A COMPARATIVE ASSESSMENT OF AN EXISTING REINFORCED CONCRETE BUILDING BY USING DIFFERENT SEISMIC REHABILITATION CODES AND PROCEDURES

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ABSTRACT

A COMPARATIVE ASSESSMENT OF AN EXISTING REINFORCED CONCRETE BUILDING BY USING DIFFERENT SEISMIC REHABILITATION CODES AND PROCEDURES

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Lateral load carrying capacities of reinforced concrete structures which are designed by considering only gravity loads or according to outdated earthquake codes can be insufficient. The most important problem for these buildings is the limited ductility of the frame elements. How to evaluate the performance of an existing structure and to what level to strengthen it had been major concerns for structural engineers.

Recent earthquakes which occurred in the Marmara Region in the last decade have increased the number of seismic assessment projects drastically. However, there was no special guideline or code dealing with the assessment of existing buildings. In order to have uniformity in assessment projects, a new chapter has been included in the revised Turkish Earthquake Code (2006).

In this study, the existing and retrofitted conditions of a reinforced concrete building were assessed comparatively by employing linear and nonlinear assessment procedures according to different seismic rehabilitation codes. The study was carried out on a six storey reinforced concrete telephone exchange building. Although there was no damage in the structure due to the recent earthquakes that occurred in the Marmara Region, the building was assessed and retrofitted in 2001 by using equivalent lateral load analysis results. The results of linear and nonlinear assessment procedures performed in the scope of this thesis, were also compared with the assessment results of this previous study.

In the nonlinear assessment procedures, pushover analysis results were used. In addition to comparison of the assessment procedures, efficiency of a widely used approximate pushover method was also investigated.

Keywords: Linear assessment, nonlinear assessment, pushover analysis, elastic equivalent lateral load procedure, performance level.

ÖZ

MEVCUT BİR BETONARME BİNANIN DEĞİŞİK DEPREM YÖNETMELİKLERİ VE YÖNTEMLER KULLANILARAK KARŞILAŞTIRMALI DEĞERLENDİRMESİ

Öztürk, İsmail
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Eski deprem yönetmeliklerine göre veya yalnızca düşey yükler altında analiz edilerek tasarlanan betonarme binaların yatay yük taşıma kapasiteleri yetersiz olabilmektedir. Bu binalardaki en büyük sorun, çerçeve elemanlarının sünek davranışlarının sınırlı olmasıdır. Mevcut binaların nasıl değerlendirileceği ve hangi düzeye kadar güçlendirileceği inşaat mühendisleri için önemli bir sorundur.

Ülkemizde geçtiğimiz on yıl içinde meydana gelen depremlerden sonra, mevcut binaların değerlendirmesi ile ilgili projelerin sayısı oldukça artmıştır. Ancak, ülkemizde mevcut bina değerlendirmesi ile ilgili olarak yürürlükte resmi bir kılavuz veya yönetmelik bulunmamaktaydı. Bu yüzden, yapılan mevcut bina değerlendirme projelerinde uyumluluk sağlamak amacıyla, yeniden düzenlenen deprem yönetmeliğine bu konu ile ilgili bir bölüm ilave edilmiştir.

Bu çalışmada, betonarme bir yapının mevcut durumu değişik yönetmeliklere göre doğrusal ve doğrusal olmayan yöntemler kullanılarak karşılaştırmalı olarak değerlendirilmiştir. Çalışmalar, telefon santral binası olarak kullanılan altı katlı betonarme bir yapı üzerinde yürütülmüştür. Çalışmaya temel oluşturan bina, yakın geçmişte Marmara Bölgesinde meydana gelen depremlerde hasar görmemiş olmasına rağmen, kullanım amacının öneminden dolayı 2001 yılında, eşdeğer statik deprem yükü yöntemi uygulanarak değerlendirilmiş ve yapılan çalışmalar sonucunda güçlendirilmiştir. Daha önce yapılmış olan bu çalışmanın sonuçları ile, bu tez çalışması kapsamında uygulanan doğrusal ve doğrusal olmayan yöntemlerin sonuçlarının da karşılaştırmalı değerlendirmesi yapılmıştır.

Doğrusal olmayan değerlendirme yöntemlerinde bina, itme analizi sonuçları kullanılanılarak değerlendirilmiştir. Ayrıca bu çalışmada değişik bina değerlendirme yöntemlerinin karşılaştırmasına ek olarak, yaklaşık bir itme analizinin yeterliliği de araştırılmıştır.

Anahtar Sözcükler: Doğrusal değerlendirme, doğrusal olmayan değerlendirme, itme analizi, elastik eşdeğer statik deprem yükü yöntemi, performans seviyesi.

To My Family

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CHAPTER I

INTRODUCTION

1.1 GENERAL

Earthquake codes in most countries generally deal with the design of new buildings. After the recent earthquakes which occurred in the Marmara Region in the last decade, the number of seismic assessment and retrofit projects has increased drastically. Since there was no official guideline for the assessment purposes, Turkish Earthquake Code (Ministry of Public Works and Settlement, 1998) was used in these retrofitting projects. Accordingly, all of the existing buildings were expected to satisfy the rules which are valid for new buildings. Since this code was incapable of predicting the existing building behavior during earthquakes, using this code in retrofit designs caused misleading results. In order to avoid these problems and to have uniformity in retrofitting projects, a new chapter has been added in the revised Turkish Earthquake Code (2006).

Generally it is thought that, linear elastic equivalent lateral load analysis is insufficient to predict the performance of a building after an earthquake. On the other hand, displacement response and failure mechanisms of frame members can be estimated well by employing nonlinear analysis tools. In the revised earthquake code, nonlinear assessment of the buildings has been allowed in addition to the linear procedures.

In this study, existing and retrofitted systems of a case study building were assessed comparatively using different codes and analysis procedures. As a part of assessment project of telecommunication buildings in İstanbul, the case study building was assessed and retrofitted in 2001 by employing reduced equivalent lateral load analysis procedure. Therefore, it was also possible to check the efficiency of the implemented retrofitting system.

1.2 OBJECTIVE

Main objectives of this study are first to compare the linear and nonlinear assessment procedures proposed in different codes and to decide the efficiency of the approximate pushover analysis procedure proposed in this study.

The study was implemented out on a 6-story telecommunication building, which is located in Gayrettepe, İstanbul. Existing and retrofitted systems were assessed according to the linear procedure proposed in the Turkish Earthquake Code 2006. Then both systems were assessed by employing the nonlinear procedures described in the Turkish Earthquake Code 2006 and FEMA 273 (ASCE, 2000) by using the pushover analysis. Analyses were carried out by using SAP 2000 V10.1 (Computers and Structures Inc., 2002) analysis program.

1.3 LITERATURE SURVEY

Although early generation of structural engineers were aware of the fact that nonlinear analysis procedures give more realistic results under the effect of cyclic loads produced by earthquakes, complexity of these methods, limited computing capacity, and considerable amount of time required to perform these analyses prevented these methods to become widespread. To overcome these problems, various simplified methods and assumptions were developed.

Inelastic time–history analysis is a common method used to determine the seismic response of structures. This procedure requires a set of carefully selected ground motion records. These selected earthquake records are applied to the analytical model and response of the structure is obtained. Clearly, inelastic time history analysis is not simple and suitable for practical purposes. Moreover, the calculated inelastic response is sensitive to the characteristics of the input motions. Therefore, selection of a set of representative acceleration time history is compulsory. In addition, this procedure requires the cyclic load–deformation characteristics of all important elements of the three dimensional soil–foundation structural systems. Since neither input nor capacities are known with accuracy, simplified methods, like pushover analysis are more suitable to estimate the strength capacity in the post elastic range. This technique may be also used to highlight the potential weak regions in structures. Pushover analysis is used to evaluate the expected performance of a structural system by predicting its strength and deformation demands under design earthquakes by a static inelastic analysis, and comparing these demands with the available capacities at the performance level of interest. Therefore, the inelastic static pushover analysis is a tool for predicting seismic force and deformation demands by applying a predefined lateral load pattern, which is distributed along the building height (Mwafy and Elnashai, 2001).

The most important and basic assumption of the pushover analysis is that the structures are forced to deform in their first mode shape, thus a multi–degree of freedom system is simplified as an equivalent single–degree of freedom system. This assumption was proposed by Pique (1976). The comparative analyses showed that the assumption yields reasonable results in simple, regular structures (Krawinkler and Seneviratna, 1998).

Saiidi and Sozen (1981) extended this equivalent single degree of freedom system and proposed a “low–cost” analytical model for the calculation of displacement histories of multistory reinforced concrete structures subjected to strong ground motions, and they called this model as “Q–model”.

Q model consists of an equivalent mass, a viscous damper, a massless rigid bar having equivalent height and a rotational spring. The model involves two simplifications:

1. Reduction of a multi degree–of–freedom (MDOF) model of a structure to a single degree–degree of–freedom oscillator.
2. Approximation of the varying incremental stiffness properties of the entire structure by a single nonlinear spring.

