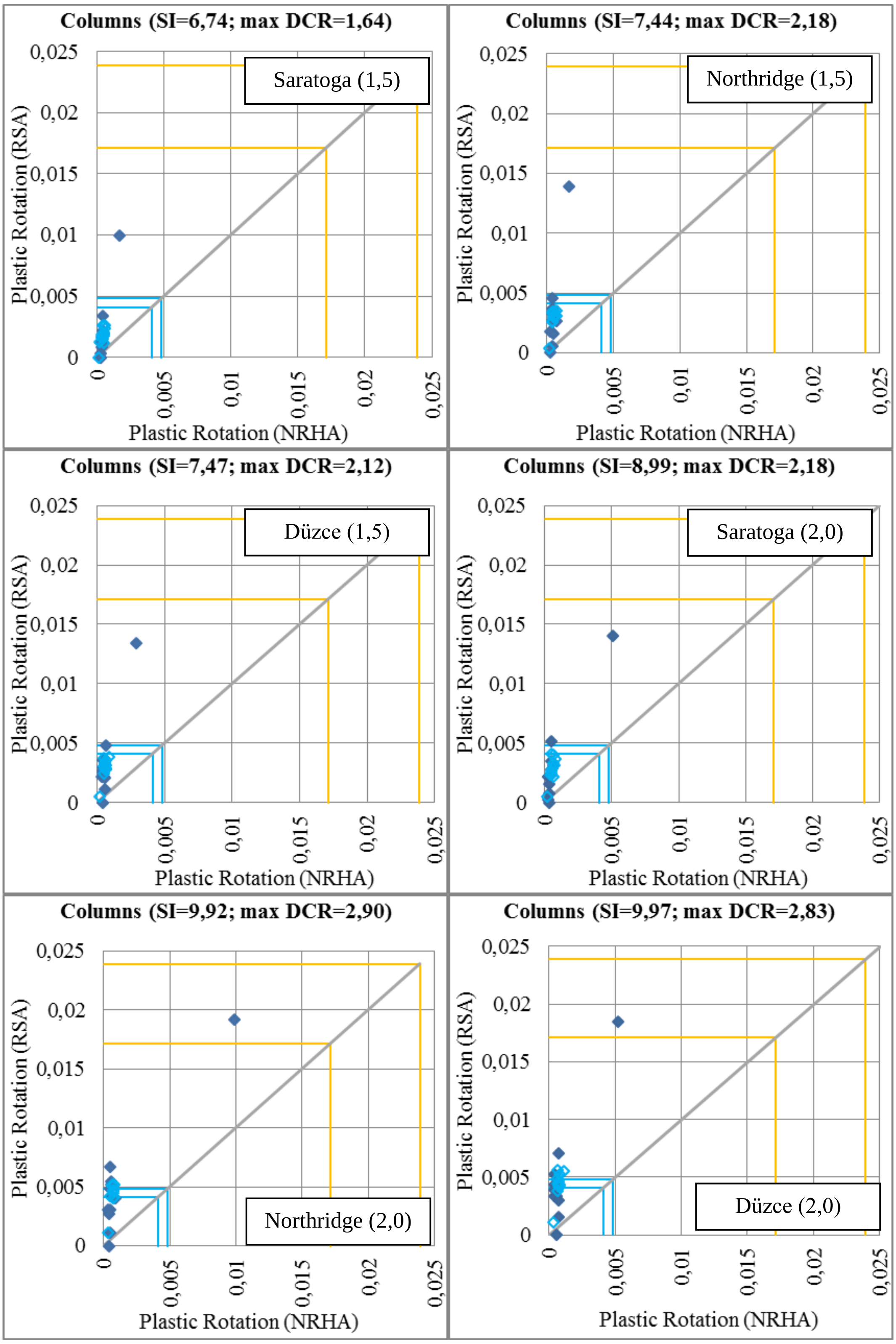


Figure 3.20 con'd: Average plastic rotation demands from RSA and NRHA for columns of the 12 story R/C frame with capacity design

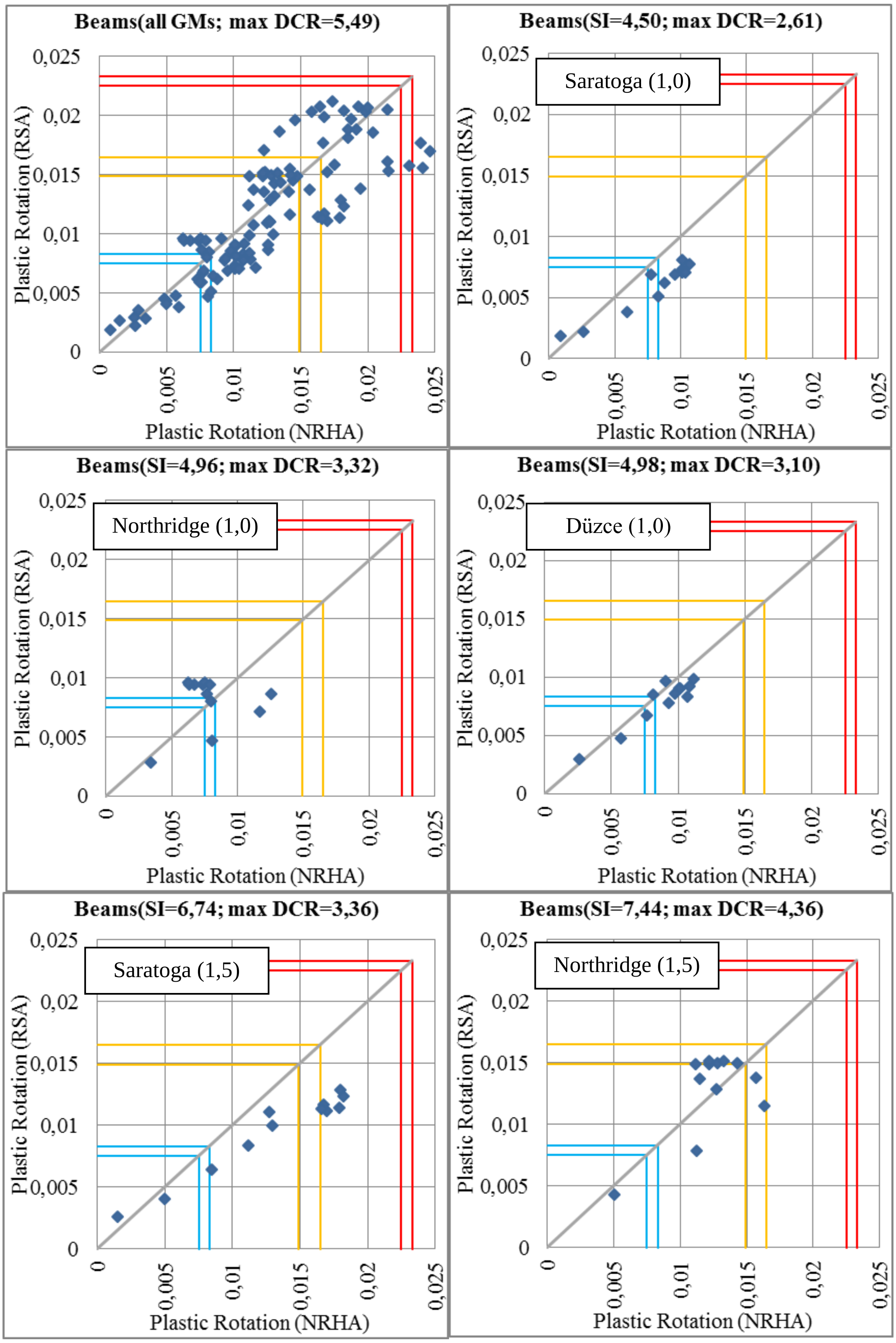


Figure 3.21: Average plastic rotation demands from RSA and NRHA for beams of the12 story R/C frame with capacity design

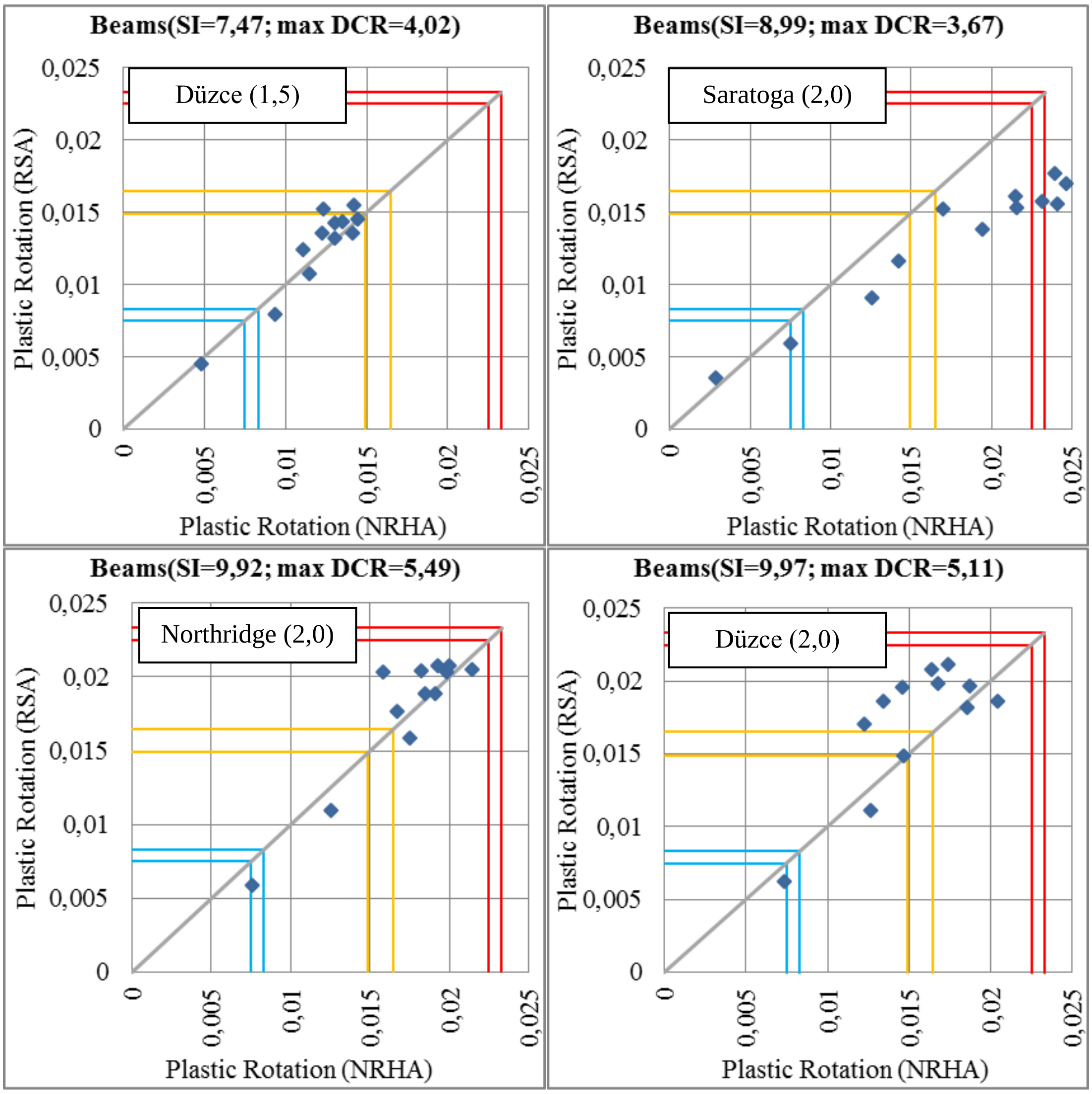


Figure 3.21 cont'd: Average plastic rotation demands from RSA and NRHA for beams of the12 story R/C frame with capacity design

The following outcomes are obtained from Figure 3.20 and 3.21 along with Tables 3.11 and 3.12.

- All of the plastic rotation demands from the linear RSA method at the column ends are larger than the ones from nonlinear method. It is clear from NRHA results that structure does not have any weak story irregularity, as opposed to ASCE41 consideration. I ends of base columns are the only ones that have considerable plastic rotation demands in NRHA. Since the structure does not have irregularities, and the only important feature for this building is the contribution of higher modes, which is also considered in response spectrum analysis, the results show that linear method is safe to employ for this building. It can be safely concluded that if r_{DCR} values of all joints are smaller than 0.75, there is no possibility for weak story irregularity and the maximum allowable column DCR can be set as 3 for this condition.

- The downside of linear method for this building is that the results are overestimated. The demands calculated by RSA are much higher than those of the benchmark NRHA. For instance, according to NRHA, maximum mean plastic rotation demand is 0.01 at the column bases whereas RSA's maximum value at column bases are almost 0.02. If assessment of this structure is done by the linear elastic procedure, then the linear procedure results would be over-conservative, but safe.
- The overall responses are very similar for beams from both linear and nonlinear methods. Unlike columns, there is no over-conservative results obtained from linear procedure. Plastic hinges are expected to occur at beam ends, and this behavior is well estimated by linear RSA procedure for this building.
- When the results under each ground motion are observed separately, RSA results do not seem accurate. For example for first, fourth and seventh graphic boxes of Figure 3.21, demands from RSA are well below demands from NRHA. Since they are all Saratoga ground motions, and this behavior cannot be explained by increase in DCR or SI, ground motion characteristics might also be effective.
- Although demands at beam ends are consistent for both linear and nonlinear analysis, the divergence trend can be observed by the increasing intensity. For the highest intensities, difference between linear and nonlinear methods start to become significant. Therefore, it might be reasonable to set a maximum limit DCR for beams as 6 for this building in order to employ linear assessment procedures.

3.5.3. Twelve Story R/C Frame with Relaxed Capacity Design (R=8)

To control the weak story irregularity and column mechanism formation, the average DCR of columns and r_{DCR} values are presented in Tables 3.13 and 3.14.

Table 3.13: Average column DCRs per story for different ground motions for 12 story frame with relaxed capacity design

	GM (scale)								
story	Düzce (1,0)	Düzce (1,5)	Düzce (2,0)	Northridge (1,0)	Northridge (1,5)	Northridge (2,0)	Saratoga (1,0)	Saratoga (1,5)	Saratoga (2,0)
1	1,44	2,16	2,88	1,56	2,33	3,11	1,05	1,56	2,08
2	0,90	1,26	1,64	0,95	1,34	1,75	0,73	1,01	1,29
3	0,81	1,16	1,52	0,82	1,17	1,52	0,67	0,93	1,20
4	0,86	1,21	1,57	0,81	1,14	1,47	0,70	0,96	1,23
5	0,99	1,42	1,87	0,92	1,32	1,73	0,79	1,10	1,43
6	0,98	1,38	1,80	0,97	1,36	1,77	0,79	1,07	1,37
7	0,98	1,37	1,77	1,04	1,47	1,91	0,78	1,05	1,33
8	0,98	1,36	1,75	1,11	1,56	2,03	0,77	1,02	1,28
9	1,56	2,15	2,76	1,80	2,52	3,28	1,20	1,55	1,93
10	1,60	2,17	2,77	1,78	2,46	3,17	1,18	1,49	1,82
11	1,52	2,02	2,54	1,57	2,10	2,66	1,10	1,32	1,57
12	1,55	1,81	2,09	1,52	1,77	2,03	1,33	1,43	1,54

Table 3.14: r_{DCR} per story for different ground motions for 12 story frame with relaxed capacity design

	GM (scale)								
story	Düzce (1,0)	Düzce (1,5)	Düzce (2,0)	Northridge (1,0)	Northridge (1,5)	Northridge (2,0)	Saratoga (1,0)	Saratoga (1,5)	Saratoga (2,0)
1	0,54	0,57	0,59	0,54	0,57	0,58	0,47	0,53	0,55
2	0,33	0,34	0,34	0,34	0,34	0,34	0,32	0,33	0,33
3	0,31	0,33	0,33	0,32	0,33	0,33	0,30	0,32	0,33
4	0,33	0,35	0,35	0,33	0,35	0,35	0,32	0,34	0,34
5	0,41	0,44	0,45	0,40	0,43	0,45	0,38	0,42	0,44
6	0,41	0,44	0,45	0,40	0,43	0,44	0,39	0,41	0,43
7	0,42	0,45	0,46	0,42	0,44	0,46	0,39	0,43	0,44
8	0,44	0,47	0,48	0,44	0,47	0,48	0,40	0,44	0,46
9	0,52	0,55	0,57	0,52	0,55	0,57	0,47	0,51	0,54
10	0,56	0,61	0,63	0,57	0,61	0,63	0,50	0,55	0,58
11	0,58	0,65	0,70	0,58	0,64	0,67	0,48	0,54	0,58
12	0,75	0,81	0,86	0,72	0,77	0,81	0,67	0,71	0,74

Although the column capacities are reduced compared to the previous case study, average column DCR to average beam DCR, r_{DCR}, does not indicate any irregularity of column mechanism. However, weak stories are expected to occur at several stories considering ASCE41 provisions and DCR values in Table 3.13.

Average plastic rotation demands per story from RSA and NRHA on columns and beams are given in Figures 3.22 and 3.23, respectively.

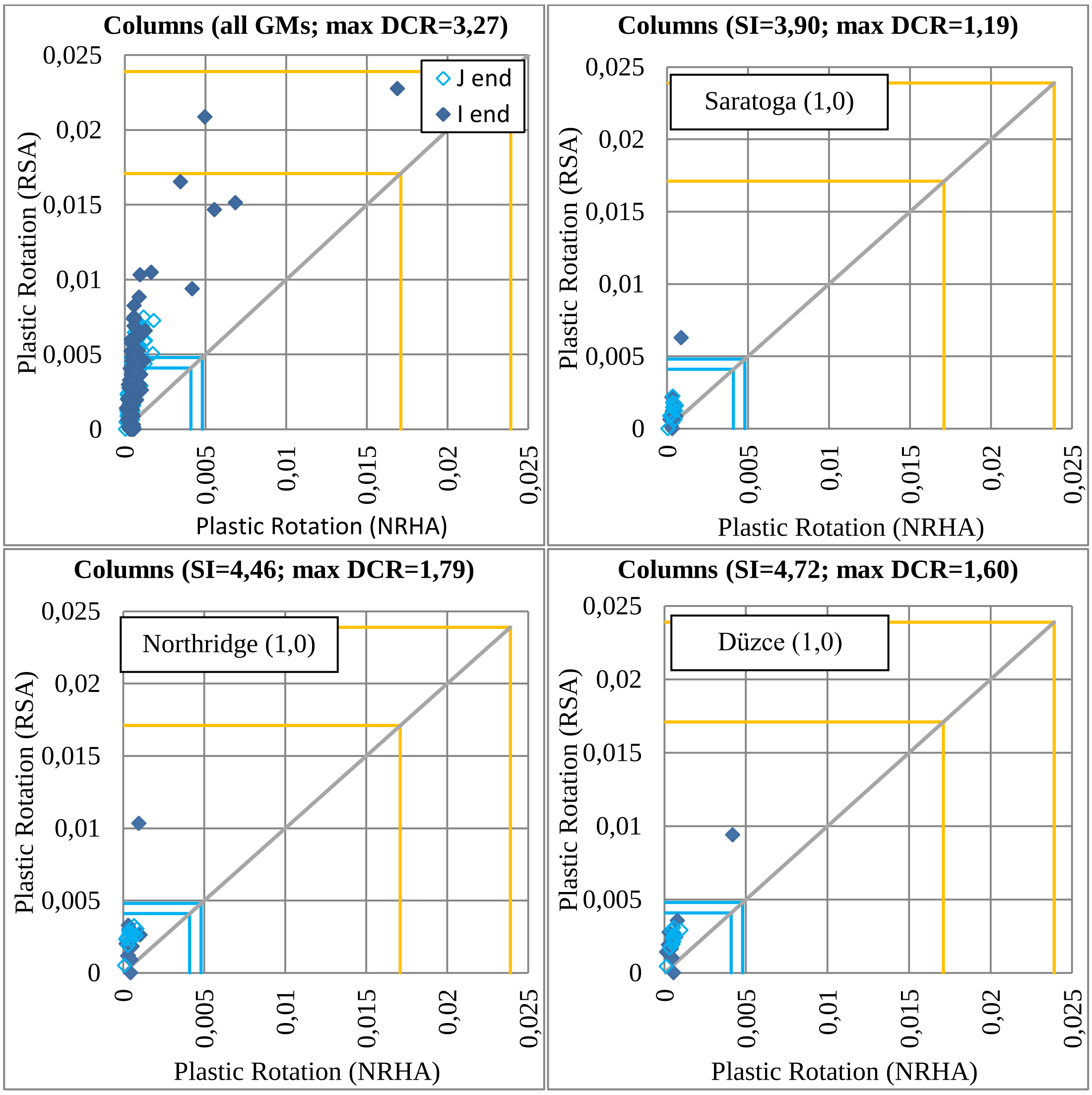


Figure 3.22: Average plastic rotation demands from RSA and NRHA for columns of 12 story R/C frame with relaxed capacity design

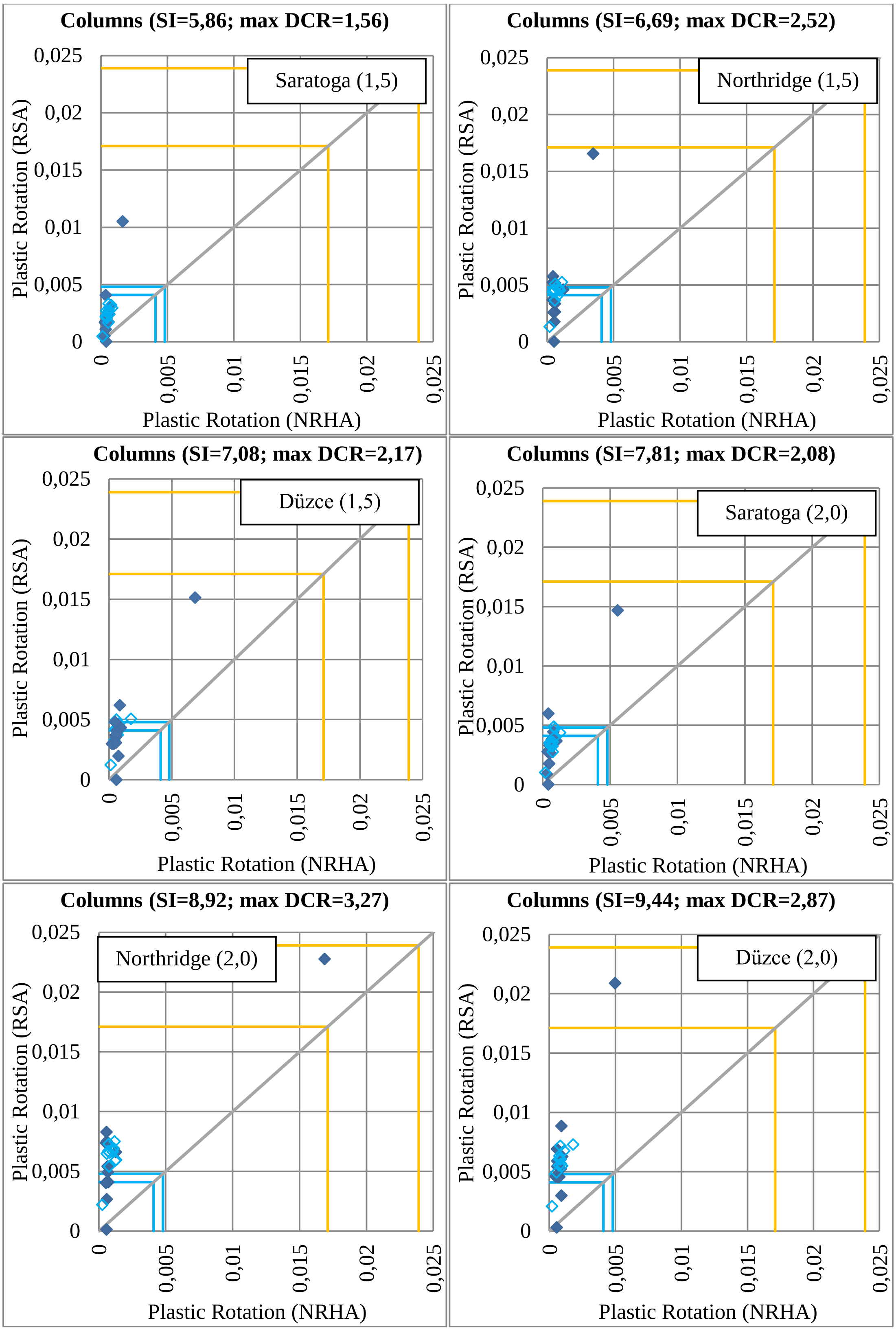


Figure 3.22 cont'd: Average plastic rotation demands from RSA and NRHA for columns of 12 story R/C frame with relaxed capacity design