The results, based on the Q–model, were compared with displacement histories in earthquake–simulation experiments of eight small–scale structures. The comparisons showed that the overall performance of the Q–model in simulating response in high–amplitude and low–amplitude ranges was satisfactory.

Due to low cost of Q–model, the usual ranges of doubt associated with information on expected ground motion and static structural response of actual structures, the sufficiency of drift estimates for making design decisions, it was concluded that Q–model provides a suitable alternative to the elaborate planar nonlinear response models.

Many researchers have used the same assumption, reduction of the multi degree–of–freedom system to an equivalent single degree–of–freedom system. Advances in the computer technology made the nonlinear methods more practical. As a result, these nonlinear methods are becoming more popular and even some country codes refer to these methods (Kappos and Manafpur, 2001). The seismic design part of the Eurocode (European Comitee for Standardization, 1996), EC8 recognizes that inelastic analysis might be used in the design procedure. On the other hand, the New Zealand Code (Standards New Zealand, 1992) is clear in specifying that the purpose of using the inelastic procedures might be either to calculate strength requirement in yielding members, or to assess inelastic demands.

Kappos and Manafpur (2001) proposed seismic design procedure that lead better seismic performance than the standard code procedure and leads to more economic design of transverse reinforcement in the members that develop very little inelastic behavior even under very strong earthquakes. This procedure is a combination of the code proposed linear elastic method and the nonlinear time history or pushover analysis. First, the flexural design of the beams of the structure is made under the effect of elastic earthquake loads and gravity loads. Then the structure is analyzed by time history or static pushover analysis method, using some percentages of the gross cross sectional properties at the serviceability or immediate occupancy performance level. From this nonlinear analysis performed by Kappos and Manafpur (2001), two performance criteria are checked;

1. Maximum drifts do not exceed the limits corresponding to damage requiring repair in the non – structural elements. If this condition cannot be satisfied, stiffening of the structure is necessary by increasing the cross – section dimensions.
2. Plastic rotations in beam critical regions do not exceed the value corresponding to “non–tolerable” cracking. If the specified ductility limits are exceeded in some members, the corresponding reinforcement is increased.

Accordingly, the nonlinear analysis of the same model for the repairable damage or life safety performance level is revised. If some of the beams has yielded during the previous analysis step, the beam reinforcements are also revised. Design and detailing of longitudinal reinforcements for the columns and shear walls are carried out according to this analysis. Using this procedure, structures can be designed more economically and for a better seismic performance.

The seismic design requirements in the Building Standard Law of Japan were revised in June 2000. According to this new code, structures are designed using performance–based procedures. The performance objectives are life safety and damage control of a building at two corresponding levels of earthquake motions. In this code, the procedure used is the capacity spectrum method. The structural response is examined by this performance – based method by comparing the linearly elastic demand spectrum of design earthquake motions and the capacity curve of an equivalent single degree–of–freedom system (Otani S., Hiraishi H., Midorikawa M., and Teshigawara M., 2000).

Besides the improvements in the nonlinear analysis procedures, structural engineers have been researching the applicability of the approximate methods, in order to overcome the problems associated with the uncertainties and complexities of the dynamic methods such as time history analysis. Krawinkler (1995), and Mwafy and Elnashai (2001) studied the validity of the assumptions and accuracy of the static pushover analysis method. They summarized basic concepts in which pushover analysis can be based, identified conditions under which the pushover would provide adequate information, and perhaps more importantly, identified cases in which the pushover predictions would be inadequate or eve misleading.

From these studies (Krawinkler, 1995 and Mwafy and Elnashai, 2001), following conclusions about pushover analysis can be drawn:

- If the pushover analysis is implemented with caution and good judgment, it will be a great improvement over presently employed elastic evaluation procedures.
- For structures that vibrate primarily in the fundamental mode, pushover analysis will give good estimates of global and local inelastic deformation demands, design weaknesses that include story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle elements such as columns and connections.

- Static pushover analysis is more suitable for low-rise and short-period frame structures. In addition, the results of static pushover analysis are in good agreement with those of dynamic analysis for well-designed buildings with structural irregularities.
- For structures in which higher mode effects are significant and story shear force vs. story drift relationships are sensitive to applied load pattern, pushover analysis may give inaccurate deformation estimates. Applying more than one loading pattern can solve this problem.
- A conservative prediction of capacity and reasonable estimation of deformation was obtained by the simple triangular or the multi – model load distribution. In the elastic range, the same load patterns underestimated slightly the demands of the same buildings. However, the uniform load pattern provides a conservative prediction of seismic in the elastic range.
- Pushover analysis may detect only first local mechanism that will form in an earthquake. However, it may not expose other weaknesses that will form when dynamic characteristics of the structure change after the formation of first local mechanism.

Pushover analysis can be implemented with other evaluation procedures such as inelastic dynamic analysis with a representative suite of ground motions, elastic dynamic (modal) analysis using the unreduced design spectrum and a suitable modal combination procedure (SRSS, CQC) if higher mode effects are important (Krawinkler and Seneviratna, 1998).

Pushover analysis needs further developments. In order to provide the accurate and realistic analysis of highly irregular structures, one development would be the continuous assessment of the effect of inelasticity on the load distribution used, taking into account the shape of the spectrum. Second one would be analysis of larger sample of buildings that include high–rise structures with heavily irregular strength distribution.

1.4 SCOPE

This thesis consists of seven chapters and three appendices as follows;

Chapter I: Introduction to the subject, background information about the assessment procedures, object and scope of this study.

Chapter II: Methodology of the linear and nonlinear assessment procedures.

Chapter III: General information about the case study building, analytical models for both existing and strengthened states.

Chapter IV: Step by step procedure for the approximate pushover analysis procedure and its verification.

Chapter V: Comparative assessment results for the existing system.

Chapter VI: Comparative assessment results for the retrofitted system.

Chapter VII: Summary, conclusions and recommendations for future studies.

Appendix A: Interaction diagrams for columns and shear walls at the typical floor.

Appendix B: Existing system, life safety performance level, graphical representation of the comparative assessment results.

Appendix C: Retrofitted system, life safety performance level, graphical representation of the comparative assessment results.

CHAPTER II

METHODOLOGY

2.1 GENERAL

Earthquake codes primarily focus on the design of new buildings. Turkish Earthquake Code 1998 is not an exception. As it was stated in the previous chapter, after recent earthquakes it became compulsory to have a guideline or code for assessment projects. Therefore, a new chapter has been added in the revised Turkish Earthquake Code 2006.

In this comparative study, Turkish Earthquake Code 2006 and FEMA 273 are employed comparatively to determine the performance level of a case study building.

2.2 DATA COLLECTION

The first step of an assessment project is data collection. Turkish Earthquake Code 2006 and FEMA 273 propose similar procedures for data collection. In the Turkish Earthquake Code 2006, necessary data to be collected for the existing buildings are classified under four headings. These are;

1. Determination of building configuration, structural system and geometry;
2. Determination of component properties by destructive and non destructive testing methods
3. Determination of site characteristics and geotechnical properties by bore holes etc.
4. Effect of adjacent buildings.

According to data collected from the existing buildings, the Turkish Earthquake Code 2006 defines three knowledge levels and three knowledge factors correspondingly. In FEMA 273 on the other hand, there are only two knowledge levels and corresponding factors. In Table 2.1 knowledge levels and factors according to the Turkish Earthquake Code 2006 and FEMA 273 are summarized. In the Turkish Earthquake Code 2006 and FEMA 273, extensive explanations of knowledge levels and corresponding factors are given.

Table 2.1 Knowledge Levels

Knowledge Level	Knowledge Factor According to the Turkish Earthquake Code 2006	Knowledge Factor According to FEMA 273
Limited	0.75	0.75
Moderate	0.90	Not defined
Comprehensive	1.00	1.00

2.3 STRUCTURAL PERFORMANCE LEVELS AND RANGES

The structural performance levels and ranges are the same in the Turkish Earthquake Code 2006 and in FEMA 273. In both guidelines there are mainly three structural performance levels defined. These are;

- Immediate Occupancy
- Life Safety
- Structural Stability

Damage ranges are described as the ranges in between given performance levels. These ranges are given as;

- Minimum Damage
- Limited Damage
- Limited Safety
- Collapse Range

If a structural member cannot reach the immediate occupancy level, this member is accepted as in the minimum damage range. The structural member is considered in the limited damage range if its performance level is in between immediate occupancy and life safety. It is accepted in limited safety range if its performance level is between life safety and structural stability. And finally it is considered as totally failed if its performance level is beyond structural stability.

For the failure types, similar classifications are used in the Turkish Earthquake Code 2006 and in FEMA 273. In both documents, if the governing component failure mode is shear or excessive axial compression, failure type is classified as brittle, if failure mode is flexure it is classified as ductile.

2.4 LINEAR ELASTIC PROCEDURES TO DETERMINE STRUCTURAL PERFORMANCE LEVEL

2.4.1 Analysis Methodology

Almost all of the seismic design codes used in different countries propose equivalent static and/or dynamic linear procedures to analyze a structure. Especially, equivalent static load method is a well understood and used procedure for uniform buildings having no particular irregularity.

For assessment of existing buildings, the revised code proposes a linear static method. Although there are some differences in calculation of base shear, general philosophy is consistent with the method mentioned in FEMA 273. For the case study building, the linear elastic procedure explained in the Turkish Earthquake Code 2006 was applied and the results were compared with the nonlinear procedures.

In the revised code there are some restrictions for equivalent static lateral load procedure. For example, the building must have maximum of eight floors excluding basement floor, and the building height above basement floor must not exceed 25 m. In addition to these, torsional irregularity coefficient, calculated without considering additional eccentricity, must be less than 1.4.

In the calculation of elastic earthquake loads, structural behavior factor and building importance factor are not applied. These two factors are taken as 1.0. But the earthquake load applied to the structure is multiplied with the coefficient λ. This coefficient is taken as 1.0 for buildings having maximum 2 floors excluding basement floor. The coefficient is taken as 0.85 for the other buildings. Furthermore, no additional eccentricity is applied to the structure. That is, unreduced elastic earthquake forces are applied to the mass center in two orthogonal directions.

As a result of the brief description given above, equivalent elastic earthquake loads can be calculated from Equation 2.1. In this equation, lateral force depends on the seismicity and the soil profile of the region, the fundamental period (T) and weight (W) of the structure.

$$V_t = WA_0S(T)\lambda \tag{2.1}$$

Calculated elastic earthquake forces are distributed through the floors according to the method given in the Turkish Earthquake Code 2006.

2.4.2 Assessment of Structural Members

According to the code, the structural elements are classified as confined and unconfined according to their lateral reinforcement. If an element satisfies the required transverse reinforcement rules given in the code, this element is considered as confined, otherwise it is accepted as an unconfined element. Moreover, according to the anticipated failure mechanism, a structural element is classified as ductile or brittle. This subject was discussed in previous sections of this chapter.

As a result of analysis of a structure under the effect of elastic earthquake forces and vertical loads, demands at each structural element are obtained. Demand to capacity ratios are evaluated at the critical sections of structural elements. In calculating the capacity, existing material strength values are used, but the evaluated capacities are multiplied by the appropriate knowledge factor.

For ductile beams, columns and shear walls for which flexure is the governing failure type, demand to capacity ratio is calculated as the ratio of demand which is obtained under the effect of earthquake forces only, to residual moment capacity. Residual moment capacity at a specific section can be calculated as the difference of moment capacity at that section and moment obtained from vertical load analysis.

For brittle beams, columns and shear walls for which shear is the governing failure mode, demand to capacity ratio is calculated by dividing the shear force at the critical section, obtained from the analysis, to the shear capacity of the section. Similarly for the structural elements where governing failure type is compression, demand to capacity ratio is evaluated by dividing the axial force obtained from the analysis to axial load capacity of the section.

All beams, columns and shear walls demand to capacity ratios are compared with the acceptability criteria, determined according to the target performance level. Acceptability criteria for the beams and columns are given in the Turkish Earthquake Code 2006. The main parameters affecting the acceptability criteria are the mode of failure and confinement of the structural element.

2.5 NONLINEAR METHOD TO DETERMINE STRUCTURAL PERFORMANCE LEVEL

2.5.1 Pushover Analysis Methodology

In the Turkish Earthquake Code 2006, nonlinear methods are also allowed for assessment purposes. There are mainly two methods explained in the revised code as well as in FEMA 273. These methods are listed below;

- Pushover analysis using equivalent static earthquake load
- Time history analysis

In the assessment of the case study building with nonlinear procedures, according to the Turkish Earthquake Code 2006 and FEMA 273, pushover analysis using equivalent static lateral load was used.

Code-based linear elastic analysis considers inelastic seismic response indirectly and implicitly. However, during severe earthquakes, inelastic behavior is unavoidable. Thus, the use of linear procedures may lead to misleading results on structural demands and underestimates the displacements.

Nonlinear procedures help to describe the inelastic behavior of the structure directly. Displacement history can be obtained more realistically by nonlinear procedures.

The objectives of nonlinear procedures are to determine the lateral load resisting capacity of the structure and to obtain more realistic displacement demands during a ground motion.

Static pushover analysis is the process of pushing the structure laterally with a predetermined loading pattern in increments until the structure reaches its ultimate deformation state. The purpose of pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation demands under design earthquakes by means of a static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest (Krawinkler H., and Seneviratna G.D.P.K, 1998). From the analysis, it is expected to obtain the following performance parameters of the structural system:

- Global deformations
- Interstorey drifts
- Inelastic element deformations

The most important information that is obtained from the pushover analysis is the capacity curve, which is plotted as applied base shear force versus top story displacement. From this analysis procedure, following response characteristics can be obtained (Krawinkler H., and Seneviratna G.D.P.K, 1998):

- Inelastic deformation demands of the elements, which exhibit nonlinear behavior.

- Consequences of the plastic hinging mechanisms occurring during the earthquake ground motion.
- Reliable interstorey drifts obtained by including the stiffness and strength reductions and P - Δ effects.
- More realistic force demands on potentially brittle members such as shear force demand in the beam – column connections and axial force demands in the columns.
- The effect of the lateral force applied on the individual members and on the overall system.
- Redistribution effects on the overall capacity of the structural system and verification of the adequacy of the load path.

In ATC 40 (Applied Technology Council, 1996) the simple step-by-step procedure for the static pushover method is proposed as follows:

- Two or three dimensional computer model is prepared.
- For performance check, each element is classified as either primary or secondary.
- Gravity loads are applied to the system.
- A predetermined lateral force pattern is applied to the structural model.

According to ATC 40 there are various acceptable lateral force patterns:

1. Simply a single horizontal load is applied at the top level of the structure.
2. A load pattern proportional to the story heights h_i and weights w_i, as described in most codes can be applied.
3. A load pattern can be in proportion to the product of story masses and the first mode shape of the elastic structure. Since the basic assumption of the pushover analysis that the fundamental mode of vibration is the predominant response of the

structure, this load pattern is the most used one. The revised code proposes to use this load pattern.

4. Until the first yielding of the system, a load pattern proportional to the first mode shape of the structure is applied, and then the load pattern is modified according to the deflected shape of the structure.
5. A load pattern same as 3 and 4 is applied, but the higher mode effects are also included by modifying progressively the applied loads in proportion to a mode shape rather than the fundamental mode shape. However, the higher mode effects may be important for structures having fundamental period larger than 1 second.

- Member forces are calculated under the effect of both lateral and gravity loads.
- The applied lateral force magnitude is adjusted so that some elements or groups of elements reach their capacity.
- The magnitude of the applied base shear and the roof displacement are recorded.
- Structural model is revised using zero or very small stiffness for the yielded elements.
- A new lateral load increment is applied so that some elements or group of the elements yields and this procedure is continued until the structure reaches its ultimate state or at a predetermined displacement level.
- The recorded lateral forces' magnitudes and top floor displacements are plotted (capacity curve) as shown in Figure 2.1.

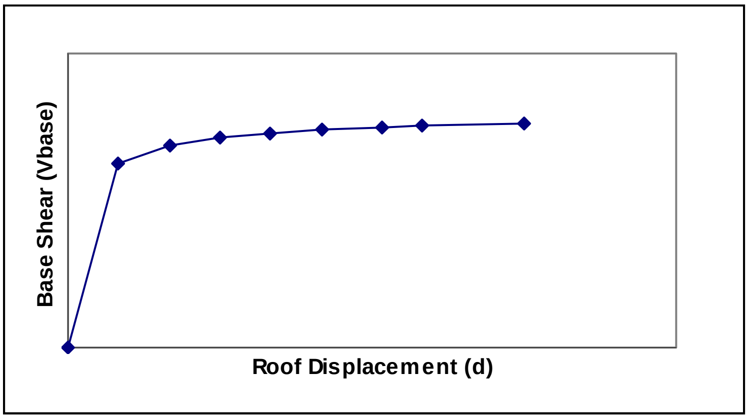


Figure 2.1 Typical Force–Displacement Relationship Obtained from Pushover Analysis

The procedure given above is called the “force controlled” pushover analysis. In the analysis when the iterations are close to ultimate state, since the overall structure loses its stiffness considerably, the control node displacement, or top floor displacement, increases abruptly without significant change of the applied lateral load. To overcome this disadvantage, Allahabadi (1987) proposed a procedure called as “displacement controlled” pushover analysis. In this analysis, the applied lateral force magnitude is increased at each step such that the control node reaches the predetermined displacement increment level. At each step, the model is revised for the yielded elements and this process is continued until the ultimate stage or a predetermined displacement level.

2.5.2 Target Displacement

Although, there is no significant difference between Turkish Earthquake Code 2006 and FEMA 273 in constructing the capacity curve, there are some differences for determining the target displacement.

Target displacement can be described as the displacement demand of the structural system under an earthquake ground motion. In the pushover analysis, it is assumed that the target displacements for the multi degree-of-freedom structure can be represented by a corresponding single degree-of-freedom system, which has a constant modal shape. There is not a unique way to find out this equivalent displacement and to relate it with the multi degree-of -freedom system.