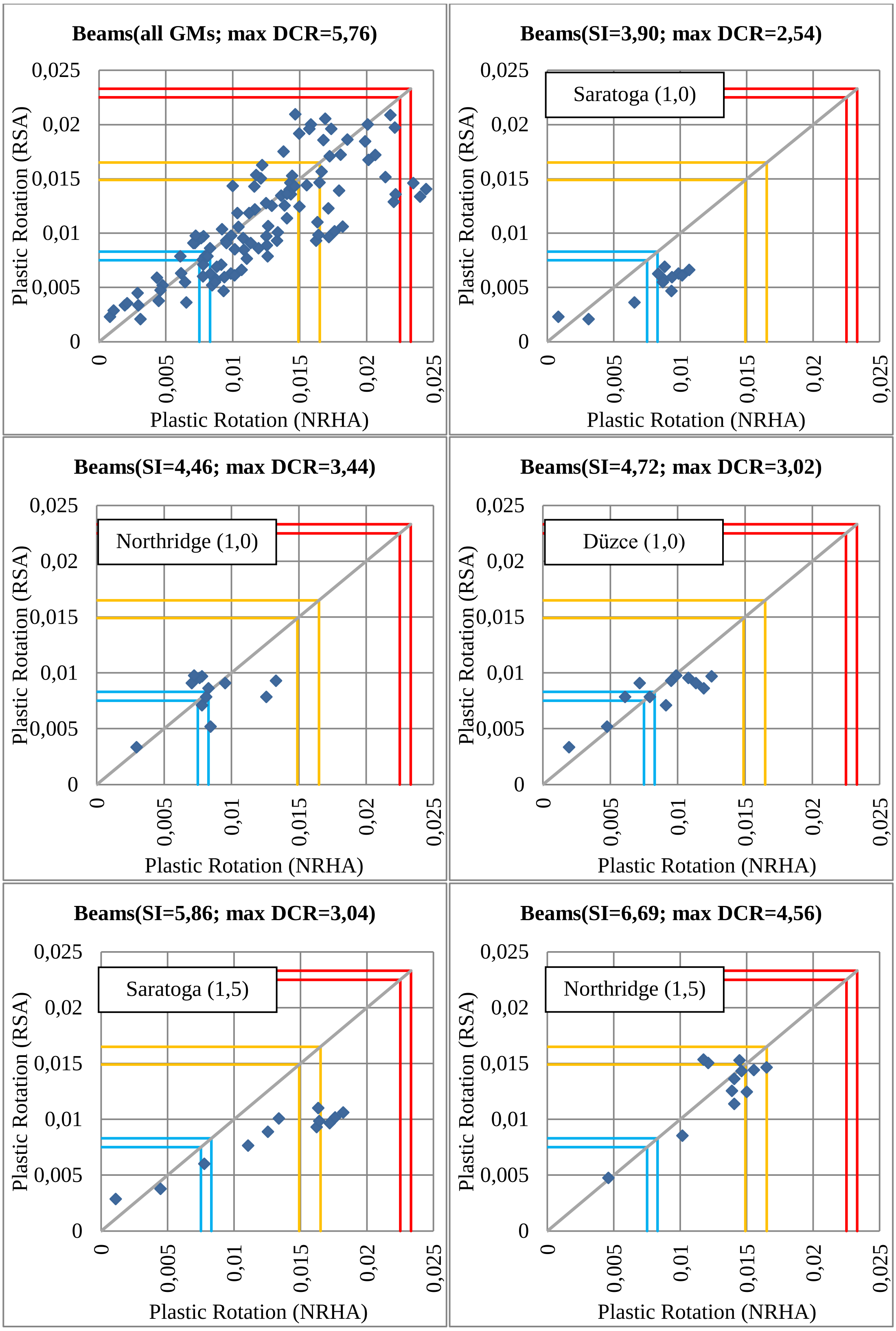


Figure 3.23: Average plastic rotation demands from RSA and NRHA for beams of the 12 story R/C frame with relaxed capacity design

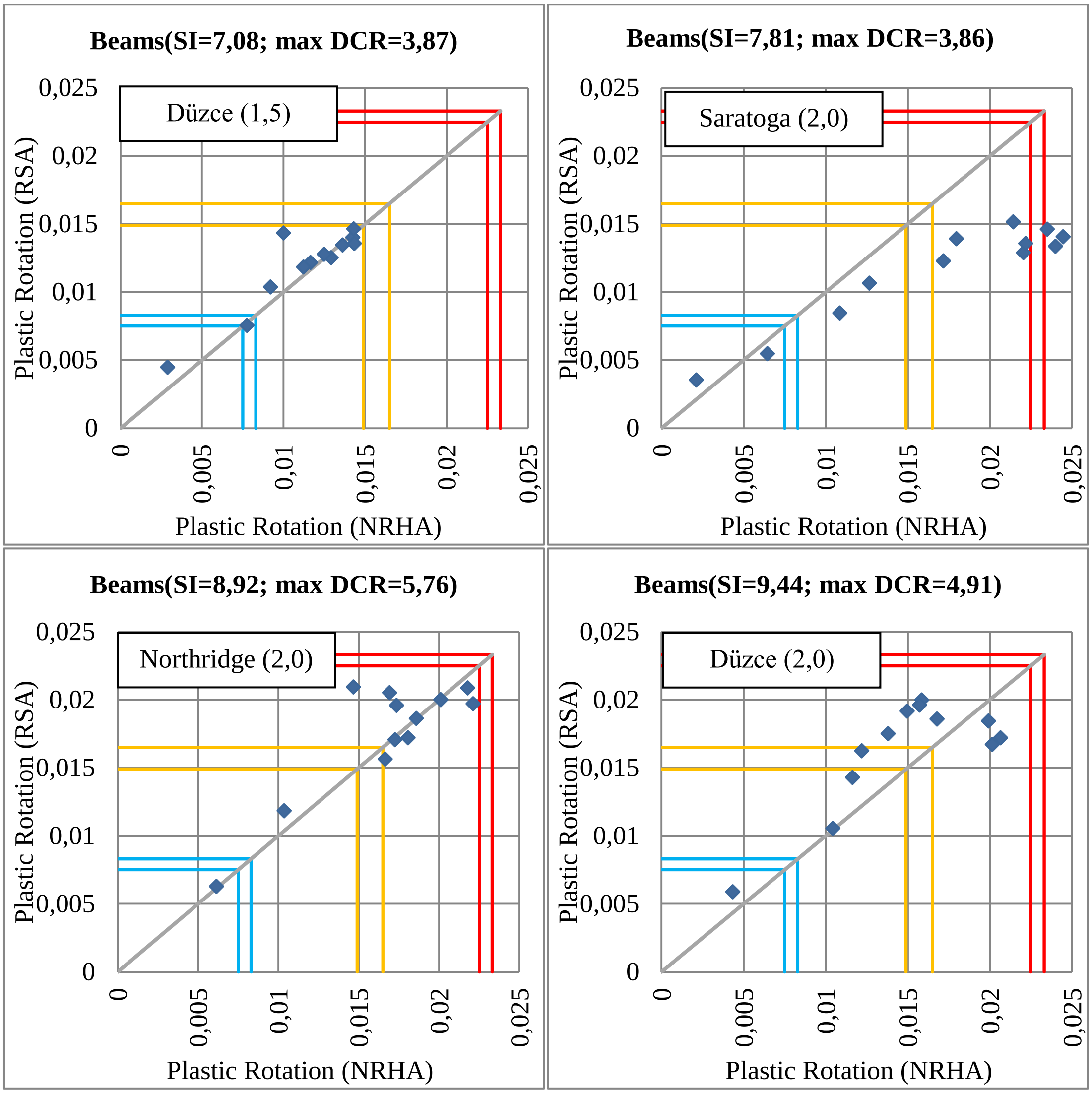


Figure 3.23 cont'd: Average plastic rotation demands from RSA and NRHA for beams of the 12 story R/C frame with relaxed capacity design

The following outcomes are obtained from Figure 3.22 and 3.23 along with Tables 3.13 and 3.14.

- Compared to full capacity design, plastic rotation demands developed at J ends of some columns at higher intensities, which suggests that local column mechanisms occur in this building. However, these mechanisms did not govern the overall response of the structure. Therefore, the results from linear analysis are acceptable.
- Although results of RSA remained on the safe side for columns for all ground motions, for higher column DCRs, RSA results tend to become closer to NRHA.

The maximum column DCR to employ linear methods safely can be set to 3 for this structure, which is consistent considering the limiting DCR value being set as 3 for columns for r_{DCR} smaller than 0.75.

- The tendency of responses obtained for beams are similar to the ones obtained for the full capacity designed structure, but RSA is less accurate for beams for this building although the errors of RSA are not significant. Linear analysis seems to be accurate up to maximum beam DCR of 6 for this building also. Similar to full capacity designed structure, RSA is unsafe for Saratoga ground motions. This can be related to the ground motion characteristics, which is not investigated herein.

3.6. Frames with Weak Story Irregularity and Column Mechanism

There is only one building in this category, which is the 5 story building of Section 3.5.1, but with reduced column capacities. Typical inelastic deformation pattern of such a building is schematized in Figure 3.24.

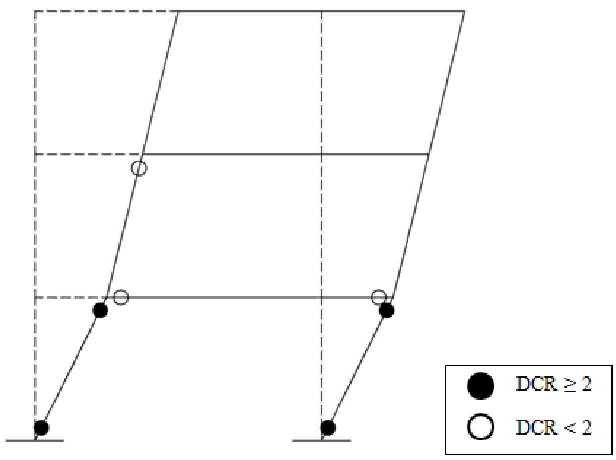


Figure 3.24: Typical inelastic deformation pattern of a frame with weak story irregularity

3.6.1 Five Story R/C Frame with Reduced Column Capacity (R=4)

In order to control the irregularities of this building, the average DCR of columns per story and r_{DCR} values similar to previous structures are provided in Tables 3.15 and 3.16, respectively.

Table 3.15 Average column DCRs per story for different scaled design spectrum for 5 story frame with reduced column capacity

	Design Spectrum Scale (%)			
story	25	50	75	100
1	1,14	2,26	3,38	4,51
2	0,98	1,78	2,61	3,45
3	0,82	1,45	2,11	2,78
4	0,68	1,11	1,56	2,03
5	0,60	0,77	0,96	1,17

Table 3.16: r_{DCR} per story for different scaled design spectrum for 5 story frame with reduced column capacity

	Design Spectrum Scale (%)			
story	25	50	75	100
1	1,30	1,54	1,58	1,59
2	1,08	1,16	1,17	1,17
3	1,00	1,12	1,15	1,16
4	0,95	1,14	1,21	1,24
5	1,20	1,33	1,39	1,40

Strong column weak beam principle is violated for almost all of the joints of this structure. This effect can also be observed from Table 3.16, in which average column DCRs are greater than the average beam DCRs. Since the average DCRs of first stories exceed 125% of the second story DCRs in Table 3.15, weak story irregularity exists for this structure according to ASCE 41.

Plastic rotation demands from RSA and PO on each column and each beam are given in Figures 3.23 and 3.24, respectively.

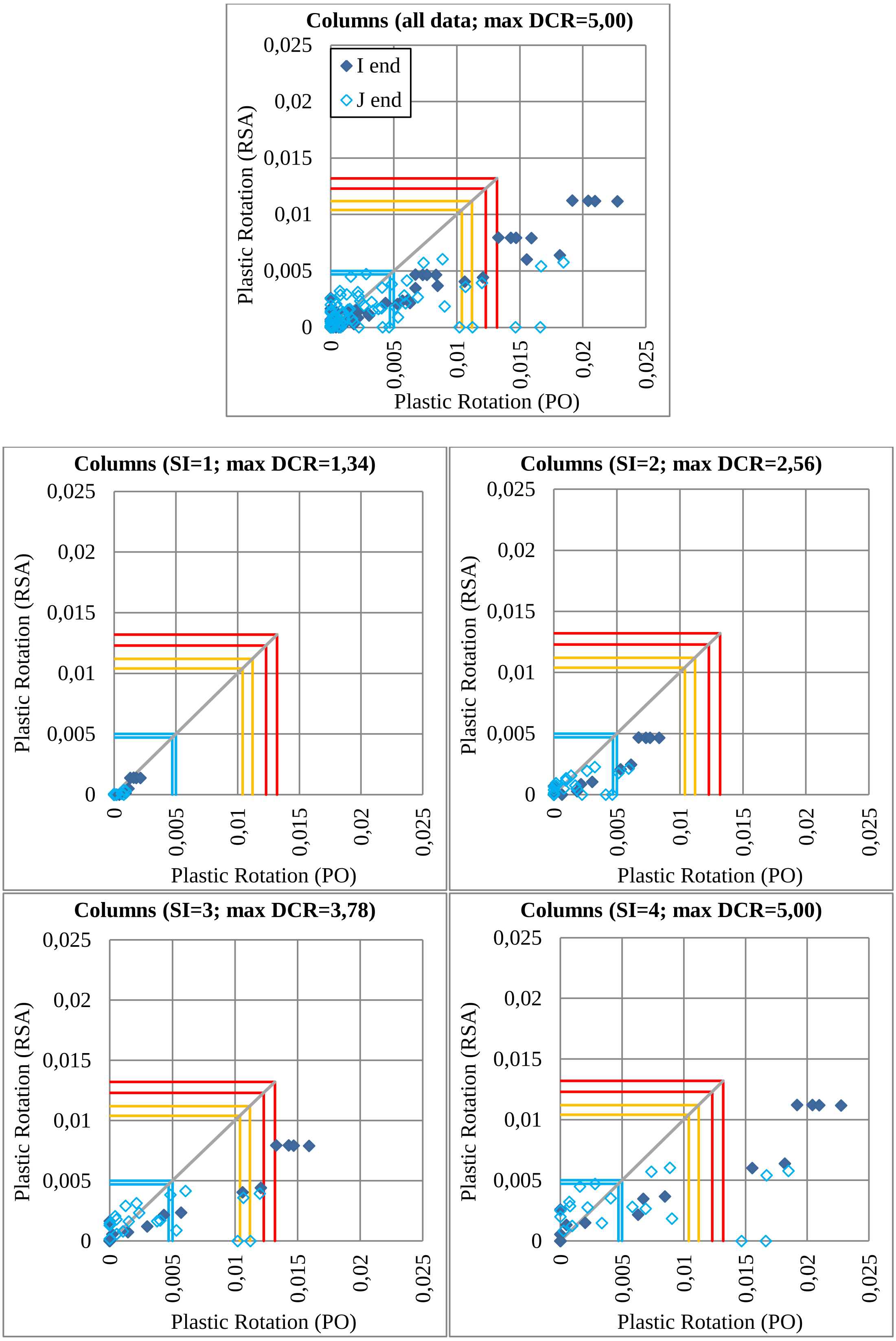


Figure 3.25: Plastic rotation demands from RSA and PO for columns of 5 story R/C frame with reduced column capacity

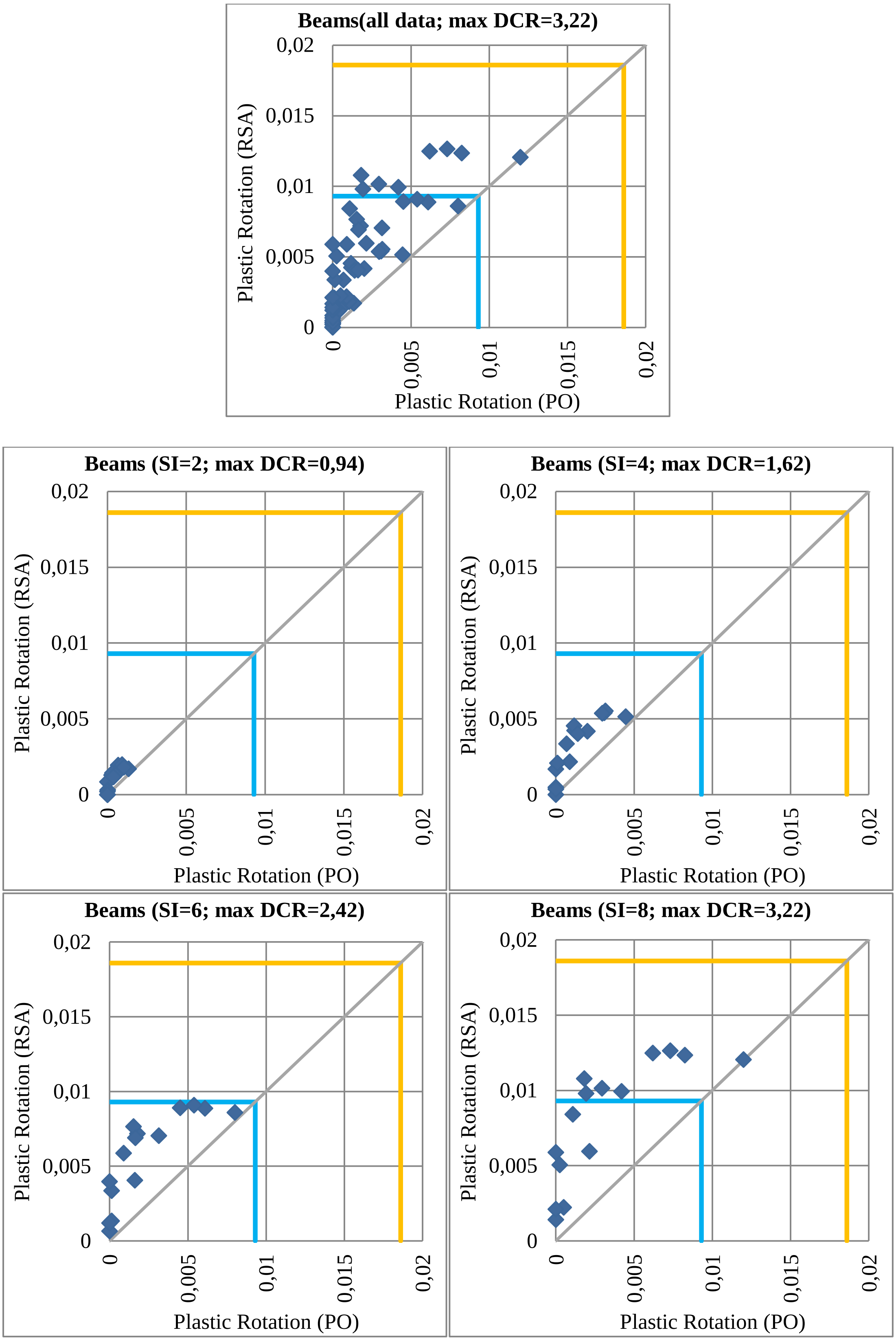


Figure 3.26: Plastic rotation demands from RSA and PO for beams of the 5 story R/C frame with reduced column capacity

The following outcomes are obtained from Figure 3.25 and 3.26 along with Tables 3.15 and 3.16.

- Almost all of the joints of this frame violate strong column-weak beam principle. This creates column mechanisms throughout the frame. Even for small intensities, plastic rotation demands develop at both ends of columns. When column mechanism occurs, all inelastic deformations are concentrated on column ends. This formation cannot be estimated by linear methods. When the inelastic deformations are concentrated at column ends (i.e. plastic hinges on column ends occur earlier than beam ends), whole deformed shape of the structure changes, and linear analysis cannot estimate the actual deformation response when this happens. That is the reason, even for small intensities, that linear method fail to estimate the plastic rotation demands at column ends.
- In almost all of the joints, r_{DCR} ratio is greater than 1. When this ratio is larger than 1, it indicates the strong possibility of development of column mechanism. When that mechanism is present, and average column DCR of any story exceeds 1.5, linear methods should not be permitted. For the smallest intensity, although the column to beam average DCR is greater than 1, since average column DCR is low, linear method can estimate the response well. However, when the average column DCR exceeds 1.5 at a story, linear procedure completely fails. As a result, when the structure has weak story irregularity, the maximum allowable DCR on columns should be maximum 1.5 for the linear procedure to be employed in assessment.
- Demands from RSA are higher than NRHA for beams. This is caused by the early plastic hinge development at column ends. Since the deformation is concentrated on columns, beam deformations are less in NRHA compared to RSA. If the plastic rotation demands of both five story buildings are compared, it is observed that demands from linear analysis is almost the same for two cases whereas demands from nonlinear analysis are reduced significantly for the building with reduced column capacity. The values for the highest intensity are shown in Table 3.17. This indicates that linear method cannot estimate the response when column mechanism occurs. Since the structure response is

completely dominated by column mechanisms, it is logical to use limiting DCR value for beam as 3, which is the same for the systems with mixed column and beam mechanism structures.

Table 3.17: Comparison of average plastic rotation demands of the beams of five story frame and five story frame with reduced column capacity for the highest intensity

	5 story with high column capacity		5 story with reduced column capacity	
story	RSA	NRHA	RSA	NRHA
1	0,0115	0,0159	0,0116	0,0074
2	0,0114	0,0122	0,0111	0,0041
3	0,0084	0,0055	0,0080	0,0017
4	0,0060	0,0019	0,0056	0,0008
5	0,0023	0,0001	0,0019	0,0002

3.7 Wall-Frame Systems

There are three buildings in this section, all of which have a very dominant shear walls. These buildings are twelve story R/C wall-frame, twenty story R/C wall-frame and four story retrofitted school building. Typical inelastic deformation pattern of a frame-wall system is shown in Figure 3.27.

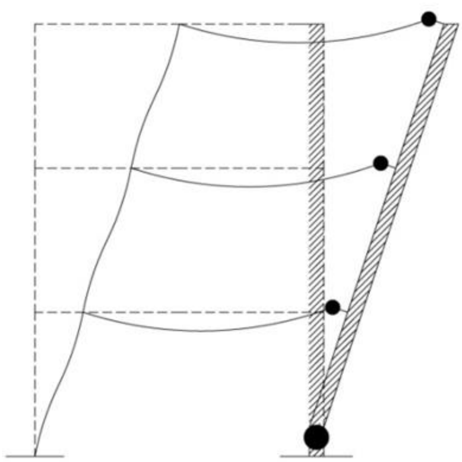


Figure 3.27: Typical inelastic deformation pattern of a frame-wall system

3.7.1. Twelve Story R/C Wall-Frame (R=7)

Shear wall in this building dominates the response of structure. About 93% of total base shear is carried by the shear wall. Therefore, only shear wall DCRs per story is given in Table 3.18.

Table 3.18: DCRs of shear wall members of 12 story R/C wall-frame

	GM name (scale)								
story	Düzce (1,0)	Düzce (1,5)	Düzce (2,0)	Northridge (1,0)	Northridge (1,5)	Northridge (2,0)	Saratoga (1,0)	Saratoga (1,5)	Saratoga (2,0)
1	1,77	2,66	3,54	2,01	3,02	4,03	1,77	2,66	3,54
2	1,32	1,97	2,63	1,61	2,41	3,22	1,44	2,16	2,88
3	1,03	1,55	2,07	1,34	2,00	2,67	1,21	1,82	2,43
4	0,88	1,32	1,76	1,12	1,68	2,24	1,02	1,53	2,04
5	0,88	1,33	1,77	0,99	1,48	1,98	0,87	1,30	1,73
6	0,95	1,42	1,90	0,88	1,31	1,75	0,71	1,06	1,42
7	1,00	1,51	2,01	0,79	1,19	1,59	0,58	0,86	1,15
8	0,99	1,49	1,98	0,72	1,07	1,43	0,47	0,71	0,95
9	0,88	1,33	1,77	0,61	0,92	1,23	0,39	0,59	0,78
10	0,68	1,02	1,37	0,47	0,71	0,94	0,32	0,47	0,63
11	0,43	0,65	0,87	0,32	0,48	0,64	0,24	0,36	0,48
12	0,19	0,28	0,38	0,18	0,27	0,36	0,16	0,24	0,31

The structure deos not have any obvious irregularity, but the effect of dominant shear wall. On the other hand, this building has a possible design deficiency. This building is simply the same structure with the twelve story with full capacity design, added a shear wall at the middle. After inserting that shear wall, beams are not redesigned, which creates some deficiency.

Average plastic rotation demands per story from RSA and NRHA on columns, beams and shear walls are given in Figures 3.28, 3.29 and 3.30, respectively.

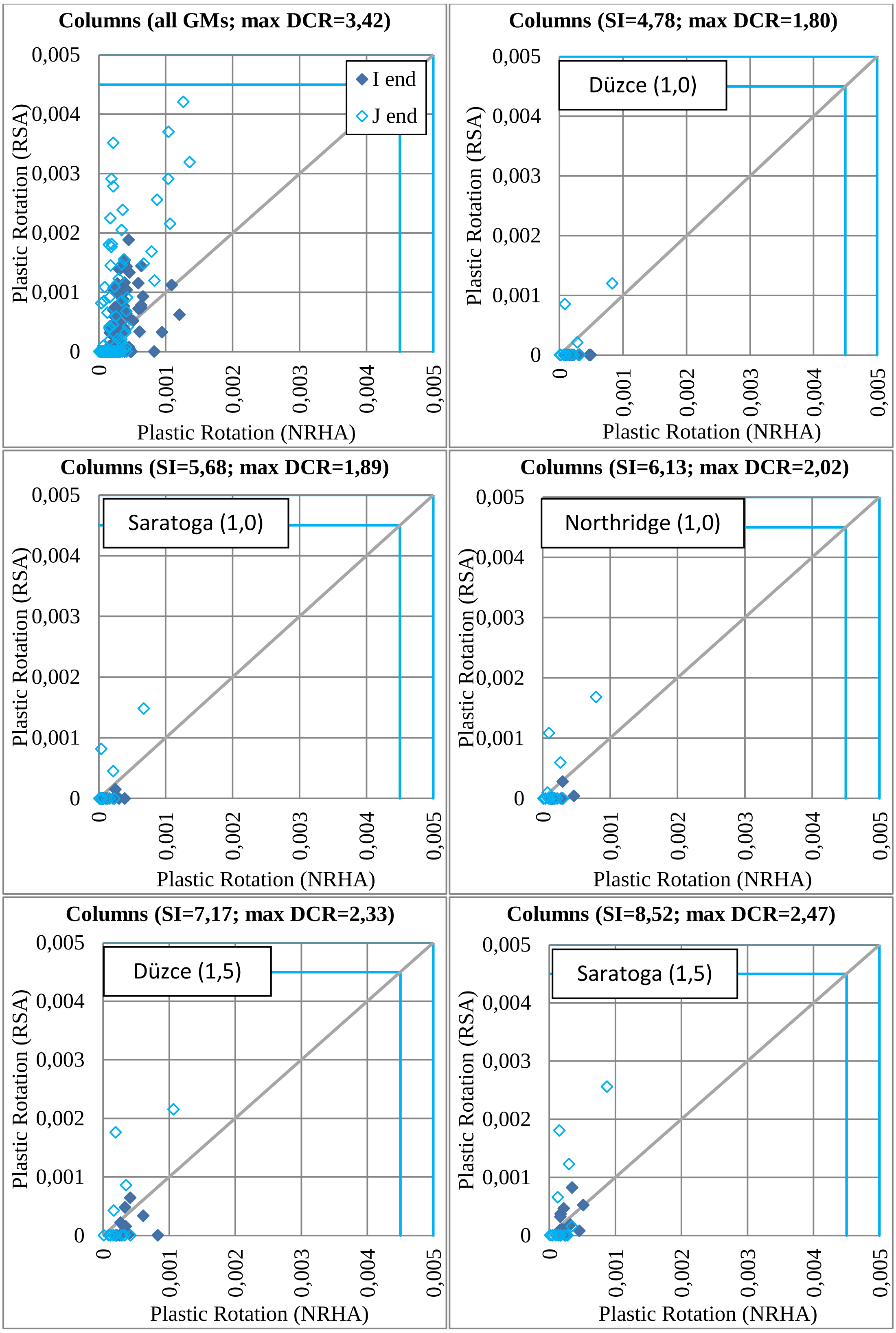


Figure 3.28: Average plastic rotation demands from RSA and NRHA for columns of 12 story R/C wall-frame

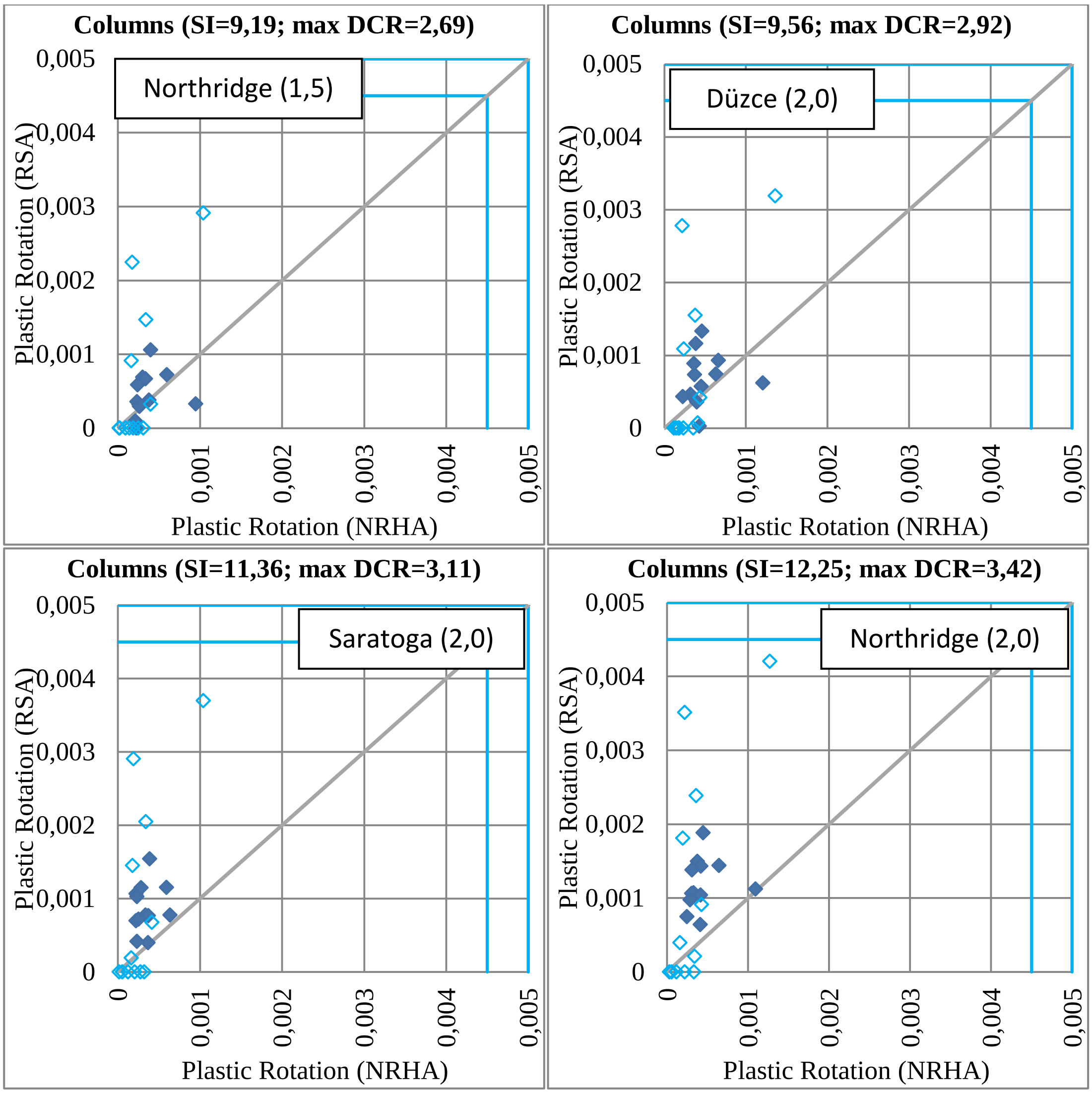


Figure 3.28 cont'd: Average plastic rotation demands from RSA and NRHA for columns of 12 story R/C wall-frame

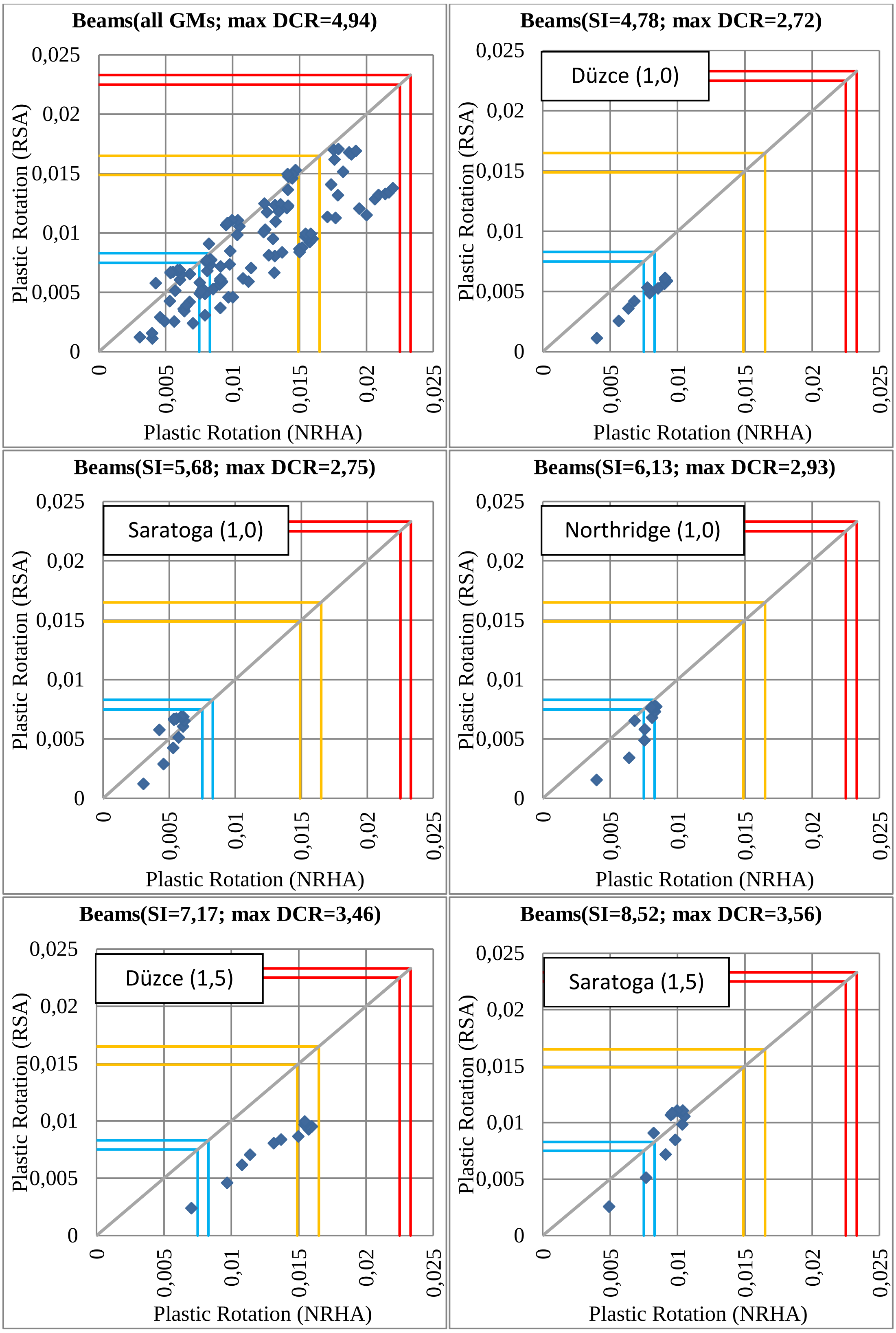


Figure 3.29: Average plastic rotation demands from RSA and NRHA for beams of 12 story R/C wall-frame

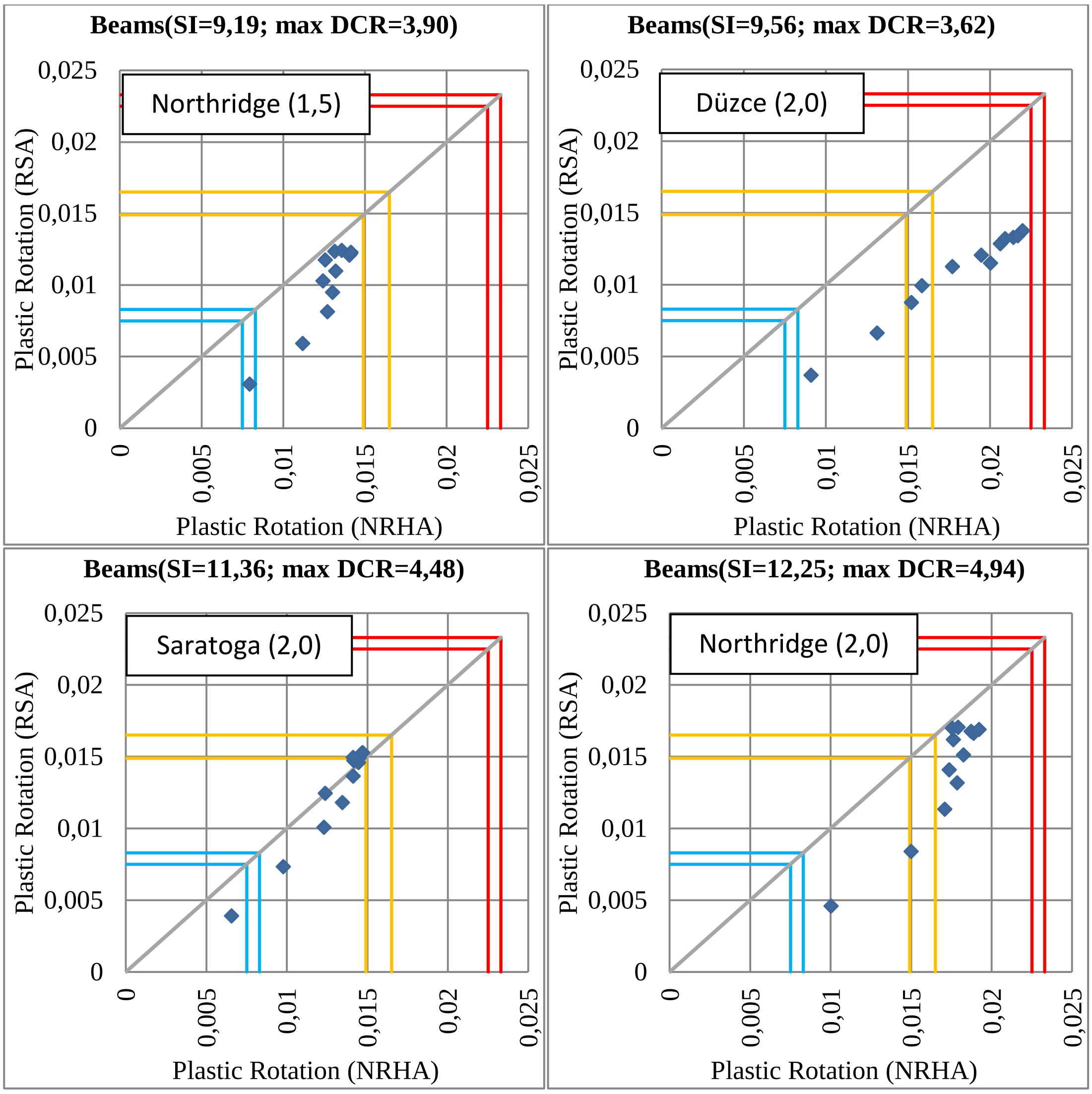


Figure 3.29 cont'd: Average plastic rotation demands from RSA and NRHA for beams of 12 story R/C wall-frame

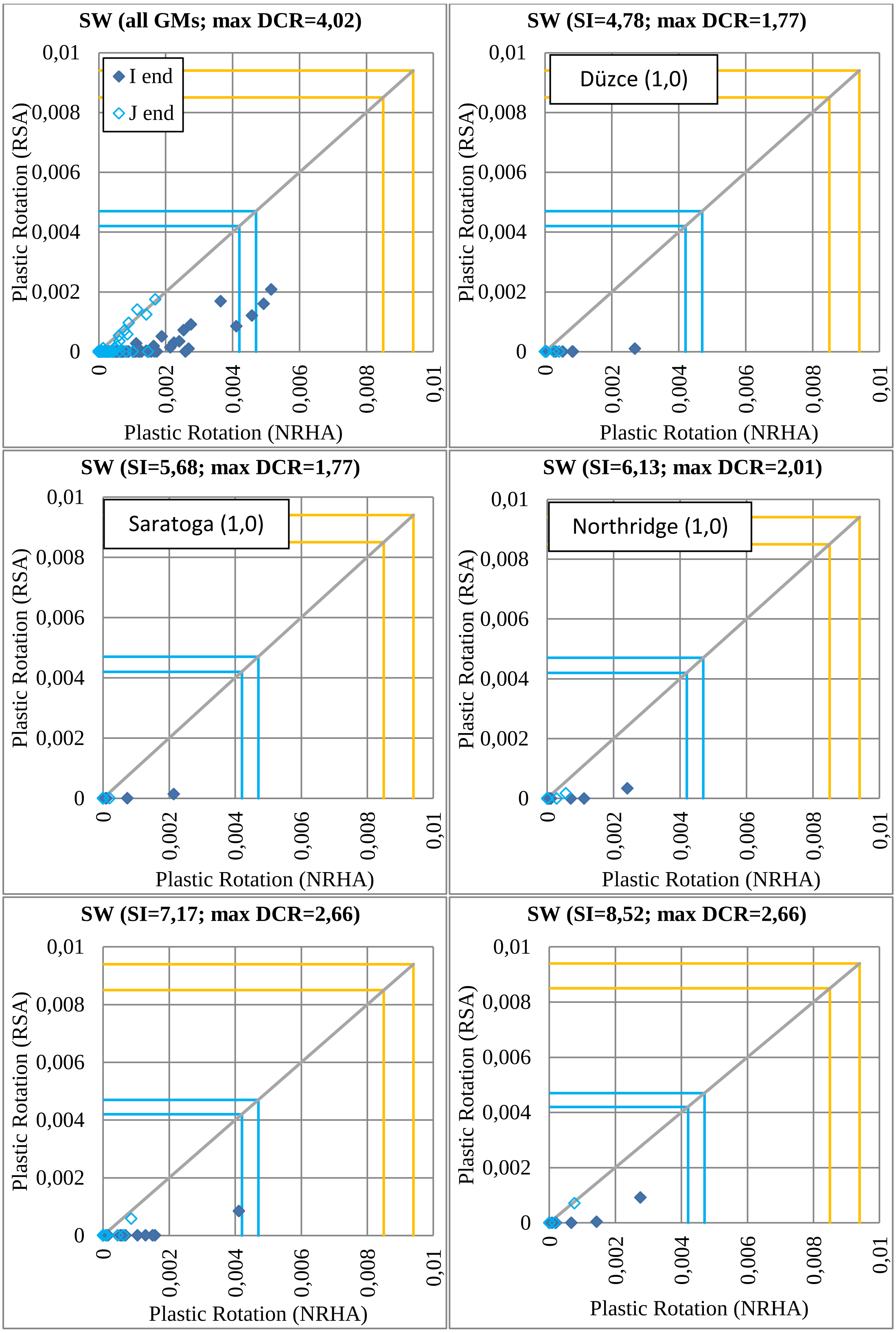


Figure 3.30: Average plastic rotation demands from RSA and NRHA for shear walls of 12 story R/C wall-frame

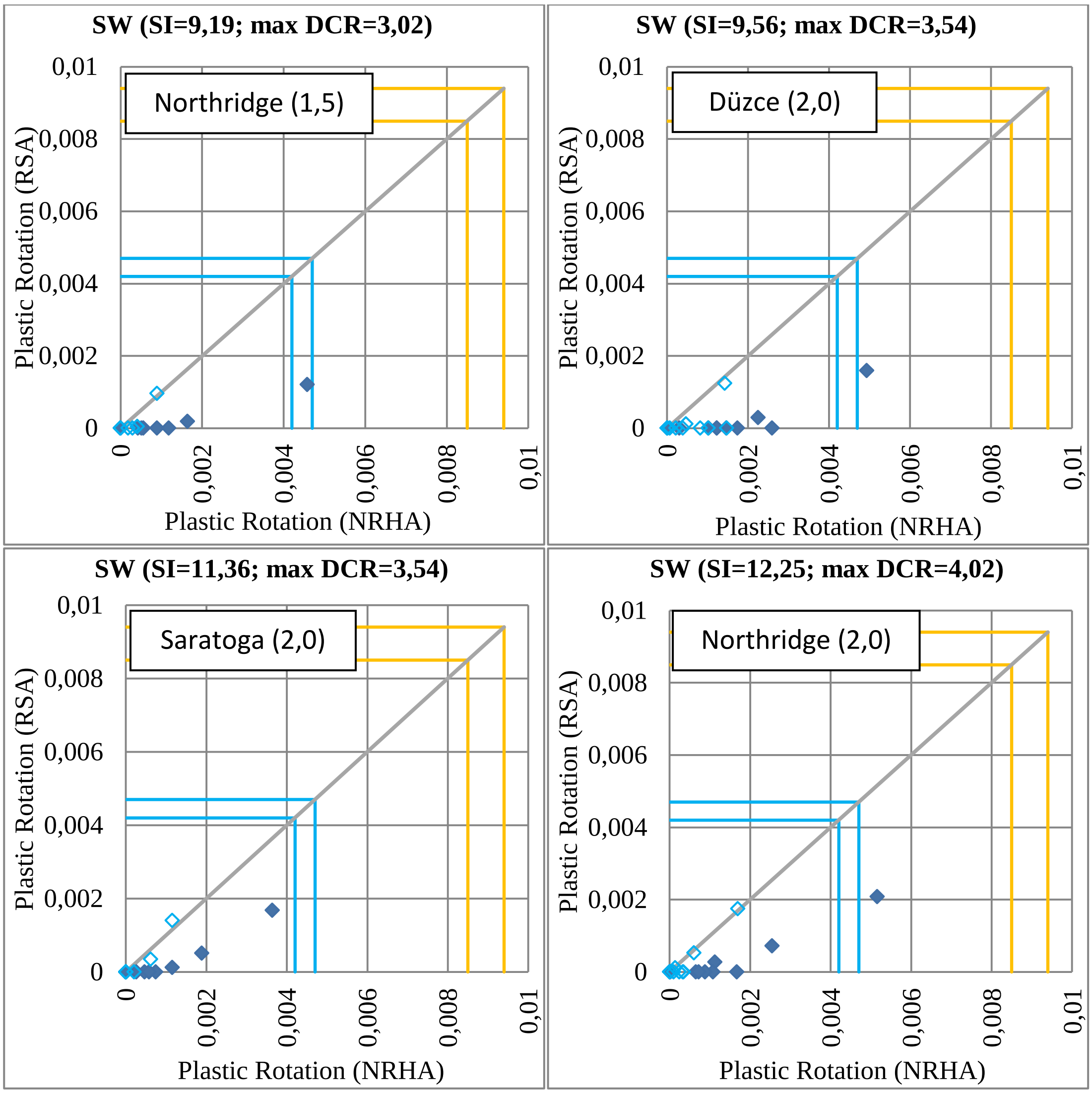


Figure 3.30 cont'd: Average plastic rotation demands from RSA and NRHA for shear walls of 12 story R/C wall-frame

The following discussions are done considering Figures 3.28 to 3.30 along with Table 3.18.

- The ratio of average shear DCR of a story to that of the adjacent story is not an effective measure to determine the applicibility of linear methods when the system is a wall-frame, and when shear walls are continous throughout the building height.
- Since shear wall is dominant in the seismic response of the structure, other vertical elements, namely columns, have very low deformation demands on them. Plastic rotation demands on columns are very low for both linear and nonlinear

methods. In addition to that, there is no obvious trend that demands from linear analysis become less accurate with increasing ground motion intensities. Therefore, it is safe to say that for structures that shear wall is very dominant, there may not be a limiting DCR value for column ends to employ linear procedures.

- After plastic hinge mechanism develops at the base of shear wall, responses between linear and nonlinear analysis change dramatically since deformed shape of plastic hinge mechanism cannot be estimated by linear analysis. This fact is the reason behind significant errors of linear analysis in Figure 3.30. For all ground motions, linear analysis failed to estimate deformation demand of shear wall. Although it is not safe to use linear method even for small intensities, since the real deformation demand is also very low, a maximum DCR limit of 2 can be assigned to shear walls in order to employ linear elastic analysis.
- The deformation demands on beams are not well estimated by linear methods especially for higher intensity ground motions. However, the error is not significant and a limit DCR of 2 for shear walls is a natural limiting condition also for beams.

3.7.2. Twenty Story R/C Wall-Frame with Capacity Design (R=7)

Shear wall in this building dominates the response of structure, similar to the previous case. About 95% of total base shear is carried by the shear wall. Therefore, only shear wall DCRs per story is given in Table 3.19.

Table 3.19: DCRs of shear wall members of 20 story R/C wall-frame with capacity design

	GM name (scale)								
story	Düzce (1,0)	Düzce (1,5)	Düzce (2,0)	Northridge (1,0)	Northridge (1,5)	Northridge (2,0)	Saratoga (1,0)	Saratoga (1,5)	Saratoga (2,0)
1	2,00	3,01	4,01	1,67	2,50	3,34	1,40	2,10	2,80
2	1,46	2,18	2,91	1,21	1,81	2,42	1,06	1,59	2,11
3	1,19	1,79	2,39	0,97	1,46	1,95	0,90	1,34	1,79
4	0,96	1,45	1,93	0,75	1,12	1,50	0,72	1,09	1,45
5	0,86	1,28	1,71	0,63	0,94	1,25	0,61	0,91	1,21
6	0,89	1,34	1,78	0,64	0,96	1,29	0,57	0,86	1,15
7	0,89	1,34	1,78	0,68	1,02	1,36	0,54	0,81	1,08
8	0,85	1,28	1,70	0,72	1,08	1,43	0,51	0,77	1,02
9	0,88	1,32	1,76	0,83	1,25	1,67	0,56	0,83	1,11
10	0,83	1,24	1,66	0,86	1,29	1,72	0,56	0,83	1,11
11	0,81	1,21	1,62	0,86	1,30	1,73	0,55	0,83	1,10
12	0,94	1,41	1,88	0,97	1,45	1,93	0,61	0,92	1,22
13	0,98	1,47	1,96	0,92	1,38	1,84	0,58	0,87	1,16
14	1,00	1,50	2,00	0,84	1,26	1,69	0,53	0,80	1,07
15	1,12	1,68	2,25	0,86	1,29	1,71	0,56	0,84	1,12
16	1,01	1,52	2,03	0,71	1,07	1,42	0,49	0,74	0,99
17	0,83	1,25	1,67	0,55	0,83	1,10	0,42	0,63	0,84
18	0,82	1,23	1,65	0,53	0,79	1,05	0,45	0,68	0,91
19	0,52	0,77	1,03	0,34	0,51	0,68	0,32	0,49	0,65
20	0,26	0,39	0,52	0,20	0,30	0,40	0,19	0,28	0,38

The structure is designed considering capacity design principles, which rules out almost all irregularities possible.

Average plastic rotation demands per story from RSA and NRHA on columns, beams and shear walls are given in Figures 3.28, 3.29 and 3.30, respectively.