Displacement coefficient method, which is proposed by FEMA 273 to determine target displacement, is the simplest way to predict target displacement. The step–by–step procedure proposed in ATC 40 can be summarized as follows:

- A bilinear representation of the capacity curve is constructed as in Figure 2.2.
- The effective fundamental period is calculated as;

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \qquad (2.2)$$

- Calculate the target displacement δ_t as;

$$\delta_t = C_0 C_1 C_2 C_3 S_a (T_e^2 / 4\pi^2) \qquad (2.3)$$

The detailed calculation procedure for these modification factors can be found in section 3.3.3.3 of FEMA 273.

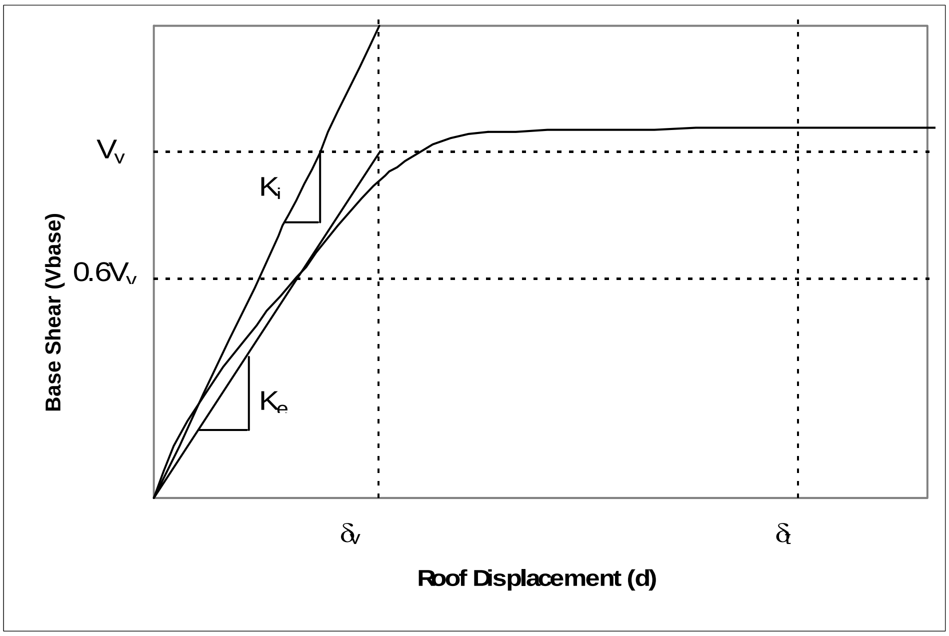


Figure 2.2 Bilinear Representation of Capacity Curve for Displacement Coefficient

In the Turkish Earthquake Code 2006, general methodology to determine the target displacement is different than that explained in the above paragraphs. A step by step procedure can be summarized as follows;

- Calculate the modal acceleration, corresponding to the first mode of the structure, at each step

$$a_1^i = \frac{V_1^i}{M_1} \tag{2.4}$$

- Calculate the modal displacement, corresponding to the first mode of the structure, at each step

$$d_1^{(i)} = \frac{u_{N1}^{(i)}}{\Phi_{N1}\Gamma_1} \tag{2.5}$$

$$\Gamma_1 = \frac{L_1}{M_1} \tag{2.6}$$

- Calculate the modal displacement demand for different exceedance probability of earthquake acceleration. Modal displacement demand is equal to nonlinear spectral displacement.

  $$d_1^{(p)} = S_{d1i} \tag{2.7}$$

- Apply the procedure to determine the nonlinear spectral displacement that is given in detail in Chapter 7C of Turkish Earthquake Code 2006.

- Calculate the top displacement demand for the last step i = p;

  $$u_{N1}^{p} = \Phi_{N1}\Gamma_1 d_1^{p} \tag{2.8}$$

All the member responses and internal forces are obtained at the load level corresponding to the target displacement.

2.5.3 Assessment of Structural Members

For inelastic methods, deformation demand of the structural members determines the performance. In order to evaluate deformation demand of the structural members, Turkish Earthquake Code 2006 and FEMA 273 uses similar methods.

In the Turkish Earthquake Code 2006, performance levels of the structural members are related to the strain capacity of concrete and reinforcement at the critical sections.

Strain capacity at the critical sections is obtained for total curvature demand by using moment curvature relation of the section. Total curvature demand is defined as the sum of plastic curvature demand and equivalent yield curvature.

$$\phi_t = \phi_y + \phi_p \tag{2.9}$$

Equivalent yield curvature is obtained directly from the moment curvature diagram. Plastic curvature demand depends on plastic rotation angle obtained from inelastic analysis results. Plastic rotation angle must be calculated or obtained at target displacement level. Plastic curvature demand at the target displacement level can be calculated as follows;

$$\phi_p = \frac{\theta_p}{L_p} \tag{2.10}$$

In this equation θ_p represents the plastic rotation angle obtained from the analysis result. L_p is defined as plastic hinge length. In many documents it is assumed as half of the section height.

For all critical sections of yielded members, strain capacity of extreme concrete fiber, confined concrete fiber and reinforcement are calculated for corresponding total curvature demand and these values are compared with the limit values.

In FEMA 273 on the other hand, performance criteria simply depends on plastic rotation angle obtained from analysis results at target displacement level. For all critical sections of yielded members, plastic rotation angles are calculated and these values are compared with the limit values given in the guideline.

2.6 DETERMINATION OF BUILDING PERFORMANCE LEVEL

Building performance level is related with the damages those are expected during earthquake ground motion. Building performance level that is decided according to damage state of the building is compared with the objective performance level. In

FEMA 273 and Turkish Earthquake Code 2006 there are mainly three building performance levels. The performance levels and expected system behavior according to ATC 40 description, for these performance levels can be given as follows:

Immediate Occupancy Performance Level (IO): Post earthquake damage state in which only very limited structural damage has occurred. The frame systems stiffness and strength retain almost their pre-earthquake state. Risk of life threatening injury is very low. The structural system members may need only minor repairs (ATC 40, 1996). According to Turkish Earthquake Code 2006 in order to accept the building in this performance level, columns and shear walls must be in minimum damage range and only 10% of beams are allowed to be in limited damage range at each floor. If the building satisfies this performance level, retrofitting is not required.

Life Safety Performance Level (LS): Significant damage to the structural members has occurred. However, collapse of the frame system is prevented. Some structural members may be damaged severely, but these damages do not lead to total or partial collapse of the structural system. Injuries may occur due to structural or non-structural damages, but life threatening injury risk is very low (ATC 40, 1996). For the code, acceptability criteria for this performance level are; at each floor only 20% of the beams and some columns may be in limited safety range. Total shear force resisted by the columns in limited safety range must not exceed 20% of earthquake load applied at that floor. Building may need to be retrofitted according to number and distribution of the members which exceed the life safety performance level.

Collapse Prevention Performance Level (CP): The structural system is close to partial or total collapse. Substantial damage has occurred at the structural system. The frame system loses its lateral load carrying capacity. However, gravity load carrying system must continue to service. In the system large permanent deformation has occurred. Significant risk of injury due to falling hazards may exist. Repairing of the structural system is not practical, economically and technically (ATC 40, 1996). According to the Turkish Earthquake Code 2006, in order to accept the building in this performance level, at each floor, only 20% of the beams and some columns may be in the collapse level. Total shear force resisted by the columns in collapse level must not exceed 20% of earthquake load applied at that floor. The structure in that performance level must be retrofitted.

In addition to these requirements for building performance levels, the code also restricts the interstorey drift ratio at each performance levels.

2.7 TARGET PERFORMANCE LEVELS FOR BUILDINGS

In almost all of the seismic design codes like the Turkish Earthquake Code 2006, design spectrum corresponds to 10% probability of exceedance in 50 years. However, for assessment purposes, building performance levels under earthquake spectra having exceedance probability of 2% in 50 years and 50% in 50 years may also need to be checked. Acceleration spectrum having 2% exceedance probability in 50 years corresponds approximately to 1.5 times the design spectrum. Spectrum having 50% exceedance probability in 50 years corresponds approximately to half of the design spectrum.

As it was stated in previous paragraphs, basically there are three performance levels defined in the codes and guidelines. Target performance level of a building depends on usage and type of the building. In the Turkish Earthquake Code 2006 and FEMA 273, for different types of buildings, different target performance levels are defined.

2.8 RETROFITTING OF REINFORCED CONCRETE STRUCTURES

For the buildings that do not satisfy the target performance levels or that have insufficient structural behavior under the earthquake loads, retrofit may be required. According to type of structural deficiency, retrofitting may be applied to individual elements, or structure may be retrofitted so that overall structural behavior is improved.

Practical ways of individual member retrofitting explained in most of the guidelines are listed below;

- Column jacketing
    - Reinforced concrete jacketing
    - Jacketing using steel profiles
    - Carbon fiber wrapping
- Beam jacketing
    - Jacketing by using additional mechanical stirrups
    - Carbon fiber wrapping
- Retrofitting of masonry infill walls
    - Retrofitting by plaster with mesh reinforcement
    - Retrofitting by carbon fiber
    - Retrofitting by prefabricated concrete panels

In order to improve overall structural behavior under the effect of earthquake loads, different methods are used. Some of them can be given as;

- Reinforced concrete infill frames
- Adding new reinforced concrete walls adjacent to existing frame system
- Adding new frame systems to the existing structure

In addition to these retrofitting methods, structural safety may be satisfied by reducing earthquake forces acting on the structure. This can be achieved by either decreasing overall mass of the structure or by introducing base isolators to the structure. In our country, although decreasing overall mass of the structure is used occasionally, introducing base isolators is a new concept and this method has been applied to only a few structures.

CHAPTER III

THE CASE STUDY BUILDING AND ITS ANALYTICAL MODELLING

3.1 THE CASE STUDY BUILDING

The case study building is located in Gayrettepe, İstanbul. The building was designed in 1972. It is used as a telephone exchange building. Although no damage was observed in the building after the recent earthquakes, the function of the building makes it important for seismic performance evaluation. As a result of the seismic assessment of the existing system, the structure was retrofitted in 2001. A photograph taken from outside of the building is shown in Figure 3.1.

The building is a six-storey reinforced concrete frame structure. It has a rectangular plan with dimensions 16.20 m in longitudinal and 32.40 m in transverse directions. Its plan area is approximately 525 m^2. The heights of the first two stories are 3.50 m. Other stories have 4.80 m. height. This is unusual when compared with regular reinforced concrete structures. Very heavy phone exchange equipments are located on the floors. This structure was selected as case study building because it is regular and there exist extensive data about the building.

3.2 THE EXISTING STRUCTURAL SYSTEM

The structural system is made up of reinforced concrete moment resisting frames in both orthogonal directions. There are four axes in the y direction and nine axes in the x direction. There are no shear walls in the structural system. This makes the building very flexible under the effect of lateral forces.

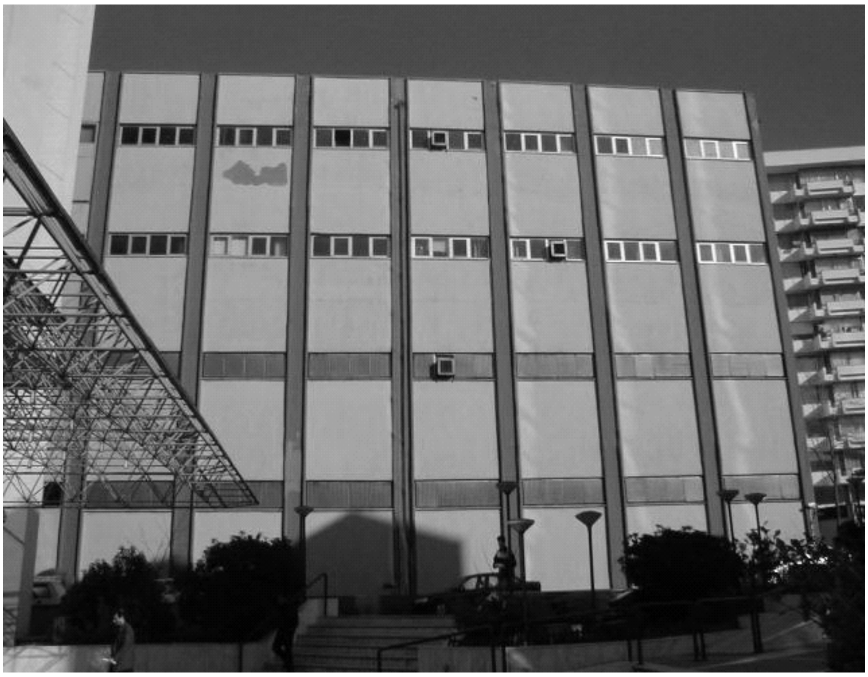


Figure 3.1 Case Study Building

The column dimensions were 60x60 cm on interior axes and 30x80 cm on exterior axes on the first two floors. Dimensions of these columns were reduced to 60x45 cm and 25x80 cm on the interior and peripheral axes respectively after the second floor. Typical beam dimensions are 25x60 cm for interior frames and 25x115 cm for peripheral frames. Floor plans of the building are given in Figure 3.2-3.4.