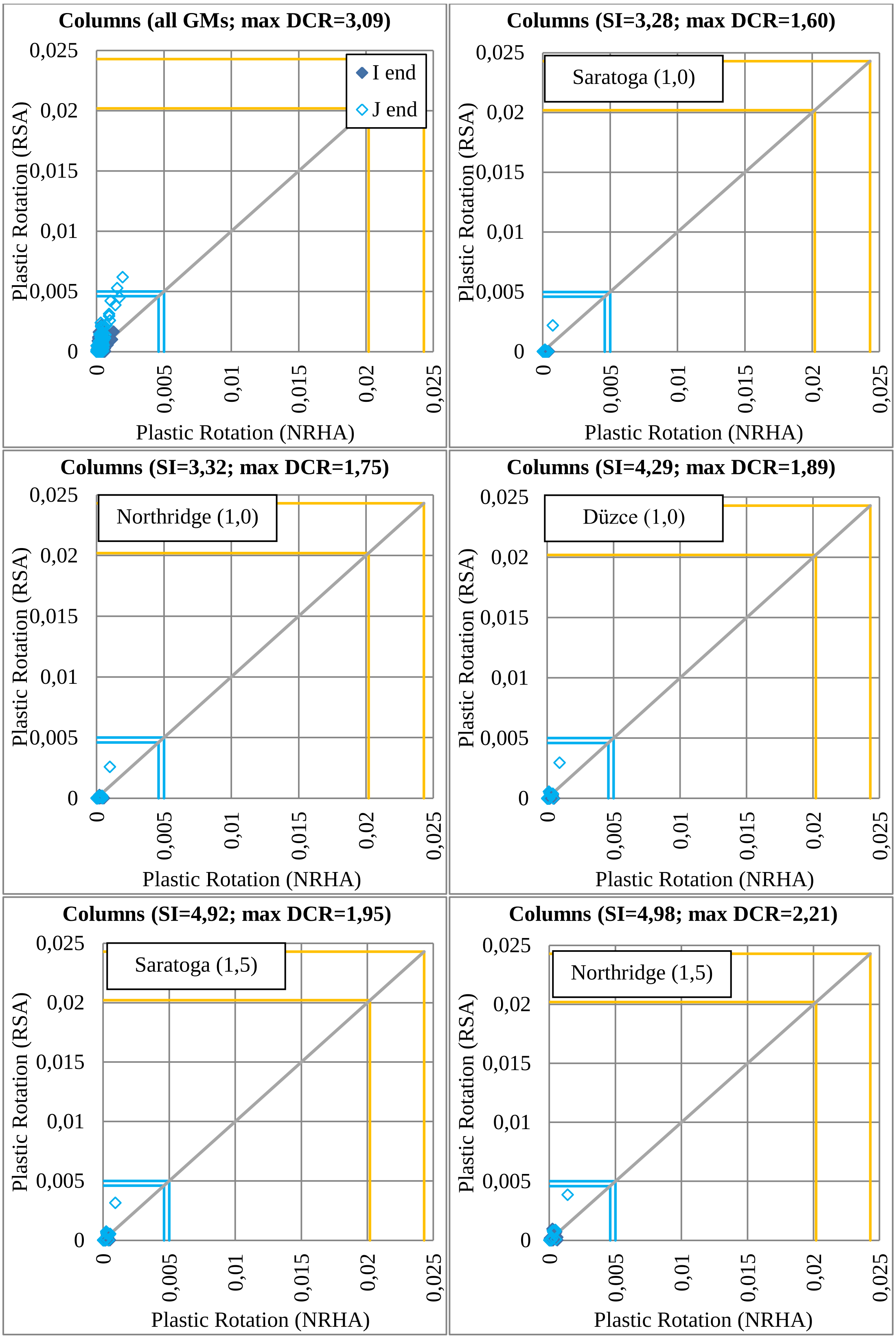


Figure 3.31: Average plastic rotation demands from RSA and NRHA for columns of 20 story R/C wall-frame with capacity design

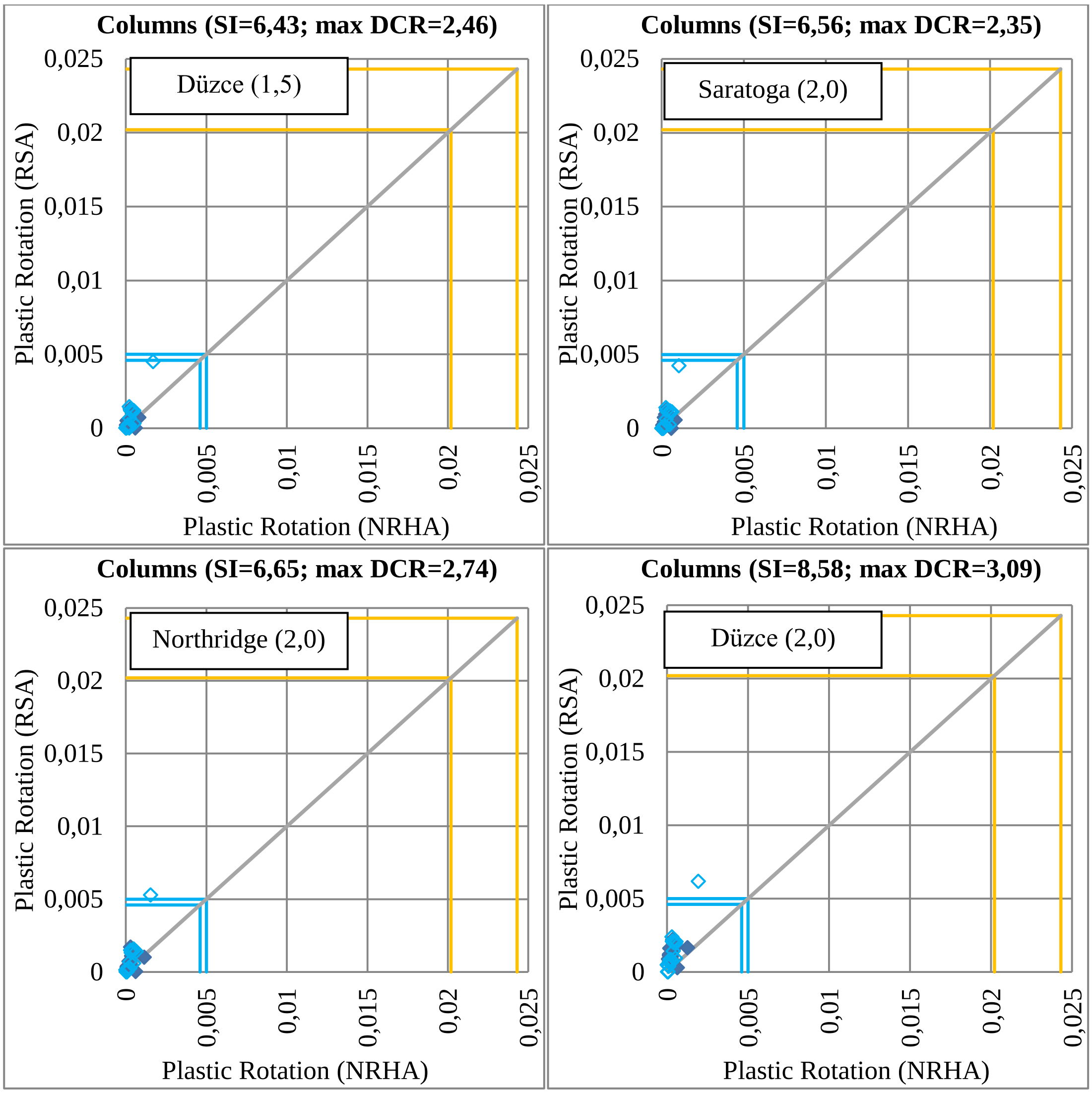


Figure 3.31 cont'd: Average plastic rotation demands from RSA and NRHA for columns of 20 story R/C wall-frame with capacity design

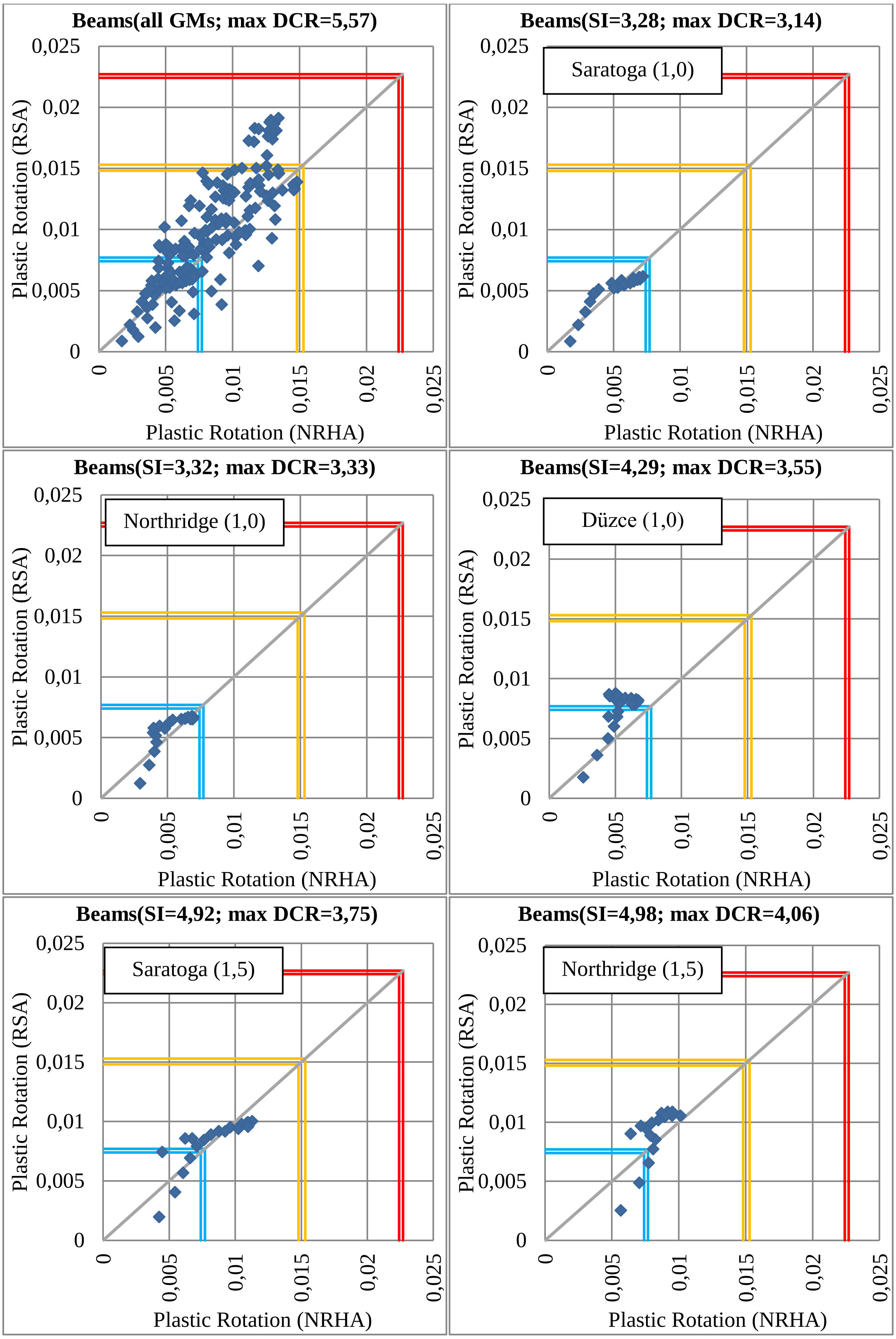


Figure 3.32: Average plastic rotation demands from RSA and NRHA for beams of 20 story R/C wall-frame with capacity design

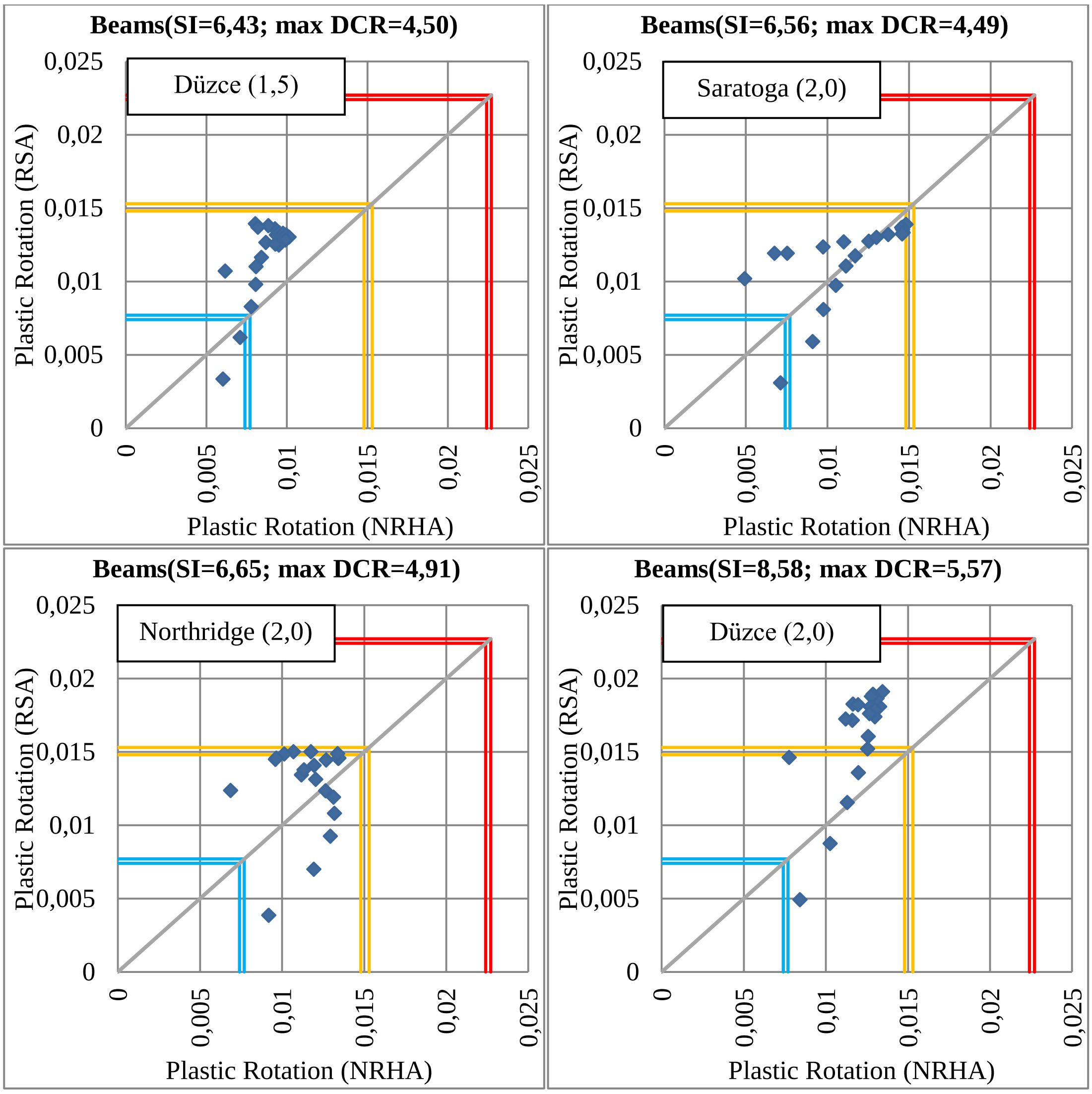


Figure 3.32 cont'd: Average plastic rotation demands from RSA and NRHA for beams of 20 story R/C wall-frame with capacity design

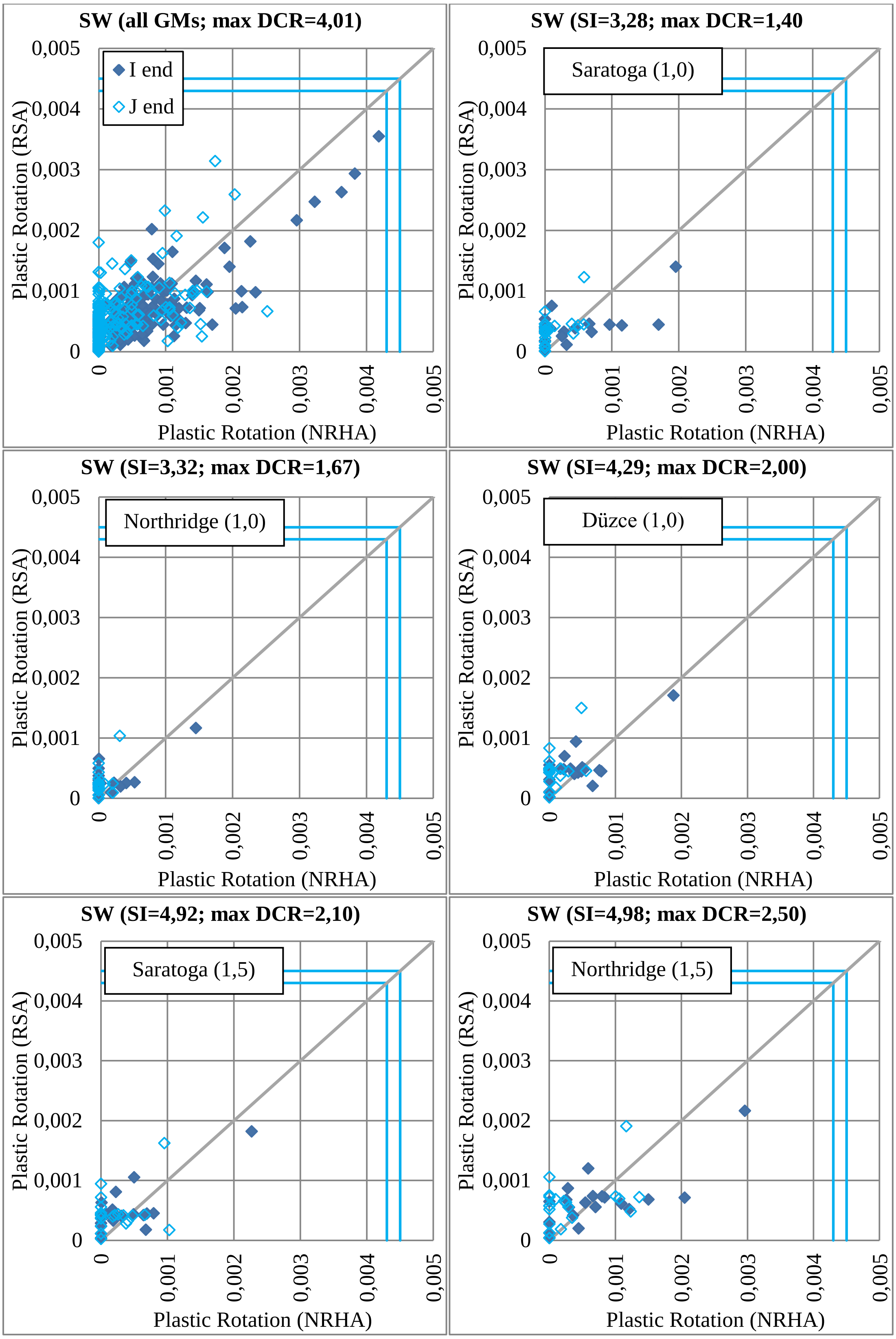


Figure 3.33: Average plastic rotation demands from RSA and NRHA for shear walls of 20 story R/C wall-frame with capacity design

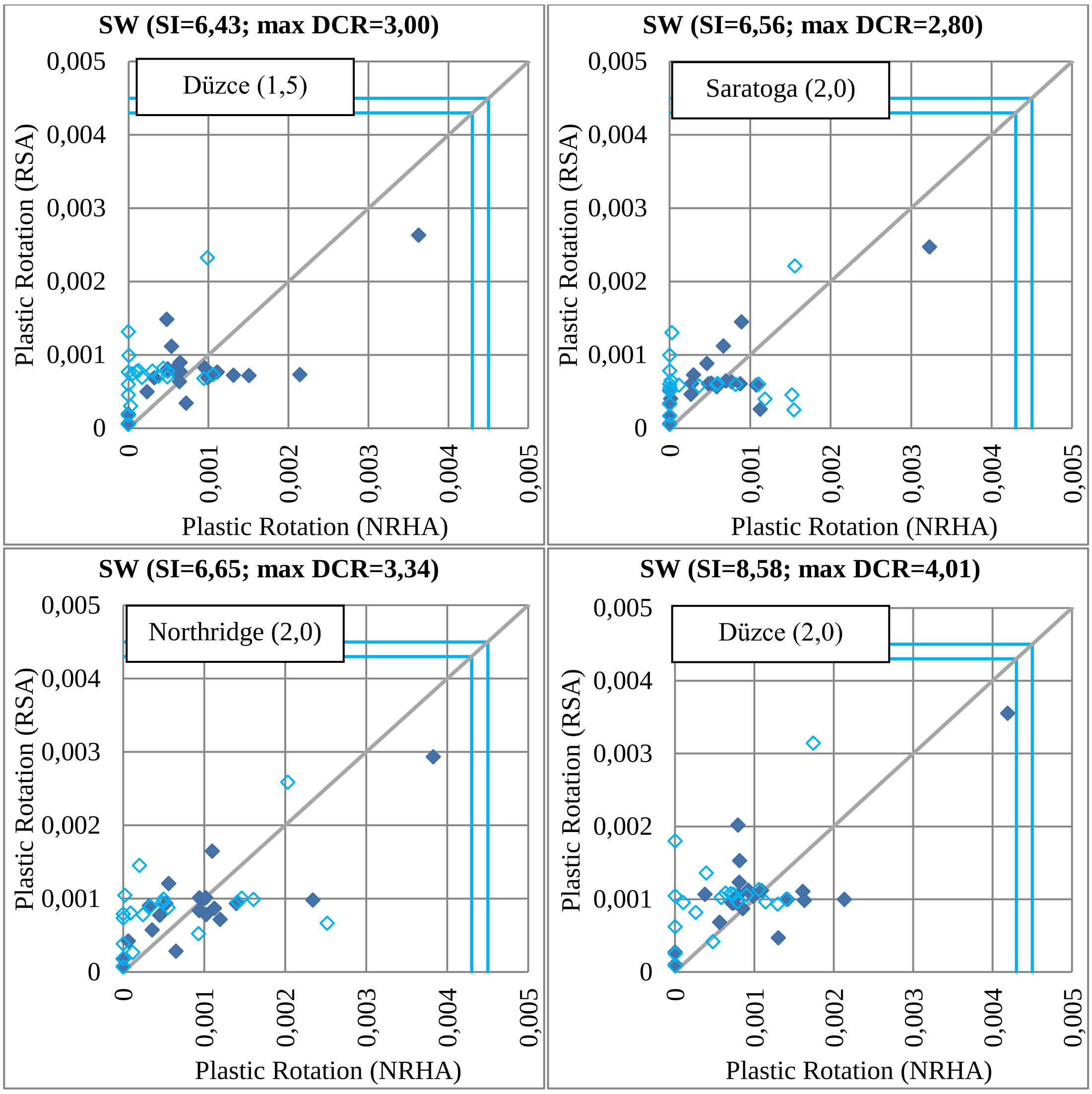


Figure 3.33: Average plastic rotation demands from RSA and NRHA for shear walls of 20 story R/C wall-frame with capacity design

The following conclusions are derived from Figures 3.31 to 3.33 along with Table 3.19.

- Similar to previous case, since shear wall is very dominant, deformation demands on columns are very low. Column behaviour is not governing for this structure. Therefore, no limitation is needed to set up considering column DCRs.
- Beam responses from linear and nonlinear analyses are consistent for all ground motions. For high intensities, they start to seperate from each other, but this seperation is in favor of linear analysis, which is the safe side. Similar to previos case, there is no limiting DCR value set up for beams for this building, too.

- For small intensities, the shear wall almost remained elastic, with very small plastic rotation demands for both linear and nonlinear analyses. Since the deformations are very small, it is not logical to further observe low intensity ground motions. As the ground motion gets stronger, the divergence of results from linear and nonlinear analyses increases as well as the unsafe results of linear analysis. After plastic hinge mechanisms is fully developed at the shear wall, linear method could not estimate the accurate response of the elements, which is very similar for the previous case strudy building. It is observed that after maximum DCR of shear wall base exceeds 2, linear analysis results are not safe for this structure.

3.7.3. Four Story Retrofitted School Building (R=4)

There are massive shear walls in this building. About 84% of total base shear is carried by the shear walls, which makes the response dependent on wall behaviour. Average shear wall DCRs per story is given in Table 3.20. The average calculated is a normalized DCR with respect to shear force demands on individual shear walls.

Table 3.20: Average DCRs of shear wall members of 4 story retrofitted building

	GM name (scale)								
story	Düzce (1,0)	Düzce (1,5)	Düzce (2,0)	Northridge (1,0)	Northridge (1,5)	Northridge (2,0)	Saratoga (1,0)	Saratoga (1,5)	Saratoga (2,0)
1	2,54	3,78	5,03	1,71	2,55	3,39	2,21	3,30	4,38
2	1,61	2,41	3,20	1,08	1,62	2,16	1,41	2,11	2,80
3	0,92	1,37	1,82	0,62	0,92	1,22	0,79	1,18	1,57
4	0,30	0,44	0,59	0,21	0,29	0,38	0,23	0,33	0,43

There are new and old shear walls in this structure along with unsatisfactory beams and columns. Original elements of building does not satisfy earthquake resistant design criteria, but with addition of a lot of shear walls, the structure is aimed to behave mostly in linear elastic range under ground motion effect. Because of complex system of this structure, there are surely deficincies that effect the response. Average plastic rotation demands of all elements from RSA and PO on are given in Figures 3.34, 3.35 and 3.36.