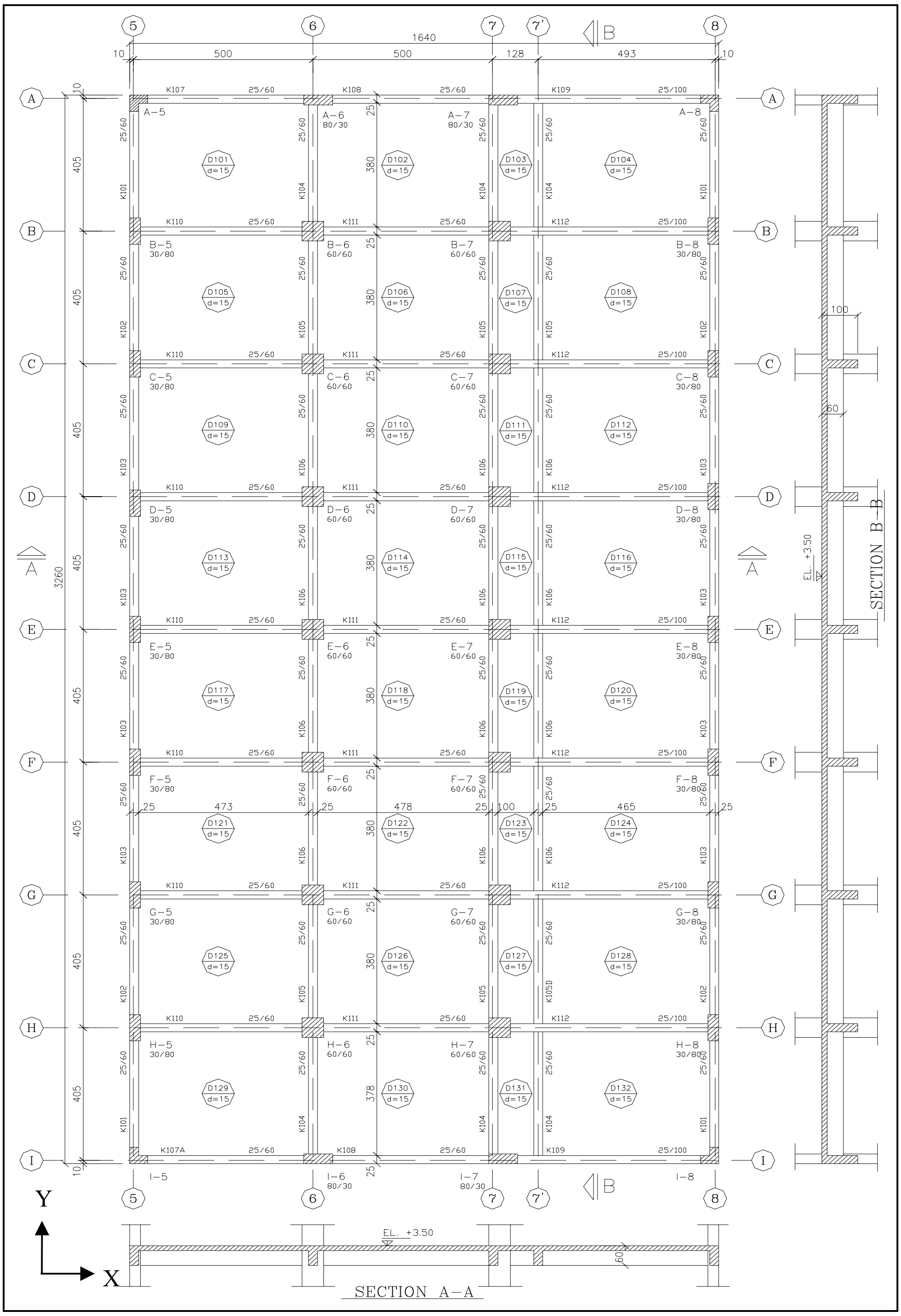


Figure 3.2. Existing System, First Floor Plan

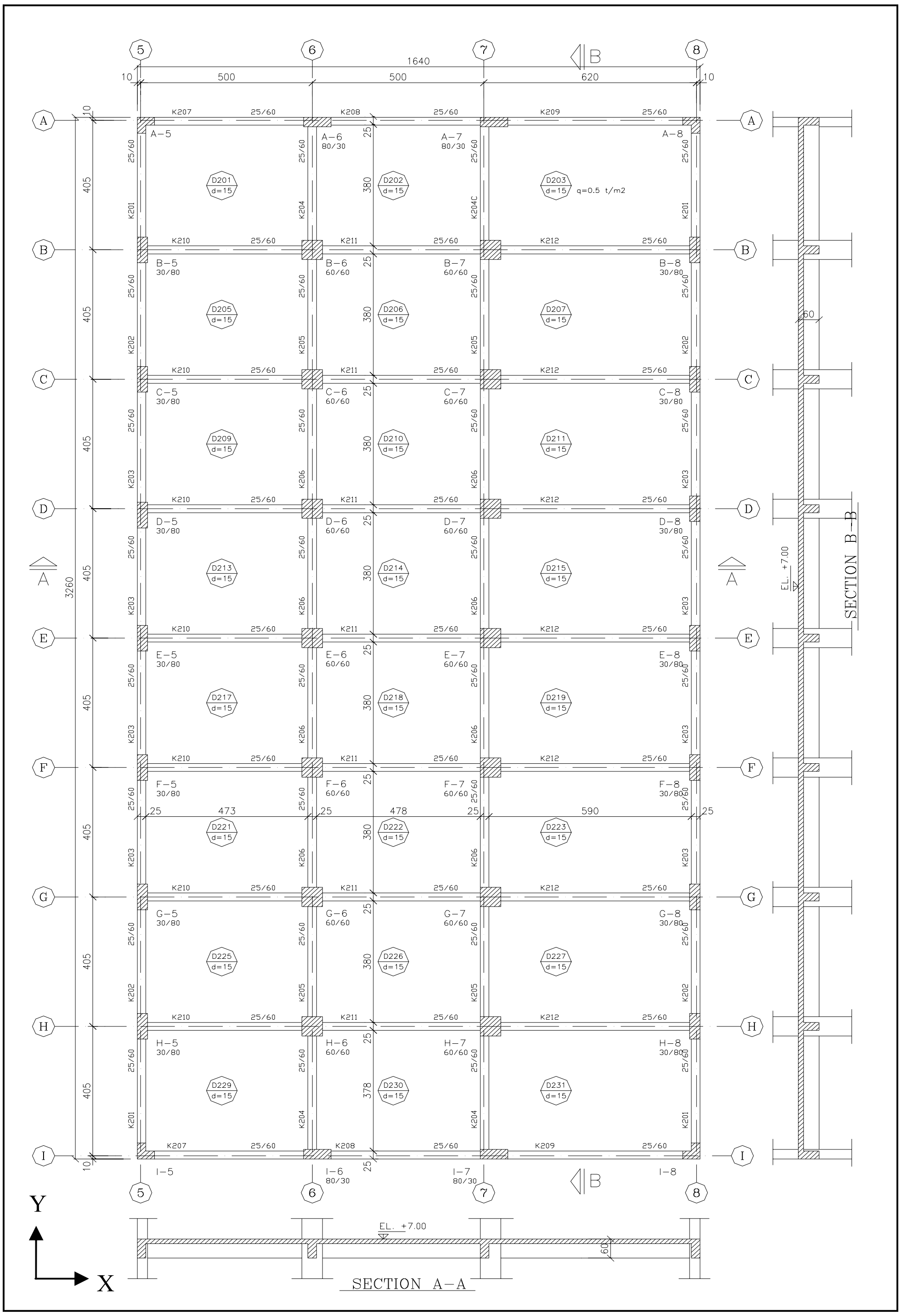


Figure 3.3 Existing System, Second Floor Plan

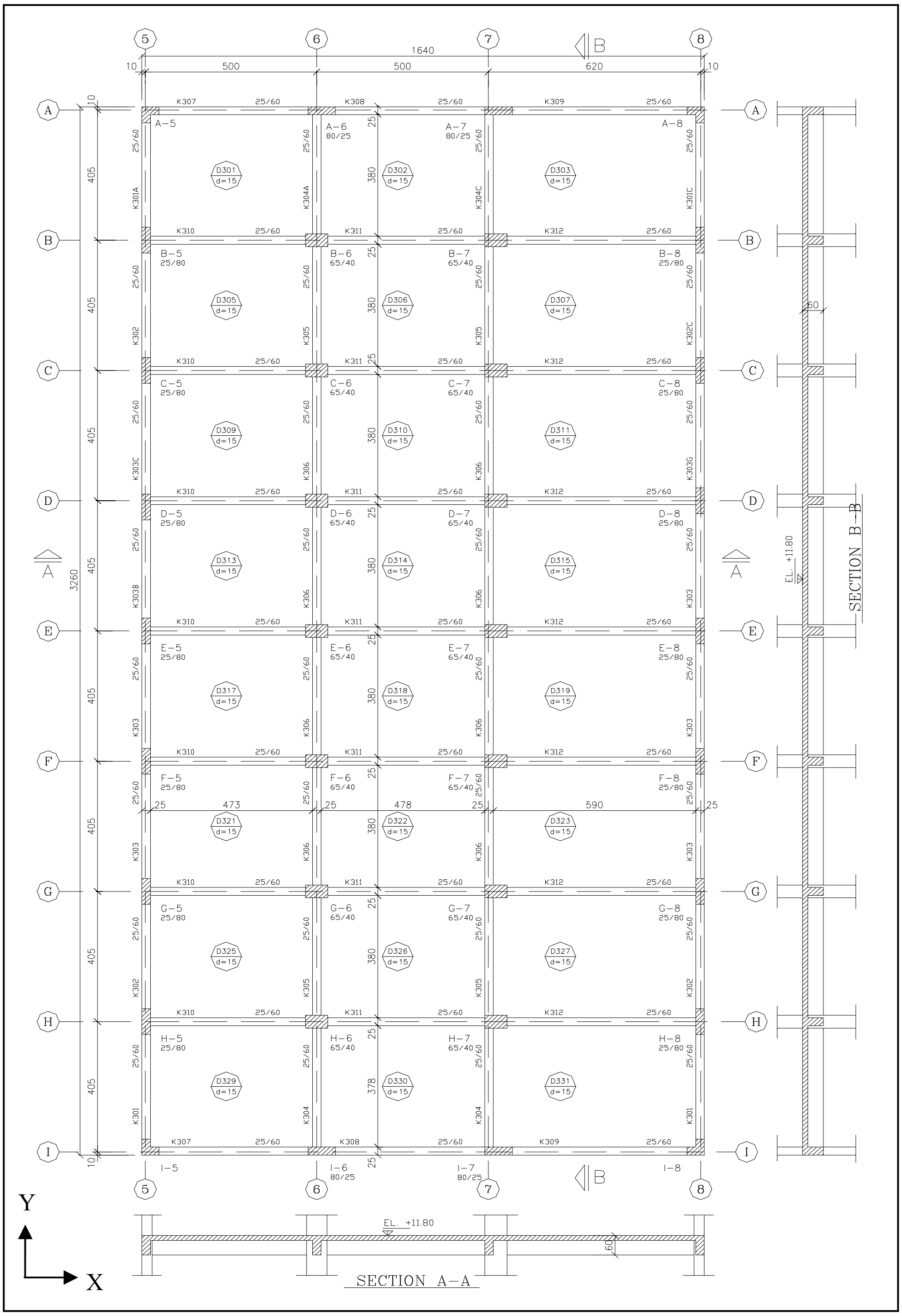


Figure 3.4 Existing System, Third, Fourth, Fifth and Sixth Floor Plans

In the detailing of frame members, there were no confined zones as the code proposes. Because of this, a ductile behavior cannot be expected from the structure during an earthquake ground motion excitation.

Reinforced concrete slabs having a thickness of 15 cm were used throughout the building. The peripheral masonry infill walls were made up of two layers of hollow bricks. There were lesser amounts of interior brick walls in the building.

Although the structure was taller and heavier as compared to usual reinforced concrete structures, the foundation system was composed of single column footings. These footings were connected to each other by tie beams in each direction. Dimensions of the footings were variable but their height was designed as 100 cm.

3.3 CONSTRUCTION MATERIALS AND SOIL CONDITIONS

In the design projects, there was no information on concrete and reinforcement grades. To determine the concrete grade used in the structure, three concrete samples were taken from each floor for testing. Compressive strength values obtained are given in Table 3.1. The average concrete strength was found as 20 MPa. Reinforcement grade available in the frame members were found as S220 MPa for both longitudinal and transverse reinforcements.

Soil properties were determined by tests. A borehole of 15 m. depth drilled near the building and samples taken from this hole were subjected to necessary laboratory tests. According to these tests;

- Soil profile in the 5 m. is a natural fill and there is mudstone formation below.
- There is no ground water in this area.
- Liquefaction is not a probable risk for the soil.

- The soil group is B and the soil class is Z2 according to the Turkish Earthquake Code 2006. Allowable stress for soil is about 2 kgf/cm^2.

Table 3.1 Concrete Test Results

Sample No	Floor No	Member Axes	Comp. Strength (Mpa)
1	1	D-6 column	21
2	1	E-5 column	19
3	2	F-7 column	21
4	2	I-6 column	23
5	2	C-7 column	20
6	2	F-8 column	22
7	3	E-5 column	19
8	3	G-8 column	22
9	3	H-6 column	25
10	4	A-6 column	22
11	4	A-8 column	21
12	4	B-7 column	20
13	5	C-5 column	19
14	5	D-8 column	22
15	5	D-6 column	21
16	6	E-8 column	23
17	6	C-6 column	21
18	6	H-5 column	24
Standard Deviation		**Average**	**Average – Standard Deviation**
1.7		21.4	19.7 ~ 20.0

3.4 THE RETROFITTED STRUCTURAL SYSTEM

Adding new shear walls in to the existing moment resisting frames of the case study building is the basic retrofitting system. In this study, effectiveness of this retrofitting system is investigated.

In Table 3.2 total area of shear walls in each direction and its ratio to floor area is presented. As can be observed form this table, amount of shear walls is higher as compared with ordinary retrofit projects.

Table 3.2 Area of Shear Walls

Typical Floor Area (m^2)	**Area of Shear Walls in x Direction (m^2)**	**Area of Shear Walls in y Direction (m^2)**	**Shear Wall Area Ratio in x Direction (%)**	**Shear Wall Area Ratio in y Direction (%)**
535	5.6	8.1	1.05	1.51

The retrofitting scheme was directly adopted from the rehabilitation project which was applied to strengthen the system by the building owner. This system was analyzed using equivalent lateral load procedure according to the Turkish Earthquake Code 1998. As it was stated previously, at beam column joints no special precautions required by the Turkish Earthquake Code 1998 were taken. However, in assessment of the structure, elastic earthquake loads were reduced by using a reduction factor of 4. That is, in the assessment of the case study building, reduction factor R was considered as 4, although the structure cannot satisfy the requirements which must be satisfied in order to use this reduction factor. The strengthened scheme of the case study building which is adopted from the rehabilitation project is shown in Figures 3.5-3.7.

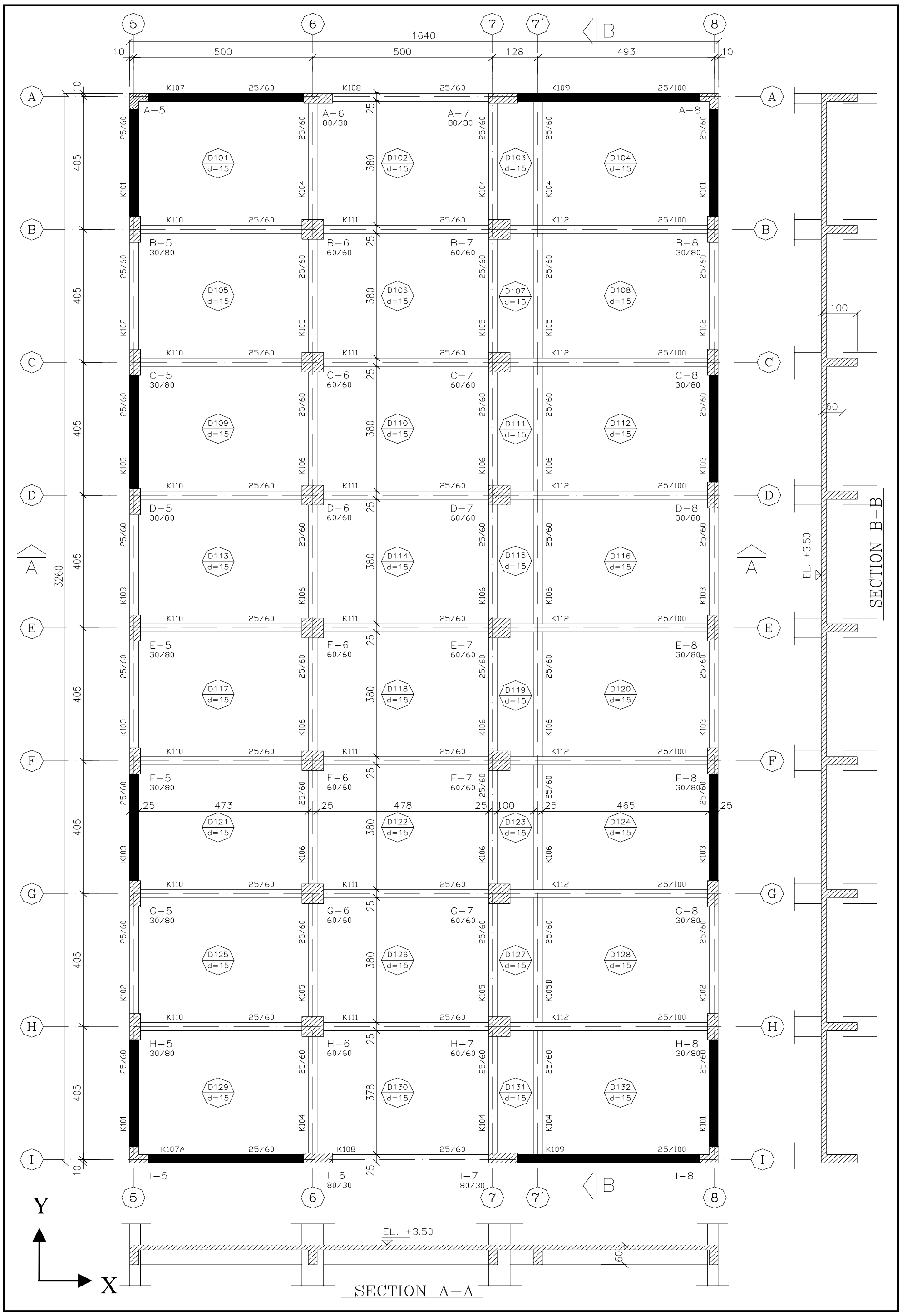


Figure 3.5. Retrofitted System, First Floor Plan

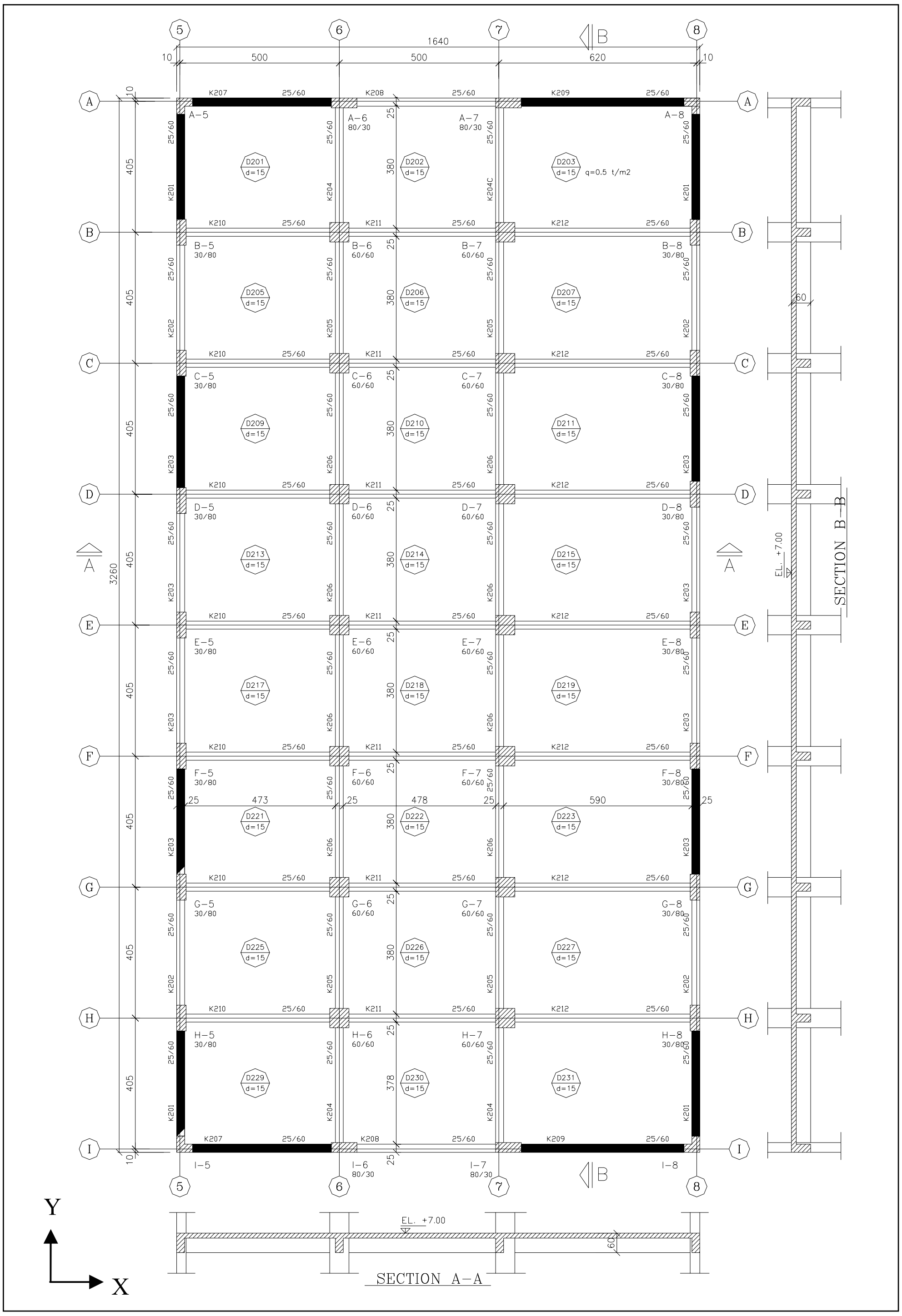


Figure 3.6. Retrofitted System, Second Floor Plan

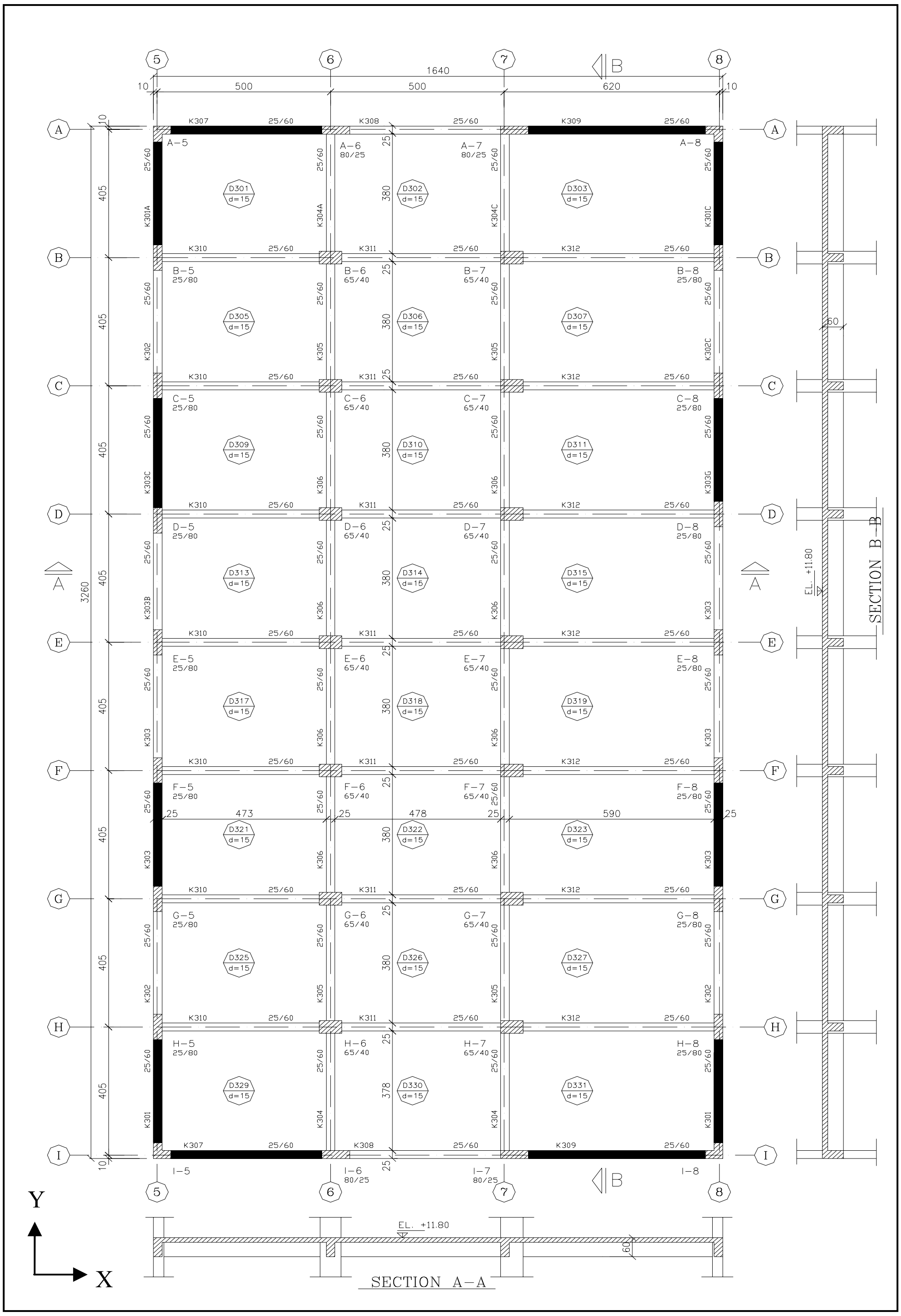


Figure 3.7. Retrofitted System Third, Fourth, Fifth and Sixth Floor Plans

3.5 GENERAL MODELLING RULES

In evaluating the seismic capacity of the building, SAP2000 analysis program was used. In this analysis program, frame elements were defined between two joints as straight lines. Nodes at the foundation level were fixed, assuming that the foundation system is infinitely rigid. In order to describe the beam to column connections, rigid end zones were defined at the starting and end points of each frame elements. All beam and column dimensions were taken from the engineering drawings. Cross sectional properties of the frame members were defined using the gross cross sections. In calculating the stiffness of the beams, rectangular cross section was assumed.

Floor slabs were not included in the analysis, but they were assumed to have sufficient lateral stiffness to transmit the earthquake loads. That is, it was assumed that they behave as rigid diaphragms.

The analysis program considers dead weight of the frame members automatically. Weight of the floor slabs and the live loads on them, were calculated according to the TS 498 (Turkish Standard Institute, 1997) and distributed to the beams. In addition, weight of masonry walls was also considered as dead loads on beams.

In the analysis of the retrofitted system, the newly added shear walls were modeled as single columns at their center, having identical cross sectional properties with the shear wall. To represent the width of the walls, rigid end zones were placed at the floor levels as shown in Figure 3.8.

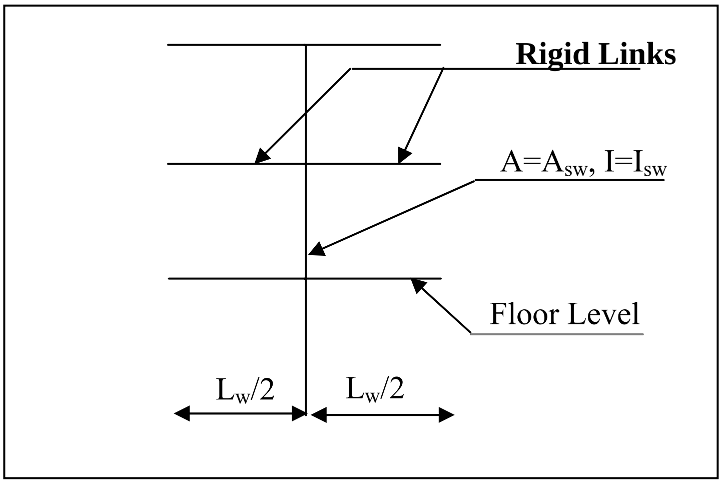


Figure 3.8 Shear Wall Model

3.6 ASSESSMENT PROCEDURES

3.6.1 Linear Elastic Procedure