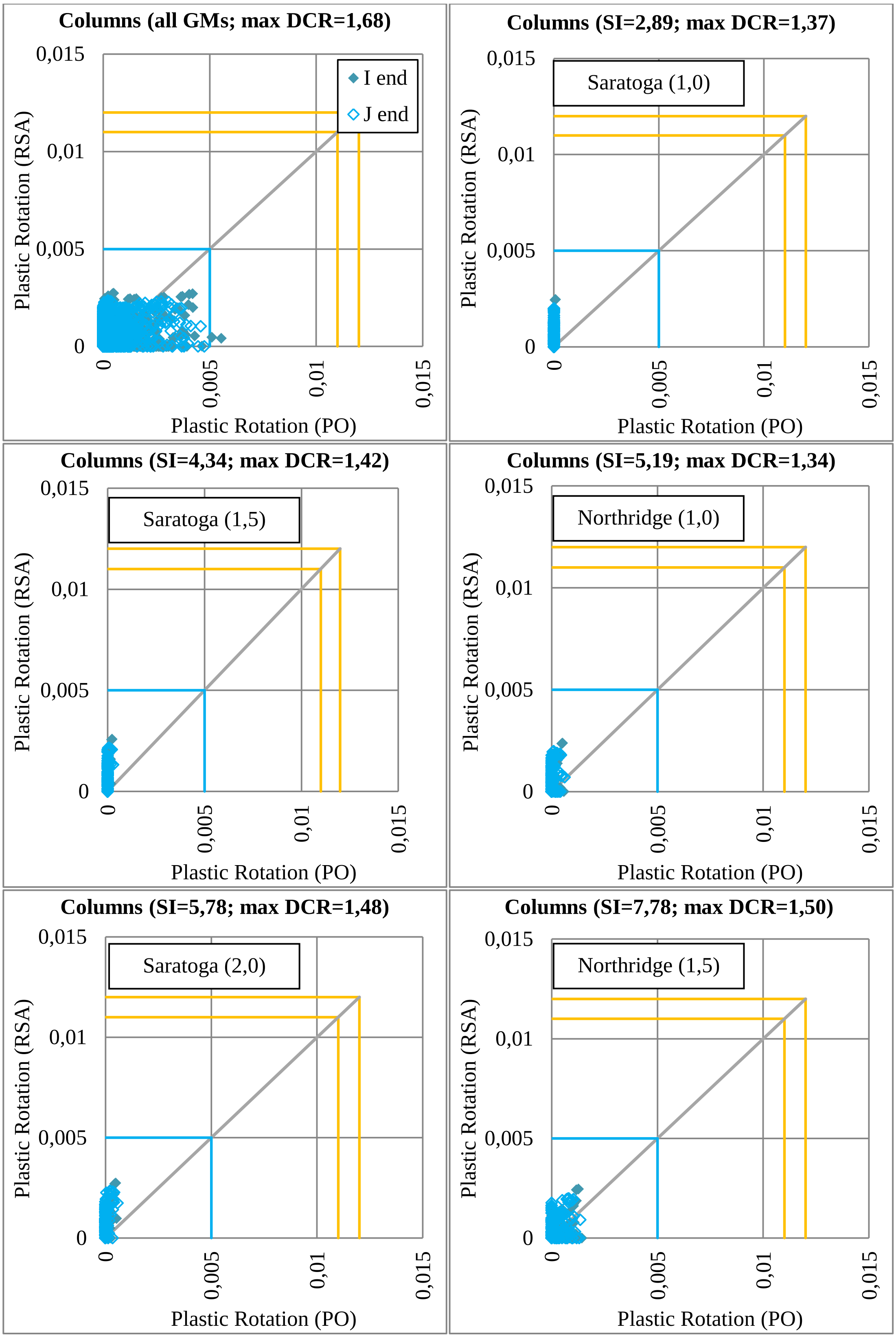


Figure 3.34: Plastic rotation demands from RSA and PO for all columns of 4 story retrofitted school building

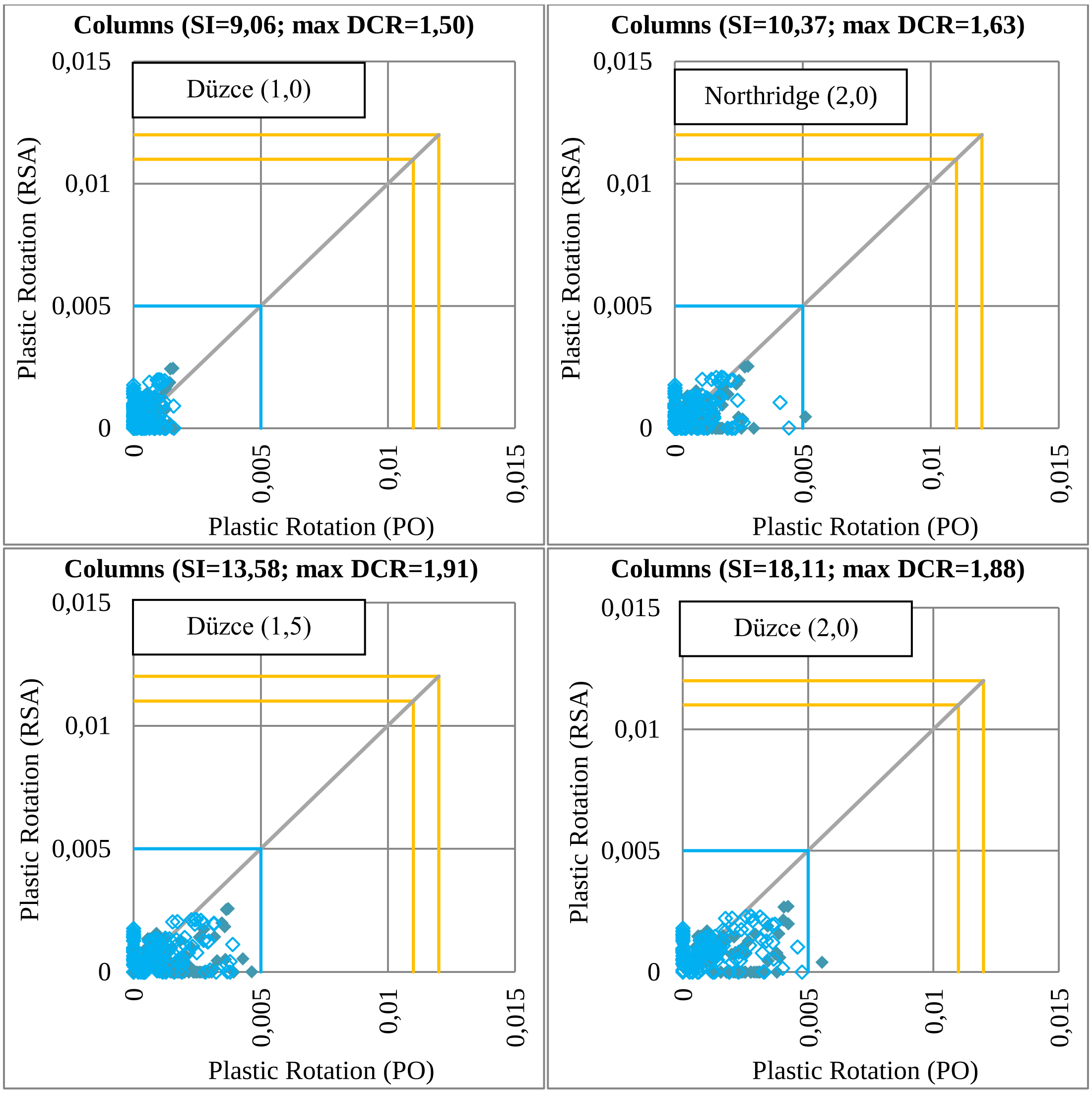


Figure 3.34 cont'd: Plastic rotation demands from RSA and PO for all columns of 4 story retrofitted school building

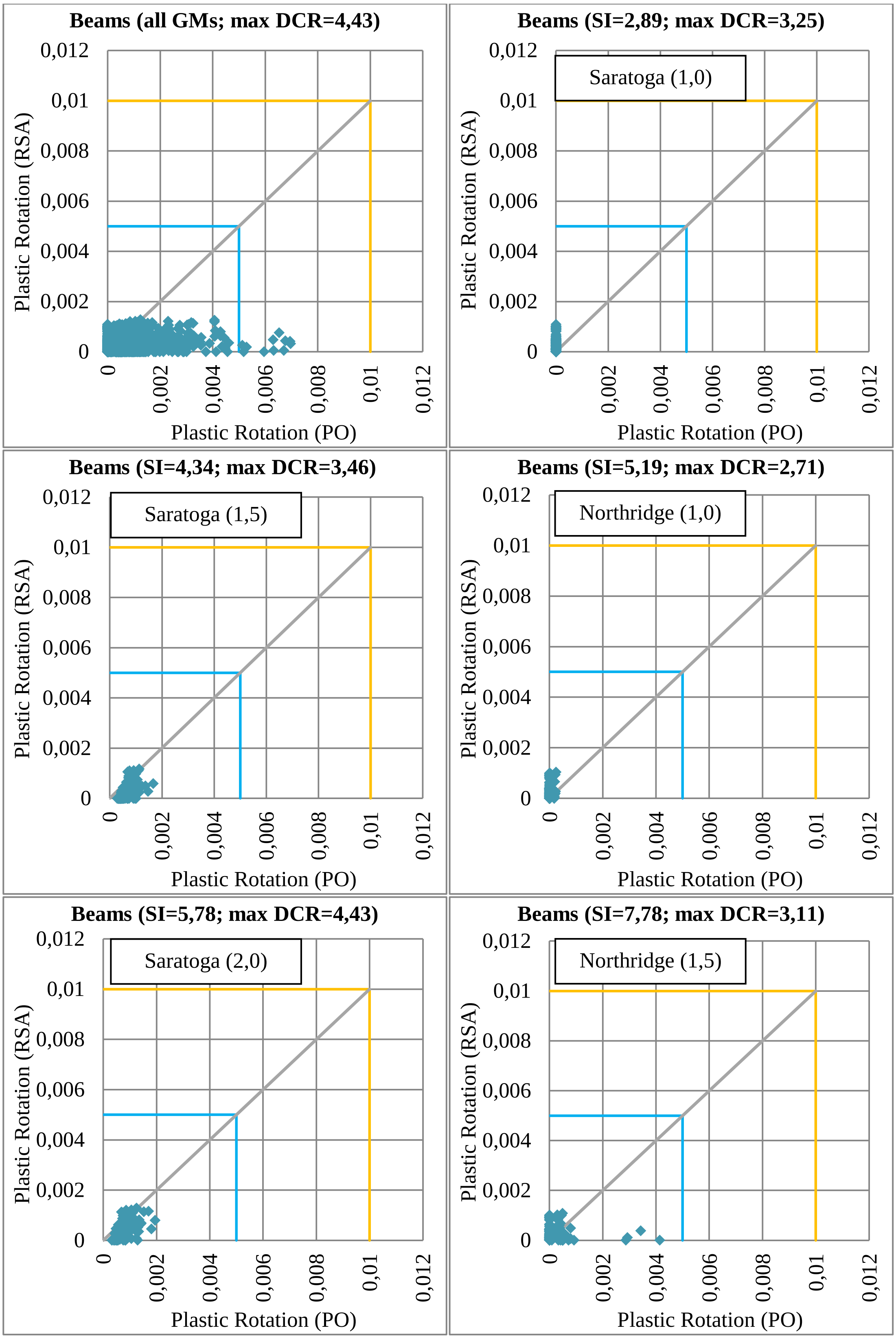


Figure 3.35: Plastic rotation demands from RSA and PO for all beams of 4 story retrofitted school building

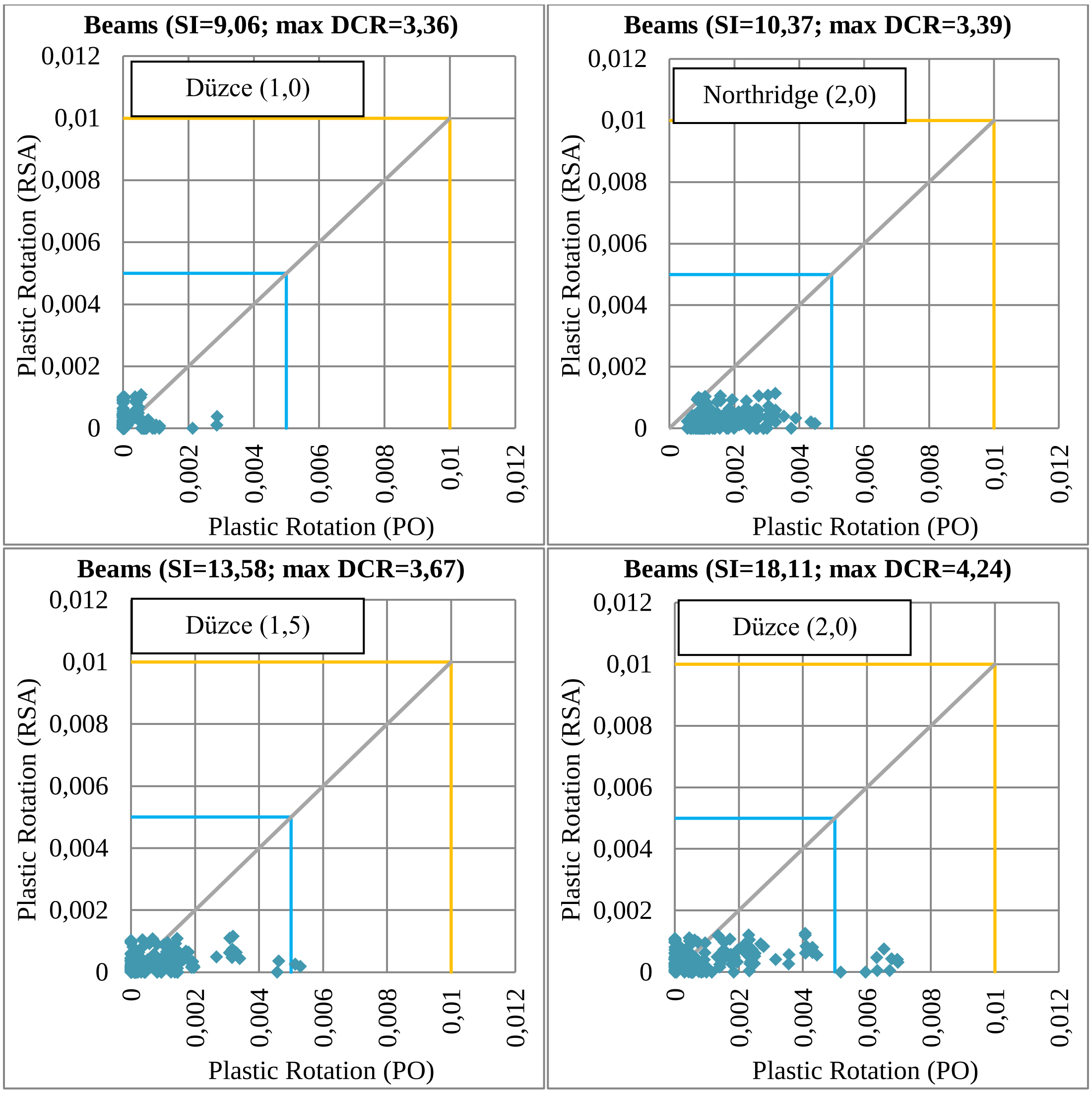


Figure 3.35 cont’d: Plastic rotation demands from RSA and PO for all beams of 4 story retrofitted school building

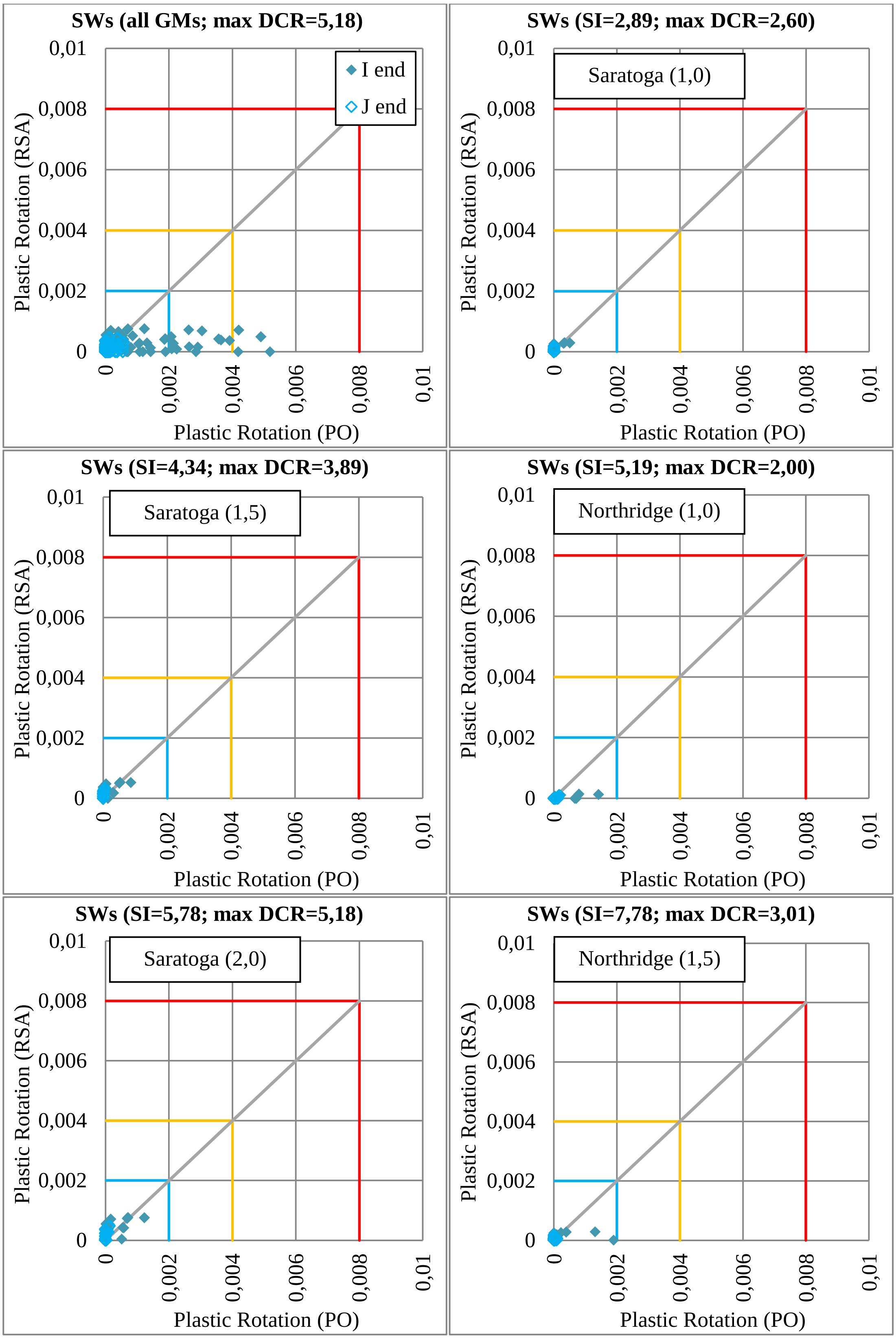


Figure 3.36: Plastic rotation demands from RSA and PO for all shear walls of 4 story retrofitted school building

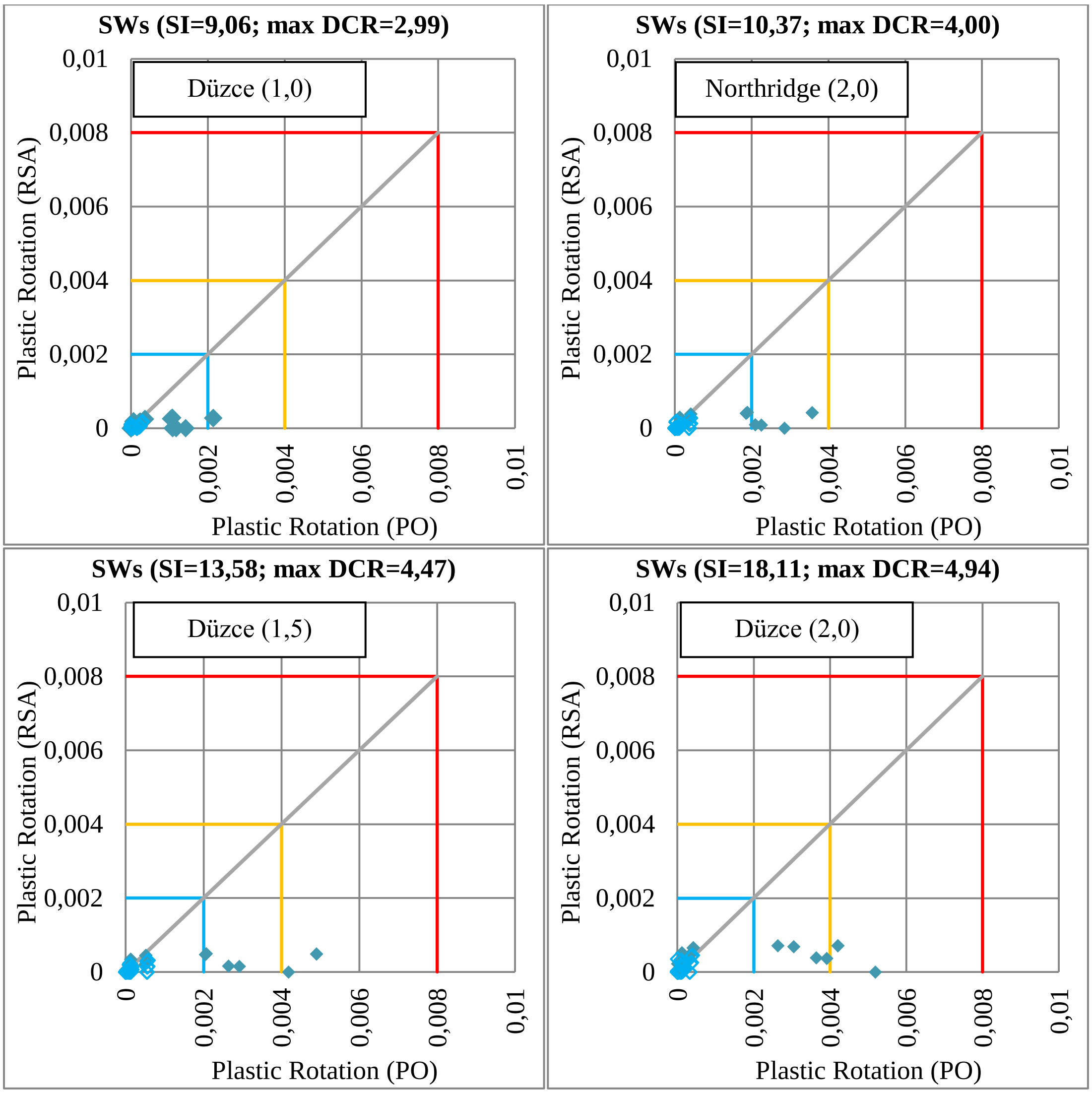


Figure 3.36 cont'd: Plastic rotation demands from RSA and PO for all shear walls of 4 story retrofitted school building

The following conclusions are derived from Figures 3.34 to 3.36 along with Table 3.20.

- After yielding at the base of shear walls start, linear method is failing to capture the response entirely for shear walls. For Saratoga ground motion, linear and nonlinear methods are consistent, but for other two, linear method is completely unsafe. Similar to previous structures with shear walls, linear method could not estimate the behavior of the structure under the effect of shear walls. However, for small intensities since the deformation demand is low in general, linear methods can be applicable. Therefore, for this building, maximum DCR for shear walls to employ linear method can be set as 2.

- Although shear walls are the main earthquake resistant structural elements for this building, excessive amount of plastic rotations are observed in columns, too. However, setting a DCR limit for shear walls to 2, no additional measure is needed to limit column DCRs because for the ground motions that are satisfying that limit, plastic deformation demands on columns are also safe for linear analysis.
- Responses obtained for beams are very similar to columns. For small ground motion intensities, results from linear and nonlinear analyses matches well, but with the increasing intensity, linear method completely underestimates the plastic rotation demands at beam ends. Therefore, the same condition explained above is valid for beams, too.

3.8. Structures with Torsional Strength Irregularity

There are two buildings in this section. They are six story 3D R/C building with capacity design and eight story 3D R/C building. Typical plan view of the inelastic deformation pattern of a frame system with torsional strength irregularity is shown in Figure 3.37. The flexible side frames undergo larger inelastic deformations compared to the stiff side frames.

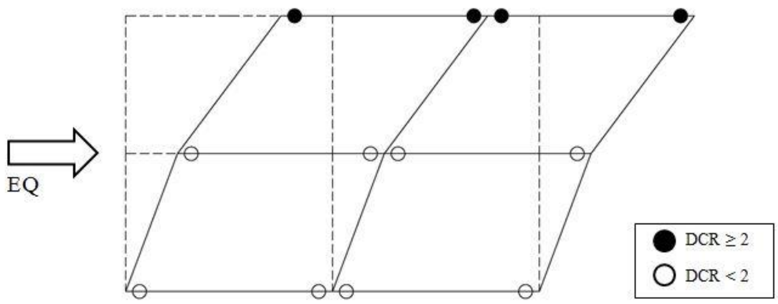


Figure 3.37: Typical plan view of the inelastic deformation pattern of a frame system with torsional strength irregularity.

3.8.1. Six Story 3D R/C Building with Capacity Design (R=8)

Torsion is very effective and a dominant factor in the response of this building. In order to check torsional irregularity, maximum beam and column DCRs, of each individual frames are calculated and given in Table 3.21. The ratios of the maximum to minimum DCRs for columns and beams at each story are also provided in Table 3.21. Another ratio calculated in this table is the ratio of the maximum frame DCR of one side of the center of resistance to the maximum frame DCR of the other side, and this ratio is called *k* for simplicity. This table is prepared for minimum (1) and maximum (2) scales of each ground motions, and it is observed that GM scale is not effective for the ratios calculated.

Table 3.21: Maximum beam and column DCRs for each frame of 6 story 3D frame with capacity design

	column						beam						GM name (scale)
story	Se	Si	Fi	Fe	max/min DCR	k	Se	Si	Fi	Fe	max/min DCR	k	
1	2,59	1,72	2,07	3,51	**2,04**	**1,36**	4,98	4,00	2,50	3,45	**1,99**	**1,44**	Düzce (1,0)
2	1,72	1,23	1,30	2,31	**1,89**	**1,34**	6,59	3,09	2,94	3,04	**2,24**	**2,16**	
3	1,56	0,93	1,24	1,89	**2,03**	**1,21**	5,95	2,77	2,69	2,71	**2,22**	**2,20**	
4	1,43	0,94	1,20	1,91	**2,04**	**1,34**	3,73	3,28	1,95	2,74	**1,91**	**1,36**	
5	0,96	0,93	0,90	1,70	**1,89**	**1,77**	2,94	2,90	1,61	2,31	**1,82**	**1,27**	
6	0,93	0,83	0,86	1,30	**1,57**	**1,40**	3,50	1,50	1,70	1,25	**2,80**	**2,06**	
1	5,17	3,43	4,13	7,01	**2,04**	**1,36**	9,32	6,58	4,69	6,31	**1,99**	**1,48**	Düzce (2,0)
2	3,31	2,21	2,49	4,43	**2,00**	**1,34**	11,31	5,62	5,01	5,85	**2,26**	**1,93**	
3	2,82	1,80	2,24	3,73	**2,07**	**1,32**	9,95	4,94	4,47	5,17	**2,23**	**1,92**	
4	2,57	1,70	2,17	3,70	**2,17**	**1,44**	6,67	4,94	3,49	4,77	**1,91**	**1,40**	
5	1,81	1,43	1,72	3,06	**2,13**	**1,69**	4,85	3,94	2,71	3,77	**1,79**	**1,29**	
6	1,28	1,08	1,27	2,08	**1,94**	**1,62**	4,17	1,96	2,11	2,00	**2,13**	**1,98**	
1	5,22	3,24	3,54	5,95	**1,83**	**1,14**	9,53	6,38	4,17	5,56	**2,29**	**1,71**	Northridge (1,0)
2	3,44	2,18	2,26	3,98	**1,83**	**1,16**	11,73	5,58	4,64	5,30	**2,53**	**2,21**	
3	2,90	1,74	1,98	3,23	**1,86**	**1,11**	10,28	4,86	4,09	4,61	**2,51**	**2,23**	
4	2,46	1,51	1,72	2,85	**1,90**	**1,16**	6,61	4,70	2,94	4,01	**2,25**	**1,65**	
5	1,56	1,19	1,16	2,11	**1,81**	**1,35**	4,57	3,64	2,12	2,95	**2,15**	**1,55**	
6	1,13	0,93	0,93	1,43	**1,54**	**1,26**	4,05	1,77	1,85	1,49	**2,71**	**2,19**	
1	6,96	4,32	4,72	7,93	**1,83**	**1,14**	12,56	8,15	5,48	7,27	**2,29**	**1,73**	Northridge (2,0)
2	4,56	2,85	2,99	5,26	**1,84**	**1,15**	15,20	7,31	5,97	7,02	**2,55**	**2,17**	
3	3,80	2,31	2,58	4,30	**1,86**	**1,13**	13,24	6,34	5,23	6,09	**2,53**	**2,17**	
4	3,21	1,96	2,24	3,77	**1,92**	**1,18**	8,63	5,82	3,82	5,15	**2,26**	**1,68**	
5	2,05	1,46	1,52	2,70	**1,84**	**1,32**	5,83	4,30	2,68	3,68	**2,18**	**1,59**	
6	1,33	1,04	1,09	1,72	**1,65**	**1,30**	4,54	2,04	2,05	1,83	**2,47**	**2,22**	
1	1,46	1,41	2,20	3,85	**2,74**	**2,63**	3,25	3,60	2,72	3,82	**1,40**	**1,06**	Saratoga (1,0)
2	1,08	1,08	1,46	2,66	**2,47**	**2,46**	4,85	2,73	3,22	3,54	**1,78**	**1,37**	
3	1,07	0,78	1,33	2,11	**2,69**	**1,96**	4,54	2,46	2,90	3,09	**1,85**	**1,47**	
4	0,96	0,76	1,15	1,86	**2,45**	**1,94**	2,59	3,04	1,98	2,84	**1,53**	**1,07**	
5	0,56	0,79	0,73	1,44	**2,58**	**1,83**	2,24	2,72	1,51	2,20	**1,80**	**1,24**	
6	0,79	0,76	0,75	1,10	**1,46**	**1,38**	3,33	1,42	1,66	1,14	**2,92**	**2,01**	
1	2,92	2,79	4,40	7,69	**2,75**	**2,63**	5,57	5,65	5,14	7,09	**1,38**	**1,25**	Saratoga (2,0)
2	1,95	1,89	2,82	5,14	**2,72**	**2,63**	7,27	4,84	5,66	6,87	**1,50**	**1,06**	
3	1,73	1,50	2,43	4,18	**2,79**	**2,41**	6,53	4,23	4,95	5,95	**1,54**	**1,10**	
4	1,51	1,31	2,06	3,57	**2,73**	**2,37**	4,03	4,26	3,56	4,97	**1,40**	**1,17**	
5	0,96	1,09	1,35	2,47	**2,58**	**2,27**	3,06	3,40	2,47	3,49	**1,41**	**1,03**	
6	0,93	0,88	1,01	1,59	**1,80**	**1,70**	3,57	1,67	1,96	1,72	**2,14**	**1,82**	

The specified *k* limit for torsional strength irregularity is 1.5 in ASCE41. When the *k* values are considered in Table 3.21, torsional strength irregularity exists in almost all stories for all ground motions, except the Saratoga ground motions. Moreover, there is weak story irregularity under all ground motions. According to ASCE 41, linear methods are not applicable if there is any irregularity.

Average plastic rotation demands of each individual frames from RSA and NRHA on columns and beams are shown in Figures 3.38 and 3.39, respectively.

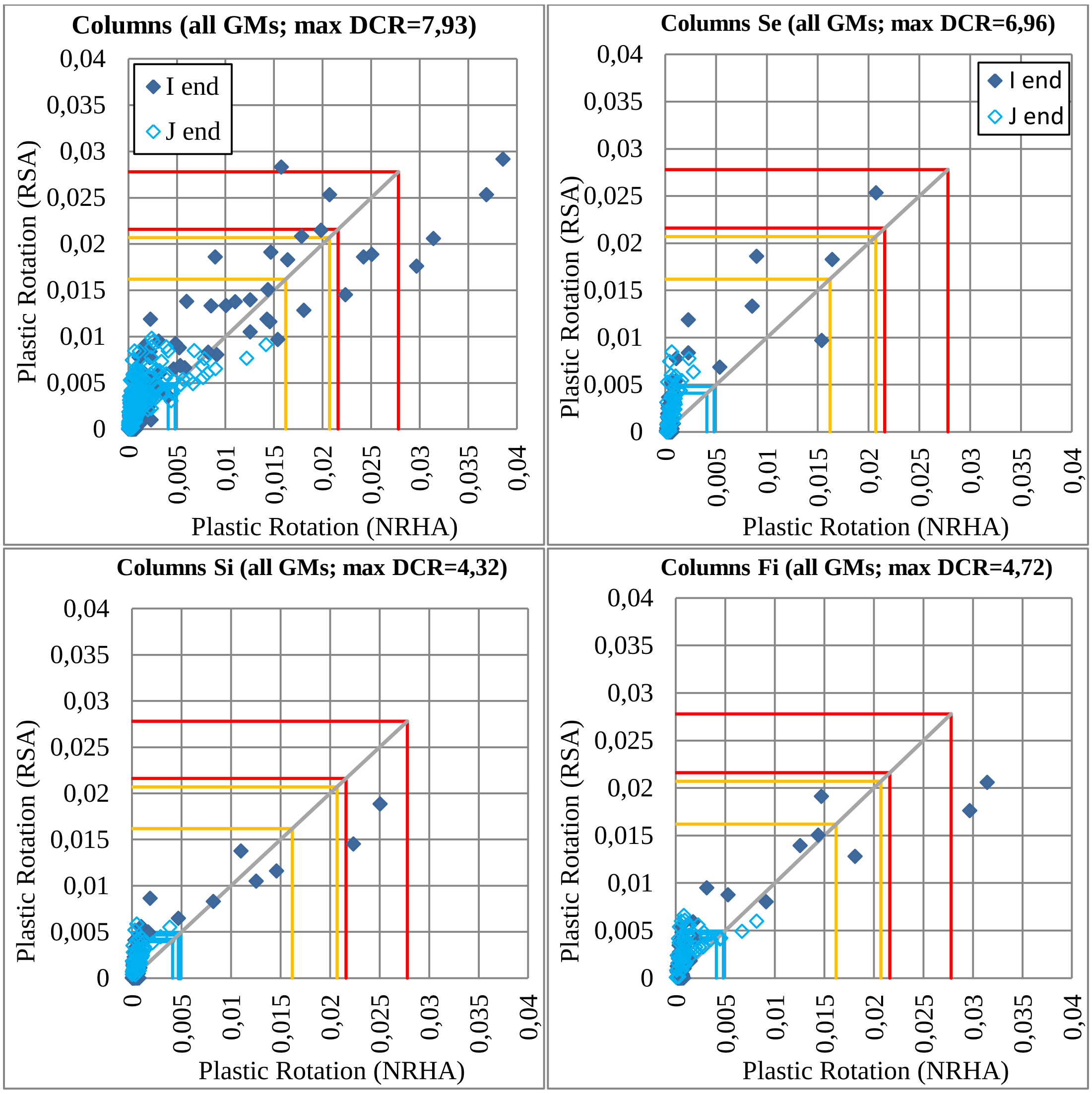


Figure 3.38: Average plastic rotation demands from RSA and NRHA for columns of 6 story 3D R/C frame with capacity design

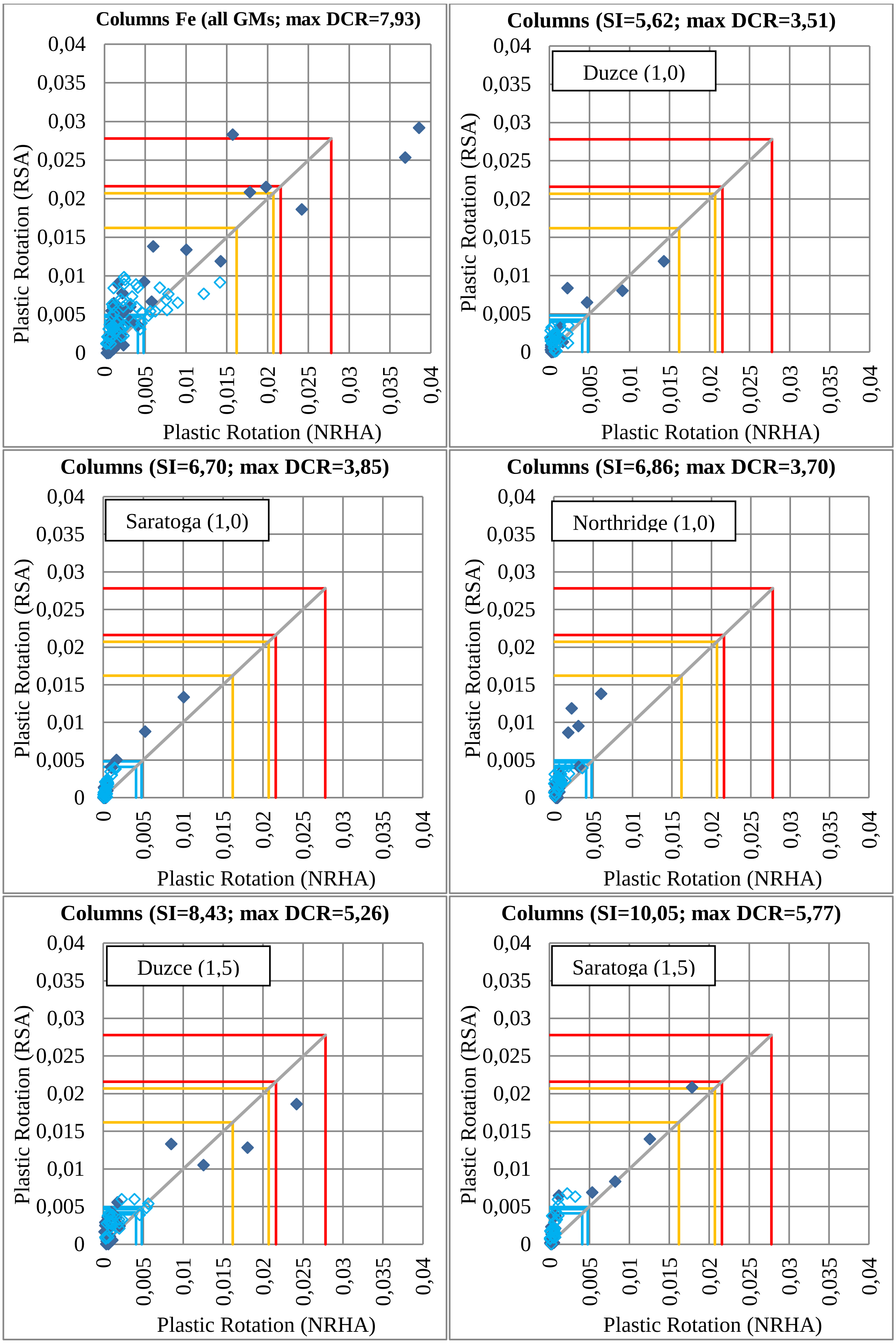


Figure 3.38 cont'd: Average plastic rotation demands from RSA and NRHA for columns of 6 story 3D R/C frame with capacity design

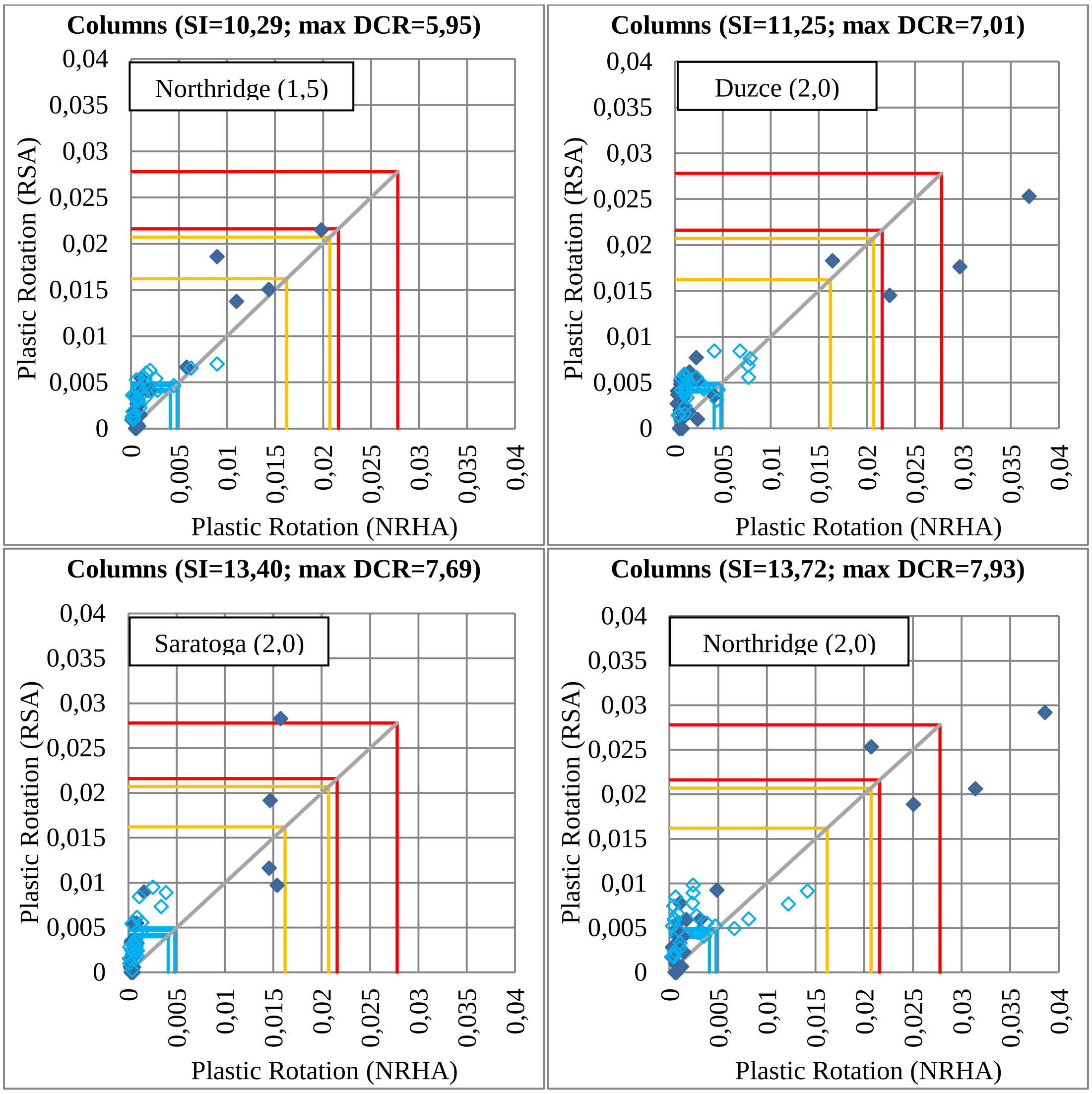


Figure 3.38 cont'd: Average plastic rotation demands from RSA and NRHA for columns of 6 story 3D R/C frame with capacity design

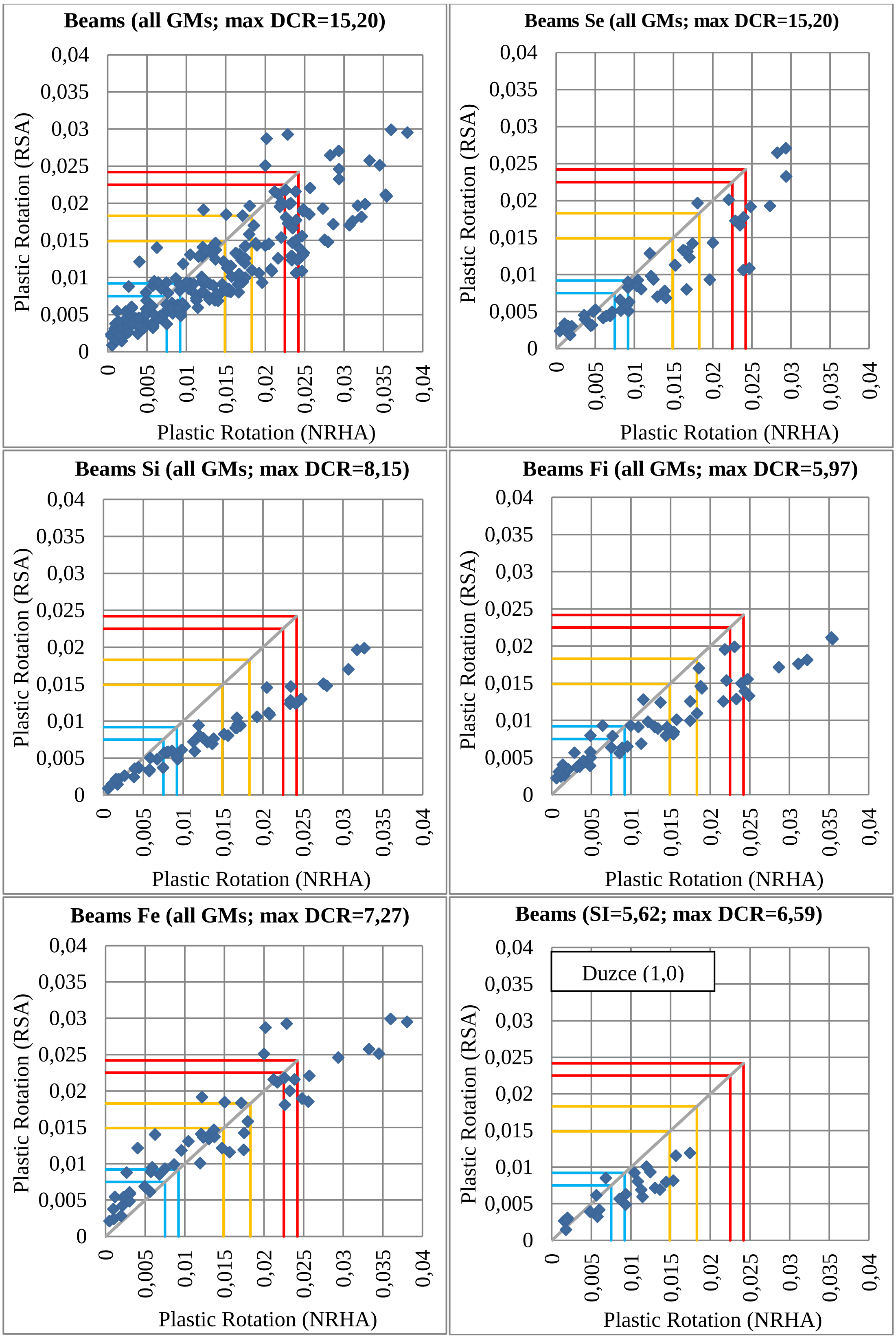


Figure 3.39: Average plastic rotation demands from RSA and NRHA for beams of 6 story 3D R/C frame with capacity design

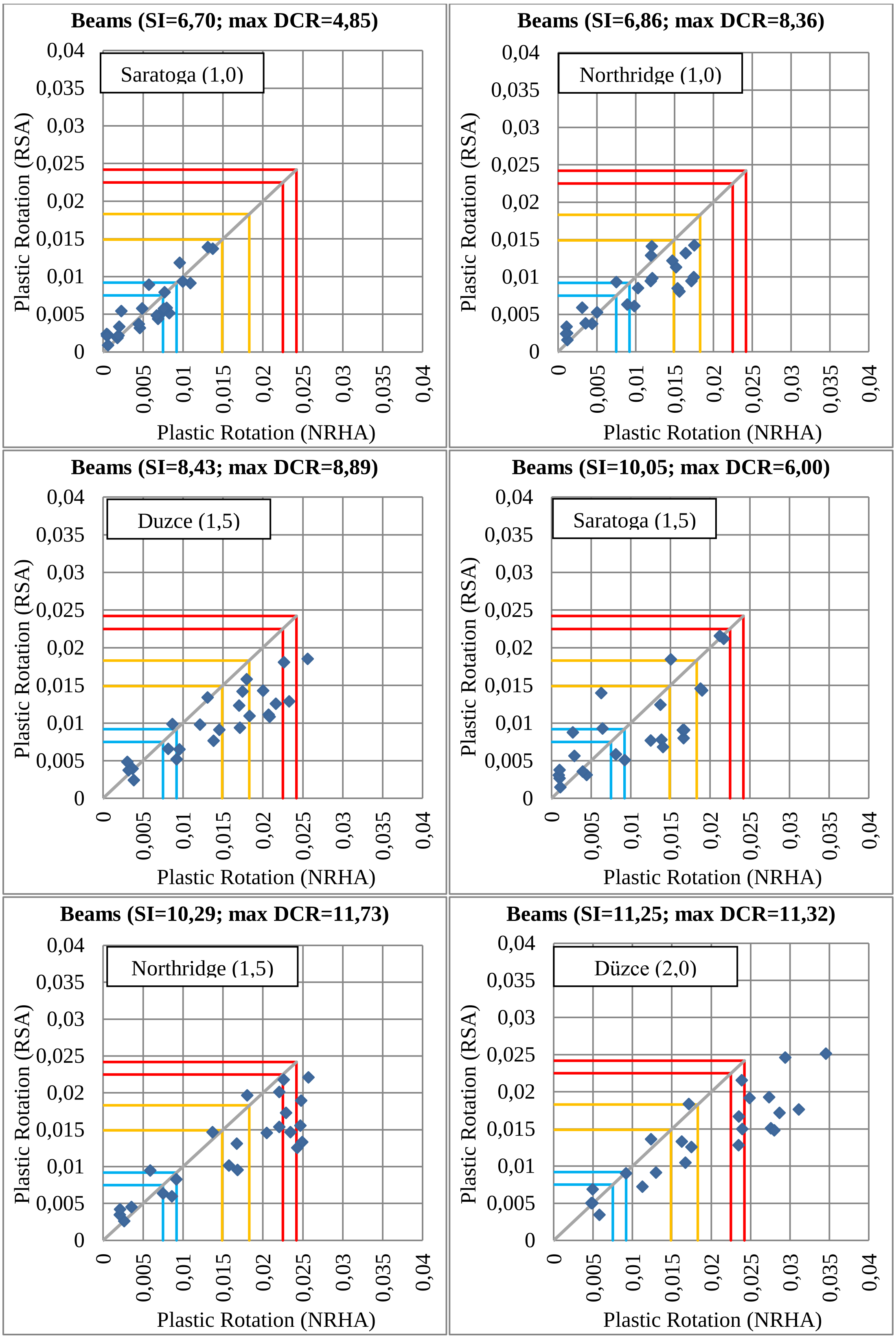


Figure 3.39 cont'd: Average plastic rotation demands from RSA and NRHA for beams of 6 story 3D R/C frame with capacity design

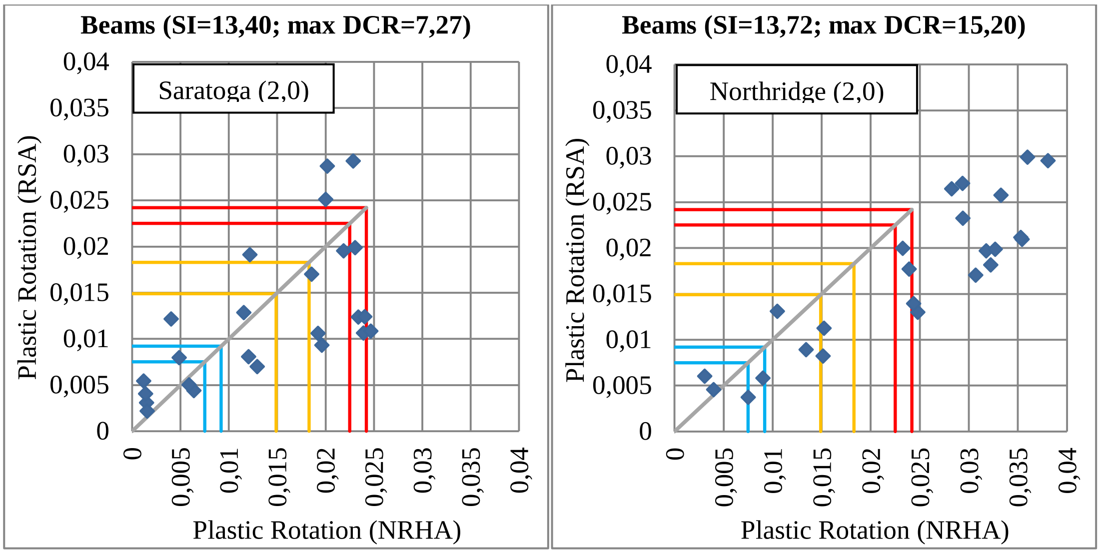


Figure 3.39 cont'd: Average plastic rotation demands from RSA and NRHA for beams of 6 story 3D R/C frame with capacity design

From Figures 3.38 and 3.39 along with Table 3.21, the following outcomes are obtained.

- The 6 story frame is classified as having severe weak story and severe torsional strength irregularity according to ASCE 41. Thus, linear elastic procedure is not applicable. Also linear elastic procedure is not applicable according to Eurocode 8 since the max to min DCR ratios are well above 2.5 under all ground motions. However deformation responses from linear and nonlinear analyses reveal that linear procedures are applicable for this structure if the maximum DCR is less than 4.
- Considering the overall responses of beams and columns, torsional strength irregularity and weak story irregularity is not effective in this building. Therefore, the ratio of maximum DCR at the flexible side to that at the stiff side at a story can be set as 2 in order to assign torsional strength irregularity, instead of 1.5, which is given in ASCE 41.
- Considering the fact that this structure does not have torsional strength irregularity according to the revised criteria above, each frame can be investigated separately for the applicability of linear procedure.

- The specified limiting DCRs at Chapters 3.5 and 3.6 are valid for frames of this building, too.

3.8.2. Eight Story 3D R/C Building (R=8)

Similar to the previous building, torsion is the most critical factor determining the response of this structure. Table 3.22 is prepared for this building to check the effect of torsion, which is the same procedure applied for previous building. Since the GM scale is not effective for the factors calculated, only one scale of each ground motion are given in Table 3.22.

Table 3.22: Maximum beam and column DCRs for each frame of 8 story 3D frame

	Columns						Beams						GM name (scale)
story	Se	Si	Fi	Fe	max/min DCR	k	Se	Si	Fi	Fe	max/min DCR	k	
1	3,62	2,59	3,36	5,71	**2,20**	**1,58**	6,94	4,56	3,95	4,96	**1,76**	**1,40**	Düzce (1,5)
2	2,46	1,70	2,08	3,64	**2,14**	**1,48**	9,17	5,66	4,44	5,33	**2,07**	**1,72**	
3	2,28	1,61	1,94	3,21	**1,99**	**1,41**	8,49	5,25	4,08	4,86	**2,08**	**1,75**	
4	2,22	1,60	1,99	3,36	**2,10**	**1,51**	5,92	3,86	3,31	4,14	**1,79**	**1,43**	
5	1,91	1,42	1,88	3,21	**2,27**	**1,68**	5,30	3,56	3,14	3,94	**1,69**	**1,35**	
6	1,93	1,54	2,08	3,46	**2,25**	**1,79**	6,41	4,24	3,42	4,03	**1,87**	**1,59**	
7	1,76	1,52	2,00	3,00	**1,98**	**1,71**	5,55	3,74	2,95	3,36	**1,88**	**1,65**	
8	1,44	1,37	1,61	2,26	**1,65**	**1,57**	2,63	1,94	1,59	1,80	**1,65**	**1,46**	
1	4,27	2,94	3,72	6,32	**2,15**	**1,48**	8,13	5,19	4,43	5,59	**1,84**	**1,45**	Northridge (1,5)
2	2,96	2,00	2,40	4,21	**2,11**	**1,43**	10,68	6,43	5,04	6,14	**2,12**	**1,74**	
3	2,72	1,85	2,20	3,66	**1,98**	**1,35**	9,95	5,98	4,64	5,60	**2,15**	**1,78**	
4	2,59	1,79	2,17	3,68	**2,05**	**1,42**	7,02	4,43	3,73	4,70	**1,88**	**1,49**	
5	2,12	1,53	1,98	3,39	**2,21**	**1,59**	5,99	3,91	3,39	4,27	**1,77**	**1,40**	
6	1,98	1,56	2,07	3,42	**2,19**	**1,73**	6,73	4,38	3,49	4,10	**1,93**	**1,64**	
7	1,72	1,48	1,88	2,76	**1,87**	**1,61**	5,61	3,75	2,90	3,25	**1,94**	**1,72**	
8	1,40	1,33	1,50	2,03	**1,52**	**1,45**	2,62	1,93	1,54	1,70	**1,71**	**1,54**	
1	3,27	2,54	3,58	8,23	**3,24**	**2,52**	6,44	4,61	4,37	7,36	**1,69**	**1,14**	Saratoga (1,5)
2	2,31	1,78	2,43	5,73	**3,21**	**2,48**	8,78	5,91	5,13	8,24	**1,71**	**1,07**	
3	2,19	1,70	2,26	5,03	**2,97**	**2,30**	8,27	5,57	4,80	7,64	**1,72**	**1,08**	
4	2,07	1,60	2,14	4,82	**3,00**	**2,32**	5,63	4,02	3,77	6,32	**1,68**	**1,12**	
5	1,63	1,28	1,77	4,02	**3,13**	**2,46**	4,83	3,46	3,19	5,26	**1,65**	**1,09**	
6	1,56	1,28	1,67	3,54	**2,78**	**2,27**	5,90	4,02	3,17	4,49	**1,86**	**1,31**	
7	1,49	1,29	1,53	2,61	**2,03**	**1,75**	5,17	3,52	2,60	3,30	**1,99**	**1,57**	
8	1,31	1,24	1,32	1,93	**1,56**	**1,47**	2,46	1,82	1,38	1,67	**1,78**	**1,47**	

Similar to the six story building, according to the ASCE 41 criteria, this building is torsional irregularity too. However if that limit is set as 2, similar to the previous case, this building shall not have torsional strength irregularity.

Average plastic rotation demands of each individual frames from RSA and NRHA on columns and beams are shown in Figures 3.40 and 3.41, respectively.

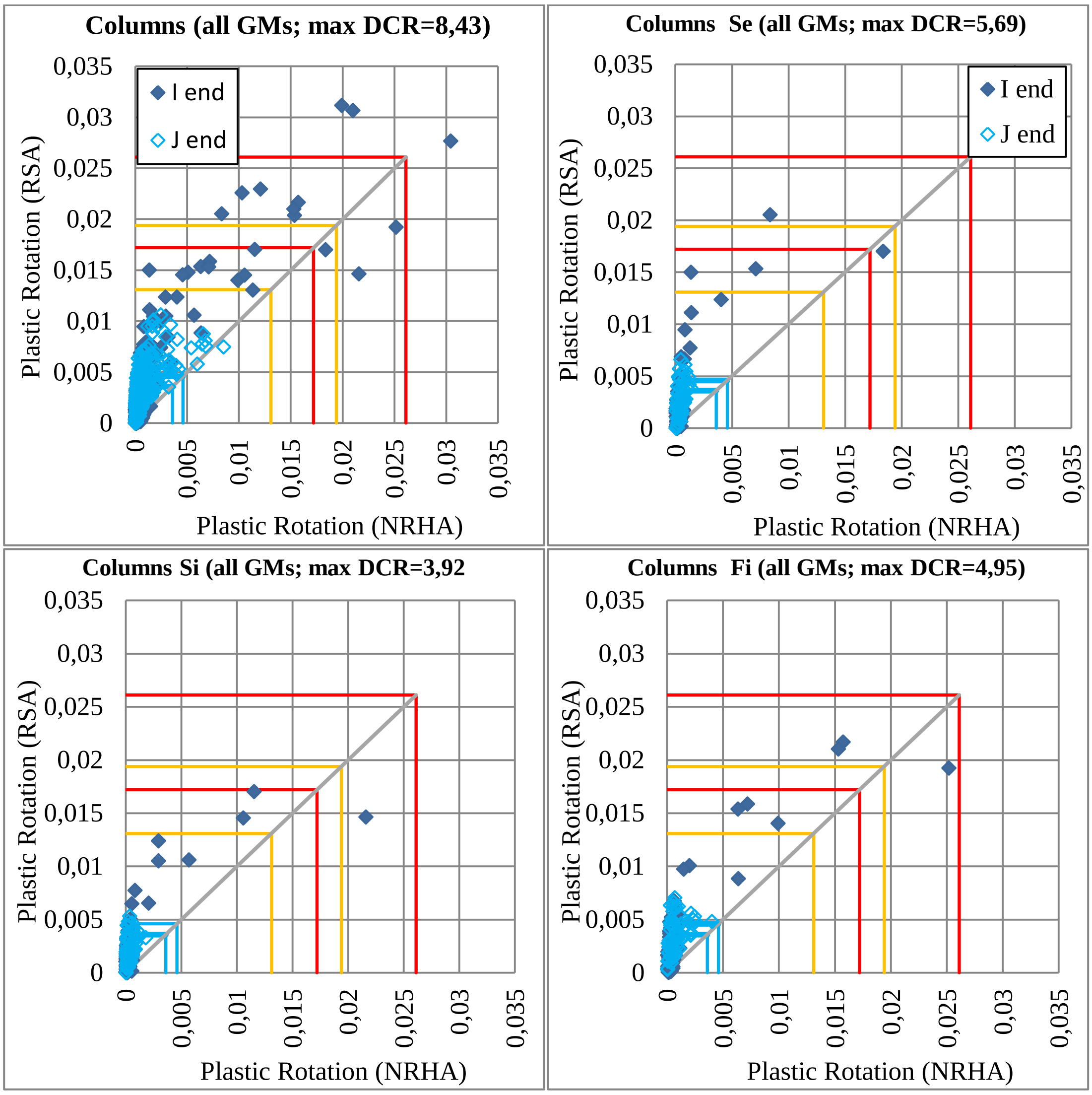


Figure 3.40: Average plastic rotation demands from RSA and NRHA for columns of 8 story 3D R/C frame

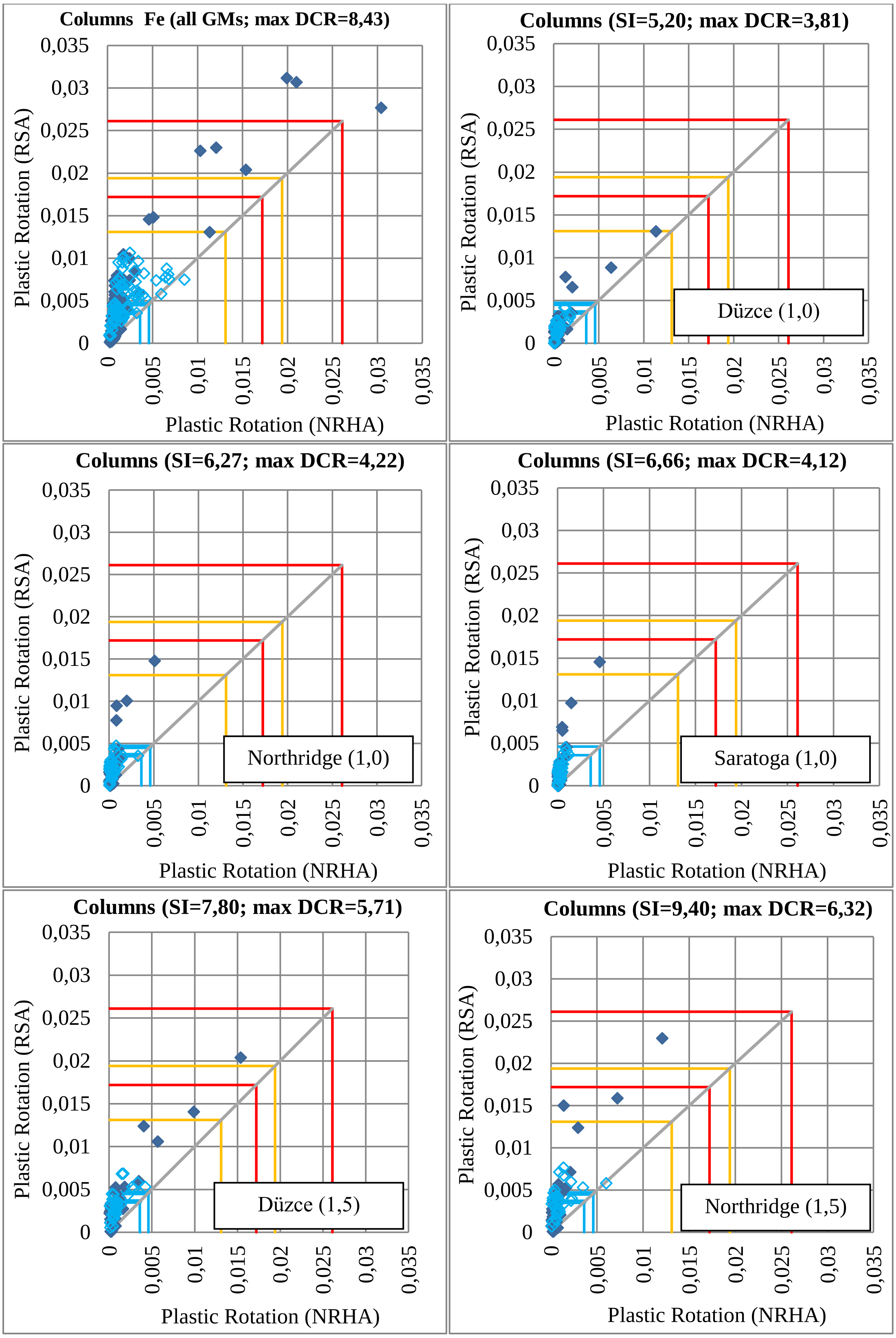


Figure 3.40 cont'd: Average plastic rotation demands from RSA and NRHA for columns of 8 story 3D R/C frame

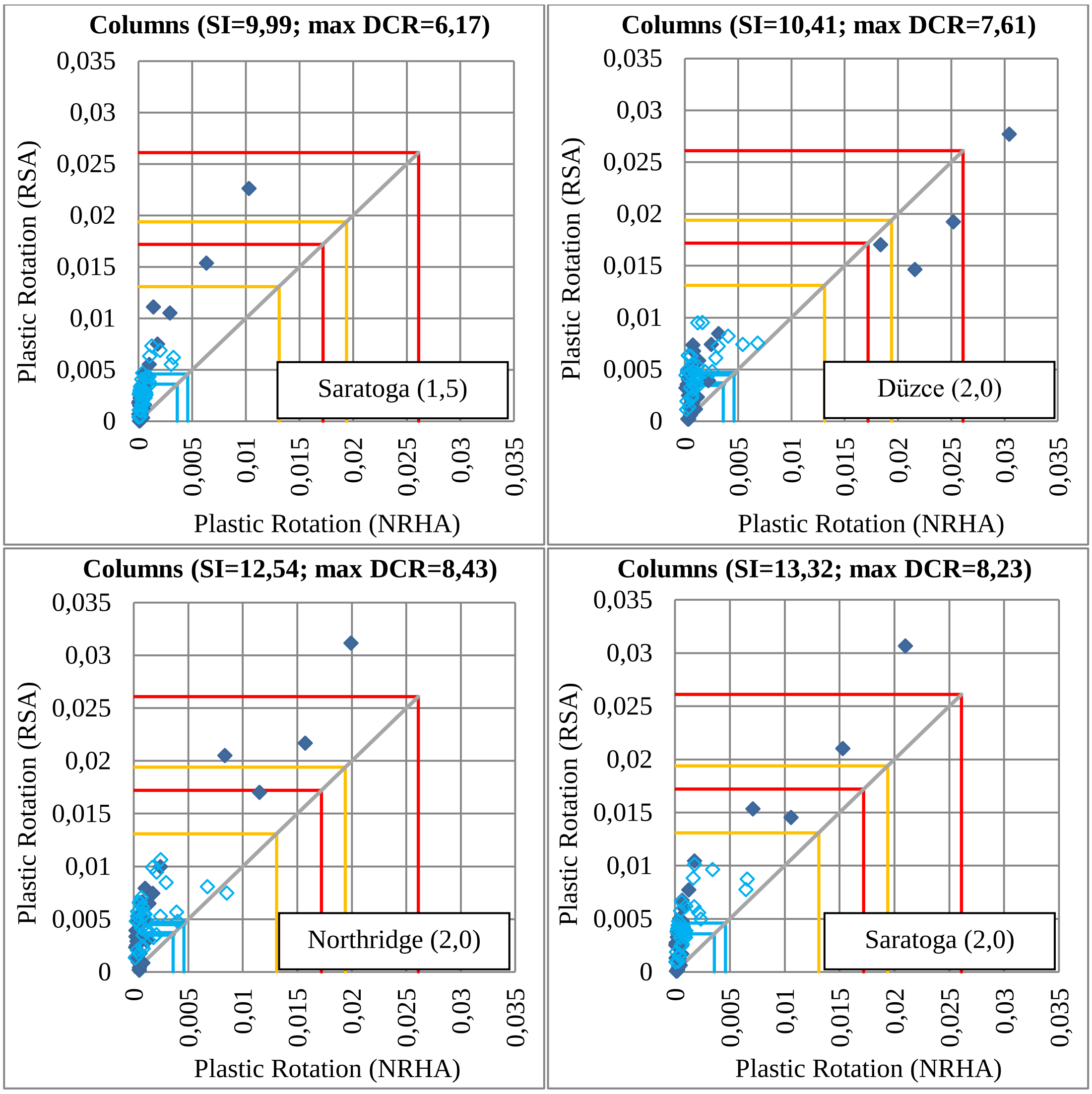


Figure 3.40 cont'd: Average plastic rotation demands from RSA and NRHA for columns of 8 story 3D R/C frame

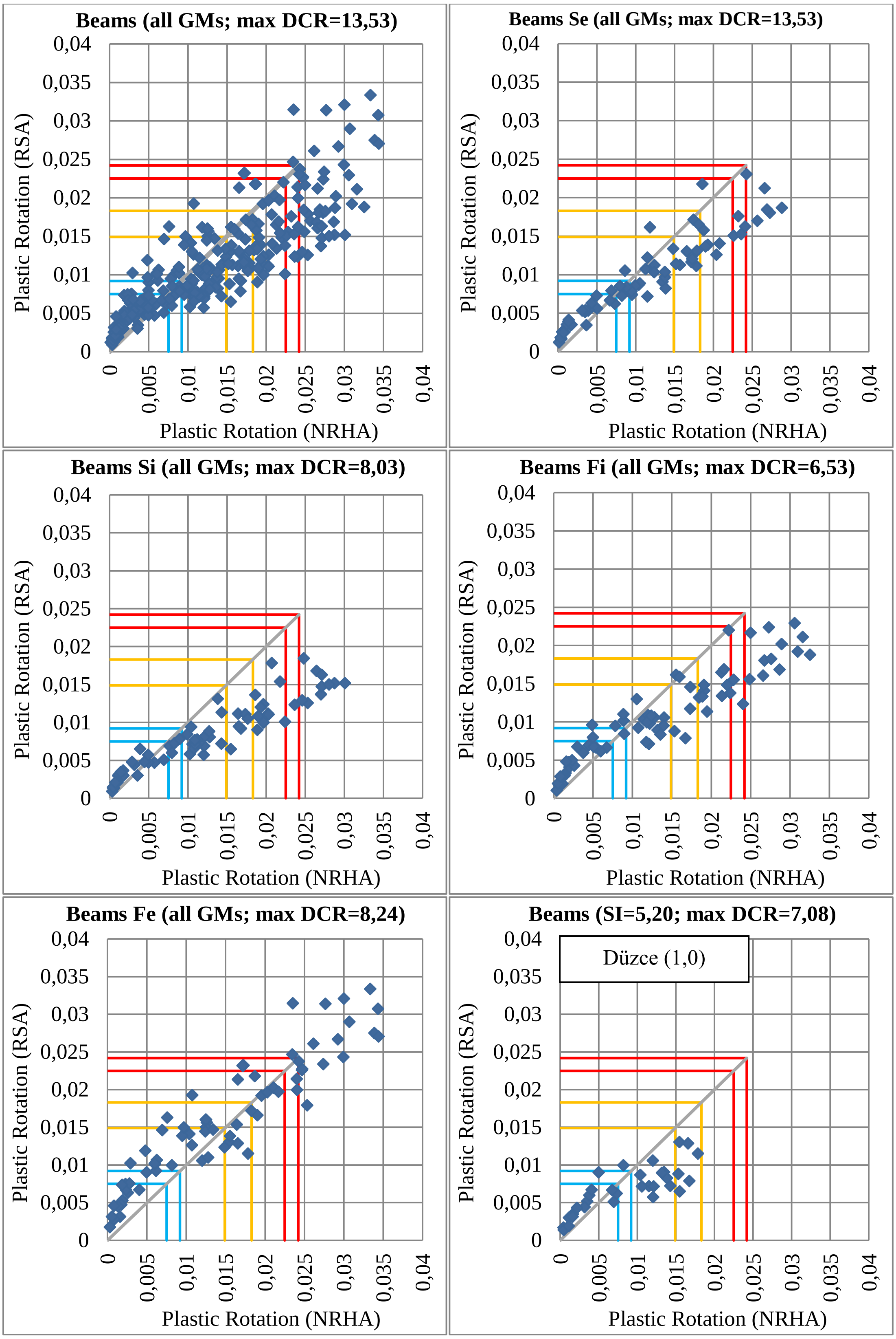


Figure 3.41: Average plastic rotation demands from RSA and NRHA for beams of 8 story 3D R/C frame

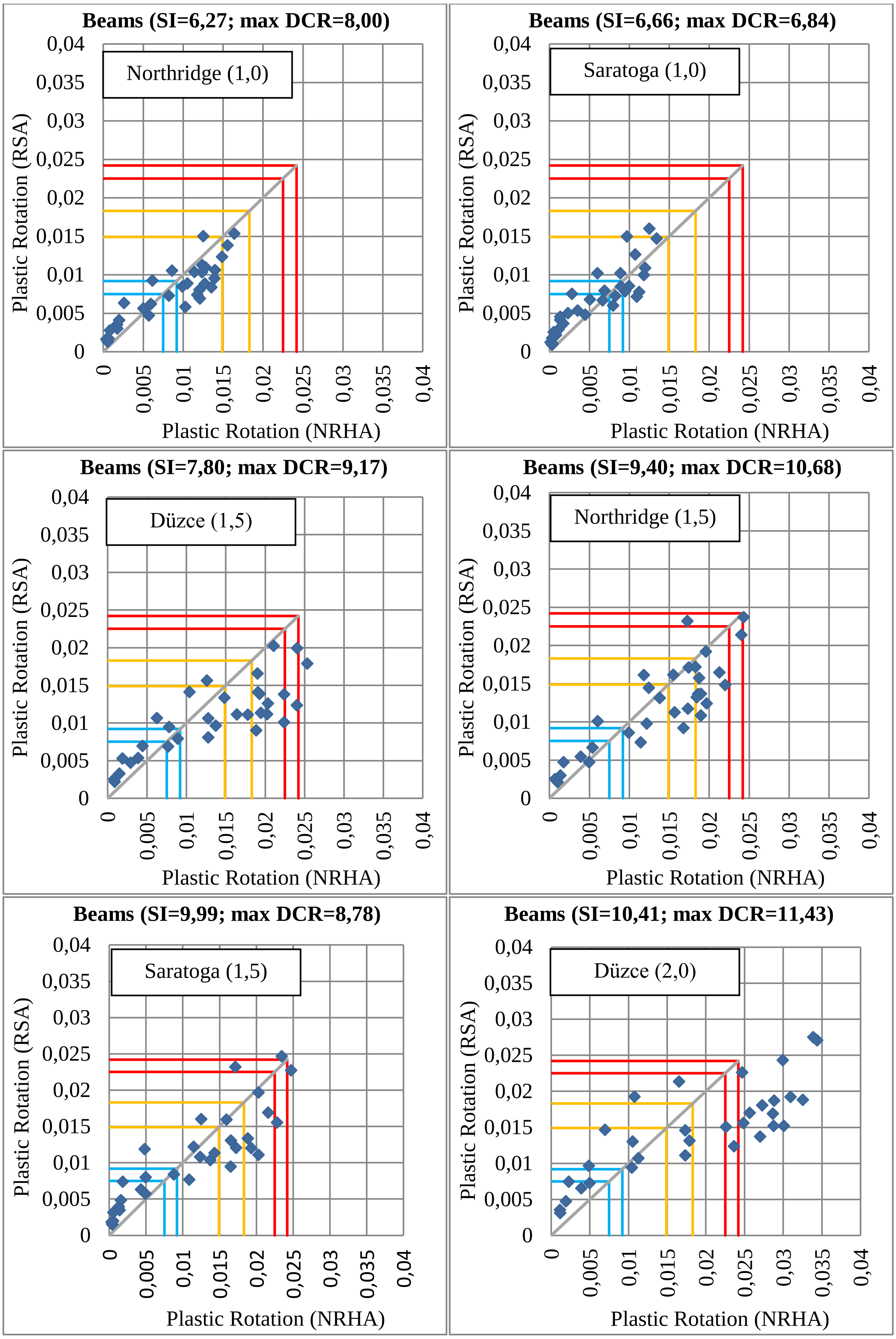


Figure 3.41 cont'd: Average plastic rotation demands from RSA and NRHA for beams of 8 story 3D R/C frame

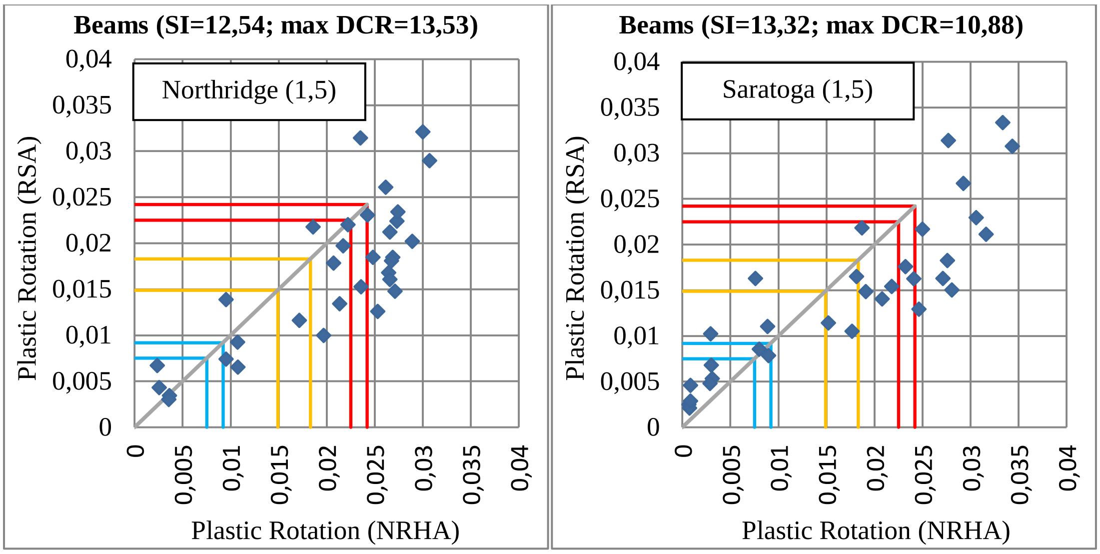


Figure 3.41 cont'd: Average plastic rotation demands from RSA and NRHA for beams of 8 story 3D R/C frame

The following discussions are reached by considering Figures 3.40, 3.41 and Table 3.22.

- Behavior of this building is very similar to the previous 6 story building. For both of the structures, torsional strength irregularity is not present according to the revised criteria proposed above. Therefore, all of the frames should be investigated separately.
- Since all of the frames can be considered individually, it is valid to employ the limiting DCR values proposed in Sections 3.5 and 3.6 for the frames of this building.

CHAPTER 4

SUMMARY AND CONCLUSIONS

4.1. Summary

This study is conducted in two parts. In the first part, the aim is to verify and calibrate the current seismic assessment procedures employed in TEC2007. For this purpose, three capacity designed frame structures and a four story retrofitted building are analyzed by using response spectrum analysis and conventional pushover analysis by using the linear elastic design spectrum given in TEC2007. Both force-based and displacement-based results are obtained and they are compared to each other. Results of force-based procedure are solely based on TEC2007 *r*-factors whereas for displacement-based procedure, plastic rotation limits of ASCE41 are used. During the calibration process, over-estimation or under-estimation of force-based procedure in TEC2007 is determined by considering the displacement-based results. Then, the performance limits of TEC2007 are adjusted accordingly for beams, columns and shear walls.

In the second part of this study, limitations of linear elastic procedures in seismic assessment of existing structures are investigated. For this part, all nine case study buildings are analyzed with response spectrum analysis, conventional pushover analysis (first mode dominant buildings) and nonlinear response history analysis. Three different ground motions, each of them further scaled up by 1.5 and 2, which makes a total of nine ground motions, are used in the analyses. Results are obtained in terms of plastic rotation demands at member ends for both linear and nonlinear analyses. Then, for each structural system, the maximum DCR where linear analysis becomes unsafe afterwards is determined. Since DCR is the only meaningful parameter to estimate the level of nonlinearity in the structure without employing nonlinear analysis, the relationship between the unsafe linear results and element DCRs for these results are obtained for different structural systems and for different elements. As a result, different limitations are suggested for linear elastic methods to be used in seismic assessment of any existing structure.

4.2. Conclusions

Conclusions reached are presented in two parts. In the first part, conclusions obtained from verification and calibration procedure of TEC2007 are presented and in the second part, conclusions obtained from limitations on linear elastic procedures for seismic assessment procedures are given.

4.2.1 Conclusions on the Verification and Calibration of TEC2007 Performance Limits

Force-based seismic assessment procedure of TEC2007 is generally reliable, considering the reasonable correlation with displacement-based procedure of ASCE 41. However, performance limits of TEC2007 are found slightly higher especially for the code conforming members, and needs to be adjusted. On average, 15 % reduction is suggested for the performance limits of code conforming members. The existing limits used for non-conforming members are generally found safe.

4.2.2 Conclusions on Limitations of Linear Elastic Procedures for Seismic Assessment of Buildings

Different limitations on linear elastic procedures for seismic assessment for different structural systems are proposed in this part.

Frame systems with no torsional irregularity:

- Weak story irregularity of ASCE 41 (the ratio of the average shear DCR of a story to that of the adjacent story in the direction of earthquake excitation) is not an effective parameter in deciding for the applicability of linear procedures to frames which satisfy the average strong column-weak beam condition at each story.
- The ratio of average column DCR to average beam DCR (r_{DCR}) at a story is an effective parameter for deciding on the limitations of linear procedure in frames.
- If $r_{DCR} < 0.75$, development of a column (soft story) mechanism is not expected at any story. Then the linear procedure can be applied only if the maximum column flexure DCR < 3 and the maximum beam flexure DCR < 6.
- If $0.75 < r_{DCR} < 1.0$ at any story, there is a possibility for the development of a mixed column and beam mechanism. Then the linear procedure can be applied only if the maximum column flexure DCR < 2 and the maximum beam flexure DCR < 3.
- If $r_{DCR} > 1.0$ at any story, there is a strong possibility for the development of a column mechanism. Then the linear procedure can be applied only if the maximum column flexure DCR < 1.5.

Frame-wall systems with no torsional irregularity:

- A system qualifies as a frame-wall system in this study if at least 75% of the total base shear force is shared by shear walls in the direction of earthquake excitation.
- A soft story mechanism is not expected in a frame-wall system.
- The ratio of the average shear DCR of a story to that of the adjacent story in the direction of earthquake excitation does not appear as an effective parameter for deciding on the limitations of linear procedures. Consistency of the results of linear and nonlinear procedures is not sensitive to this ratio, which does not generally exceed 1.5 when the shear walls are continuous throughout the building height.
- Maximum flexure DCR of the shear walls is the controlling parameter for the validity of linear procedures since shear walls dominate seismic deformation

response. When a shear wall yields significantly at the base, linear and nonlinear deformation patterns deviate significantly.

- It is suggested that the linear procedures can be applied safely only if the wall maximum flexure DCR < 2.

Systems with torsional strength irregularity:

- Torsional strength irregularity is suggested to exists when the ratio of maximum DCR at the flexible side to that at the stiff side at a story exceeds 2, both for columns and beams separately.
- When there is no torsional strength irregularity, each frame can be treated separately as a frame system with no torsional irregularity.
- No data is obtained for systems with severe torsional strength irregularity, however it may be suggested that linear procedures can be applied if maximum member DCR < 2 at all frames in the direction of earthquake excitation.

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APPENDIX A

PERFORMANCE LIMIT TABLES OF TEC2007 AND ASCE41

Force-Based Performance Limit Tables of TEC2007

Table A.1: Demand to capacity ratios that define performance limits for R/C beams of TEC2007 (r_s)

Sünek Kirişler			Hasar Sınırı		
$\frac{\rho-\rho'}{\rho_b}$	Sargılama	$\frac{V_e}{b_w d\ f_{ctm}}$ (1)	MN	GV	GÇ
≤ 0.0	Var	≤ 0.65	3	7	10
≤ 0.0	Var	≥ 1.30	2.5	5	8
≥ 0.5	Var	≤ 0.65	3	5	7
≥ 0.5	Var	≥ 1.30	2.5	4	5
≤ 0.0	Yok	≤ 0.65	2.5	4	6
≤ 0.0	Yok	≥ 1.30	2	3	5
≥ 0.5	Yok	≤ 0.65	2	3	5
≥ 0.5	Yok	≥ 1.30	1.5	2.5	4

Table A.2: Demand to capacity ratios that define performance limits for R/C columns of TEC2007 (r_s)

Sünek Kolonlar			Hasar Sınırı		
$\frac{N_K}{A_c f_{cm}}$ (1)	Sargılama	$\frac{V_e}{b_w d \; f_{ctm}}$ (2)	MN	GV	GÇ
≤ 0.1	Var	≤ 0.65	3	6	8
≤ 0.1	Var	≥ 1.30	2.5	5	6
≥ 0.4 ve ≤ 0.7	Var	≤ 0.65	2	4	6
≥ 0.4 ve ≤ 0.7	Var	≥ 1.30	1.5	2.5	3.5
≤ 0.1	Yok	≤ 0.65	2	3.5	5
≤ 0.1	Yok	≥ 1.30	1.5	2.5	3.5
≥ 0.4 ve ≤ 0.7	Yok	≤ 0.65	1.5	2	3
≥ 0.4 ve ≤ 0.7	Yok	≥ 1.30	1	1.5	2
≥ 0.7	–	–	1	1	1

Table A.3: Demand to capacity ratios that define performance limits for R/C shear walls of TEC2007 (r_s)

Sünek Perdeler	Hasar Sınırı		
Perde Uç Bölgesinde Sargılama	MN	GV	GÇ
Var	3	6	8
Yok	2	4	6

Force-Based Performance Limit Tables of ASCE41

Table A.4: Numerical acceptance criteria for linear procedures of ASCE41 – R/C beams

Conditions			*m*-factors[3]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]							
$\frac{\rho-\rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d\sqrt{f'_c}}$ [4]					
≤ 0.0	C	≤ 3	3	6	7	6	10
≤ 0.0	C	≥ 6	2	3	4	3	5
≥ 0.5	C	≤ 3	2	3	4	3	5
≥ 0.5	C	≥ 6	2	2	3	2	4
≤ 0.0	NC	≤ 3	2	3	4	3	5
≤ 0.0	NC	≥ 6	1.25	2	3	2	4
≥ 0.5	NC	≤ 3	2	3	3	3	4
≥ 0.5	NC	≥ 6	1.25	2	2	2	3
ii. Beams controlled by shear[1]							
Stirrup spacing $\leq d/2$			1.25	1.5	1.75	3	4
Stirrup spacing $> d/2$			1.25	1.5	1.75	2	3
iii. Beams controlled by inadequate development or splicing along the span[1]							
Stirrup spacing $\leq d/2$			1.25	1.5	1.75	3	4
Stirrup spacing $> d/2$			1.25	1.5	1.75	2	3
iv. Beams controlled by inadequate embedment into beam-column joint[1]							
			2	2	3	3	4

1. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at $\leq d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.
4. V is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.4.2.4.1.

Table A.5: Numerical acceptance criteria for linear procedures of ASCE41 – R/C columns

Conditions			m-factors[3]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
Condition i. [1]							
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$						
≤ 0.1	≥ 0.006		2	2.5	3	4	5
≥ 0.6	≥ 0.006		1.25	1.8	1.9	1.9	2
≤ 0.1	≤ 0.002		2	2	2.6	2.6	3
≥ 0.6	≤ 0.002		1.1	1.1	1.2	1.2	1.4
Condition ii. [1]							
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f_c}}$ [4]					
≤ 0.1	≥ 0.006	≤ 3	2	2.5	3	4	5
≤ 0.1	≥ 0.006	≥ 6	2	2	2.5	4	5
≥ 0.6	≥ 0.006	≤ 3	1.25	1.8	1.9	1.9	2
≥ 0.6	≥ 0.006	≥ 6	1.25	1.5	1.6	1.6	1.8
≤ 0.1	≤ 0.0005	≤ 3	1.2	1.3	1.4	1.4	1.6
≤ 0.1	≤ 0.0005	≥ 6	1	1	1.1	1.1	1.2
≥ 0.6	≤ 0.0005	≤ 3	1	1	1.1	1.1	1.2
≥ 0.6	≤ 0.0005	≥ 6	1	1	1	1	1
Condition iii. [1]							
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$						
≤ 0.1	≥ 0.006		1	1	1	4	5
≥ 0.6	≥ 0.006		1	1	1	1.6	1.8
≤ 0.1	≤ 0.002		1	1	1	1.1	1.2
≥ 0.6	≤ 0.002		1	1	1	1	1
Condition iv. Columns controlled by inadequate development or splicing along the clear height[1]							
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$						
≤ 0.1	≥ 0.006		1	1	1	4	5
≥ 0.6	≥ 0.006		1	1	1	1.6	1.8
≤ 0.1	≤ 0.002		1	1	1	1.1	1.2
≥ 0.6	≤ 0.002		1	1	1	1	1

1. Refer to Section 6.4.2.2.2 for definition of conditions i, ii, and iii. Columns will be considered to be controlled by inadequate development or splices when the calculated steel stress at the splice exceeds the steel stress specified by Equation 6-2. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Where $P > 0.7 A_g f'_c$, the m-factor shall be taken as unity for all performance levels unless columns have transverse reinforcement consisting of hoops with 135 degree hooks spaced at $\leq d/3$ and the strength provided by the hoops (V_s) is at least three-fourths of the design shear. P is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.
3. Linear interpolation between values listed in the table shall be permitted.
4. V is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.4.2.4.1.

Table A.6: Numerical acceptance criteria for linear procedures of ASCE41 – R/C shear walls and associated components controlled by flexure

Conditions			*m*-factors[~~4~~6]				
			Performance Level				
				Component Type			
				Primary		Secondary	
			IO	LS	CP	LS	CP
i. Shear walls and wall segments							
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$ [~~6~~5]	$\frac{V}{t_w l_w \sqrt{f'_c}}$ [~~5~~4]	Confined Boundary[1]					
≤ 0.1	≤ ~~3~~4	Yes	2	4	6	6	8
≤ 0.1	≥ 6	Yes	2	3	4	4	6
≥ 0.25	≤ ~~3~~4	Yes	1.5	3	4	4	6
≥ 0.25	≤ 6	Yes	1.25	2	2.5	2.5	4
≤ 0.1	≤ ~~3~~4	No	2	2.5	4	4	6
≤ 0.1	≥ 6	No	1.5	2	2.5	2.5	4
≥ 0.25	≤ ~~3~~4	No	1.25	1.5	2	2	3
≥ 0.25	≥ 6	No	1.25	1.5	1.75	1.75	2
~~**ii. Columns supporting discontinuous shear walls**~~							
~~Transverse reinforcement[2]~~							
~~Conforming~~			~~1~~	~~1.5~~	~~2~~	~~n.a.~~	~~n.a.~~
~~Nonconforming~~			~~1~~	~~1~~	~~1~~	~~n.a.~~	~~n.a.~~
iii. Shear wall coupling beams[~~4~~3]							
Longitudinal reinforcement and transverse reinforcement[~~3~~2]		$\frac{V}{t_w l_w \sqrt{f'_c}}$ [~~5~~4]					
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	2	4	6	6	9
		≥ 6	1.5	3	4	4	7
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	1.5	3.5	5	5	8
		≥ 6	1.2	1.8	2.5	2.5	4
Diagonal reinforcement		n.a.	2	5	7	7	10

1. A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. Otherwise, boundary elements shall be considered not confined. ~~Requirements for a confined boundary are the same as those given in ACI 318.~~

~~2. Requirements for conforming transverse reinforcement in columns are: (a) hoops over the entire length of the column at a spacing ≤ $d/2$, and (b) strength of hoops V_s ≥ required shear strength of column.~~

~~3~~2. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups $V_s \geq 3/4$ of required shear strength of the coupling beam.

~~4~~3. For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.

~~5~~4. V is the design shear force calculated using limit-state analysis procedures in accordance with Section 6.7.2.4.

~~6~~5. P is the design axial force in the member. Alternatively, use of axial loads determined based on a limit state analysis shall be permitted.

~~7~~6. Linear interpolation between values listed in the table shall be permitted.

Displacement-Based Performance Limit Tables of ASCE41

Table A.7: Modeling parameters and numerical acceptance criteria for nonlinear procedures of ASCE41 – R/C beams

Conditions			Modeling Parameters[3]			Acceptance Criteria[3,4]				
						Plastic Rotations Angle, radians				
						Performance Level				
			Plastic Rotations Angle, radians		Residual Strength Ratio		Component Type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
i. Beams controlled by flexure[1]										
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf.[2]	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.0	C	≤ 3	0.025	0.05	0.2	0.010	0.02	0.025	0.02	0.05
≤ 0.0	C	≥ 6	0.02	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≥ 0.5	C	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≥ 0.5	C	≥ 6	0.015	0.02	0.2	0.005	0.005	0.015	0.015	0.02
≤ 0.0	NC	≤ 3	0.02	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.0	NC	≥ 6	0.01	0.015	0.2	0.0015	0.005	0.01	0.01	0.015
≥ 0.5	NC	≤ 3	0.01	0.015	0.2	0.005	0.01	0.01	0.01	0.015
≥ 0.5	NC	≥ 6	0.005	0.01	0.2	0.0015	0.005	0.005	0.005	0.01
ii. Beams controlled by shear[1]										
Stirrup spacing ≤ d /2			0.0030	0.02	0.2	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d /2			0.0030	0.01	0.2	0.0015	0.0020	0.0030	0.005	0.01
iii. Beams controlled by inadequate development or splicing along the span[1]										
Stirrup spacing ≤ d /2			0.0030	0.02	0.0	0.0015	0.0020	0.0030	0.01	0.02
Stirrup spacing > d /2			0.0030	0.01	0.0	0.0015	0.0020	0.0030	0.005	0.01
iv. Beams controlled by inadequate embedment into beam-column joint[1]										
			0.015	0.03	0.2	0.01	0.01	0.015	0.02	0.03

1. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ $d/3$, and if, for components of moderate and high ductility demand, the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.
3. Linear interpolation between values listed in the table shall be permitted.
4. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.

Table A.8: Modeling parameters and numerical acceptance criteria for nonlinear procedures of ASCE41 – R/C columns

Conditions			Modeling Parameters[3]			Acceptance Criteria[3,4]				
			Plastic Rotations Angle, radians		Residual Strength Ratio	Plastic Rotations Angle, radians				
						Performance Level				
							Component Type			
							Primary		Secondary	
			a	b	c	IO	LS	CP	LS	CP
Condition i. [1]										
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$									
≤ 0.1	≥ 0.006		0.035	0.060	0.2	0.005	0.026	0.035	0.045	0.060
≥ 0.6	≥ 0.006		0.010	0.010	0.0	0.003	0.008	0.009	0.009	0.010
≤ 0.1	$= 0.002$		0.027	0.034	0.2	0.005	0.020	0.027	0.027	0.034
≥ 0.6	$= 0.002$		0.005	0.005	0.0	0.002	0.003	0.004	0.004	0.005
Condition ii. [1]										
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$	$\frac{V}{b_w d \sqrt{f'_c}}$								
≤ 0.1	≥ 0.006	≤ 3	0.032	0.060	0.2	0.005	0.024	0.032	0.045	0.060
≤ 0.1	≥ 0.006	≥ 6	0.025	0.060	0.2	0.005	0.019	0.025	0.045	0.060
≥ 0.6	≥ 0.006	≤ 3	0.010	0.010	0.2	0.003	0.008	0.009	0.009	0.010
≥ 0.6	≥ 0.006	≥ 6	0.008	0.008	0.2	0.003	0.006	0.007	0.007	0.008
≤ 0.1	≤ 0.0005	≤ 3	0.012	0.012	0.0	0.005	0.009	0.010	0.010	0.012
≤ 0.1	≤ 0.0005	≥ 6	0.006	0.006	0.0	0.004	0.005	0.005	0.005	0.006
≥ 0.6	≤ 0.0005	≤ 3	0.004	0.004	0.0	0.002	0.003	0.003	0.003	0.004
≥ 0.6	≤ 0.0005	≥ 6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Condition iii. [1]										
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$									
≤ 0.1	≥ 0.006		0.0	0.060	0.0	0.0	0.0	0.0	0.045	0.060
≥ 0.6	≥ 0.006		0.0	0.008	0.0	0.0	0.0	0.0	0.007	0.008
≤ 0.1	≤ 0.0005		0.0	0.006	0.0	0.0	0.0	0.0	0.005	0.006
≥ 0.6	≤ 0.0005		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Condition iv. Columns controlled by inadequate development or splicing along the clear height[1]										
$\frac{P}{A_g f'_c}$ [2]	$\rho = \frac{A_v}{b_w s}$									
≤ 0.1	≥ 0.006		0.0	0.060	0.4	0.0	0.0	0.0	0.045	0.060
≥ 0.6	≥ 0.006		0.0	0.008	0.4	0.0	0.0	0.0	0.007	0.008
≤ 0.1	≤ 0.0005		0.0	0.006	0.2	0.0	0.0	0.0	0.005	0.006
≥ 0.6	≤ 0.0005		0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

1. Refer to Section 6.4.2.2.2 for definition of conditions i, ii, and iii. Columns will be considered to be controlled by inadequate development or splices when the calculated steel stress at the splice exceeds the steel stress specified by Equation 6-2. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
2. Where $P > 0.7A_g f'_c$, the plastic rotation angles shall be taken as zero for all performance levels unless columns have transverse reinforcement consisting of hoops with 135 degree hooks spaced at $\le d/3$ and the strength provided by the hoops (V_s) is at least three-fourths of the design shear. Axial load, P, shall be based on the maximum expected axial loads due to gravity and earthquake loads
3. Linear interpolation between values listed in the table shall be permitted.
4. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.

Table A.9: Modeling parameters and numerical acceptance criteria for nonlinear procedures of ASCE41 – R/C shear walls and associated components controlled by flexure

Conditions			Plastic Hinge Rotation (radians) a	Plastic Hinge Rotation (radians) b	Residual Strength Ratio c	Acceptable Plastic Hinge Rotation[45][56] (radians) — Performance Level: IO	Component Type: Primary LS	Primary CP	Secondary LS	Secondary CP
i. Shear walls and wall segments										
$\frac{(A_s - A'_s)f_y + P}{t_w l_w f'_c}$	$\frac{V}{t_w l_w \sqrt{f'_c}}$	Confined Boundary[1]								
≤ 0.1	≤ ~~3~~4	Yes	0.015	0.020	0.75	0.005	0.010	0.015	0.015	0.020
≤ 0.1	≥ 6	Yes	0.010	0.015	0.40	0.004	0.008	0.010	0.010	0.015
≥ 0.25	≤ ~~3~~4	Yes	0.009	0.012	0.60	0.003	0.006	0.009	0.009	0.012
≥ 0.25	≥ 6	Yes	0.005	0.010	0.30	0.0015	0.003	0.005	0.005	0.010
≤ 0.1	≤ ~~3~~4	No	0.008	0.015	0.60	0.002	0.004	0.008	0.008	0.015
≤ 0.1	≥ 6	No	0.006	0.010	0.30	0.002	0.004	0.006	0.006	0.010
≥ 0.25	≤ ~~3~~4	No	0.003	0.005	0.25	0.001	0.002	0.003	0.003	0.005
≥ 0.25	≥ 6	No	0.002	0.004	0.20	0.001	0.001	0.002	0.002	0.004
~~**ii. Columns supporting discontinuous shear walls**~~										
~~Transverse reinforcement[2]~~										
~~Conforming~~			~~0.010~~	~~0.015~~	~~0.20~~	~~0.003~~	~~0.007~~	~~0.010~~	~~n.a.~~	~~n.a.~~
~~Nonconforming~~			~~0.0~~	~~0.0~~	~~0.0~~	~~0.0~~	~~0.0~~	~~0.0~~	~~n.a.~~	~~n.a.~~
iii. Shear wall coupling beams[34]										
Longitudinal reinforcement and transverse reinforcement[23]		$\frac{V}{t_w l_w \sqrt{f'_c}}$								
Conventional longitudinal reinforcement with conforming transverse reinforcement		≤ 3	0.025	0.050	0.75	0.010	0.02	0.025	0.025	0.050
		≥ 6	0.020	0.040	0.50	0.005	0.010	0.020	0.020	0.040
Conventional longitudinal reinforcement with nonconforming transverse reinforcement		≤ 3	0.020	0.035	0.50	0.006	0.012	0.020	0.020	0.035
		≥ 6	0.010	0.025	0.25	0.005	0.008	0.010	0.010	0.025
Diagonal reinforcement		n.a.	0.030	0.050	0.80	0.006	0.018	0.030	0.030	0.050

1. A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. Otherwise, boundary elements shall be considered not confined.
2. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ $d/3$, and (b) strength of closed stirrups V_s ≥ 3/4 of required shear strength of the coupling beam.
3. For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.
4. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.
5. Linear interpolation between values listed in the table shall be permitted.~~1. A boundary element shall be considered confined where transverse reinforcement exceeds 75% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. It shall be permitted to take modeling parameters and acceptance criteria as 80% of confined values where boundary elements have at least 50% of the requirements given in ACI 318 and spacing of transverse reinforcement does not exceed $8d_b$. Otherwise, boundary elements shall be considered not confined.Requirements for a confined boundary are the same as those given in ACI 318.~~

~~2. Requirements for conforming transverse reinforcement in columns are: (a) hoops over the entire length of the column at a spacing ≤ d/2, and (b) strength of hoops Vs ≥ required shear strength of column.~~
~~3. Conventional longitudinal reinforcement consists of top and bottom steel parallel to the longitudinal axis of the coupling beam. Conforming transverse reinforcement consists of: (a) closed stirrups over the entire length of the coupling beam at a spacing ≤ d/3, and (b) strength of closed stirrups Vs ≥ 3/4 of required shear strength of the coupling beam.~~
~~4. For secondary coupling beams spanning <8'-0", with bottom reinforcement continuous into the supporting walls, secondary values shall be permitted to be doubled.~~
~~5. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength in accordance with Section 3.4.3.2.~~
~~6. Linear interpolation between values listed in the table shall be permitted.~~