Both of the existing and retrofitted structural systems were assessed according to the linear elastic procedure given in the Turkish Earthquake Code 2006. In the analysis, elastic earthquake loads are used. The total elastic base shear (V_t) to be applied to the structure was calculated in each direction according to Turkish Earthquake Code 2006.

In evaluating the structure according to the Turkish Earthquake Code 2006 the parameters used in the analysis can be listed as below:

- Coefficient of Effective Ground Acceleration A_0 = 0.30 (Earthquake Zone 2, taken from Table 2.4 of Turkish Earthquake Code 2006)
- Elastic acceleration spectrum corresponding to soil class Z2 (taken from Figure 2.5 of the Turkish Earthquake Code 2006)

- Since the case study building has six floors, λ in Eq. (2.1) is taken as 0.85 in evaluating the earthquake load.
- Building knowledge level is accepted as comprehensive knowledge level.

3.6.2 Nonlinear Procedure

In order to determine structural performance level more precisely, both existing and retrofitted systems were subjected to nonlinear analysis. In this assessment procedure, static pushover analysis was used. It should be noted that static pushover analysis can only be applied to elasto-plastic systems.

Nonlinear performance assessment of the case study building was investigated by using both the Turkish Earthquake Code 2006 and FEMA 273 guidelines. Plastic strain or rotation demands calculated at the target displacement level are compared with the limit values given in the associated guidelines.

3.6.3. Target Performance Level

As it was stated previously, the case study building is a telephone exchange building. These types of buildings are important after the earthquake. They must remain operational after a design earthquake. Therefore, during a design earthquake limited damage in the structural elements are permitted and the building must be usable by some minor repairs after such an earthquake. During a more severe earthquake, on the other hand, significant damage may occur but it is expected that overall risk of life-threatening injury as a result of structural damage is low.

According to the used guidelines, basically there are three objective performance levels. As it was explained briefly in Chapter II, in the Turkish Earthquake Code 2006 these objective performance levels are related with the occurrence probability of different earthquakes. In the code, the case study building is accepted as the building that must be operational after an earthquake. The buildings in this category must satisfy two objective performance levels for different earthquakes having different exceedance probability. For the earthquake having 10% exceedance probability in 50 years the building must satisfy immediate occupancy performance level. For the earthquake having 2% exceedance in 50 years on the other hand, the building must satisfy the life safety performance level.

3.7 ANALYSIS STAGES OF THE EXISTING AND RETROFITTED SYSTEMS

The structural system was analyzed by using SAP 2000 analysis program. The existing and retrofitted structural systems were assessed by both linear and nonlinear methods. The analysis stages can be classified as follows;

- Linear assessment proposed in the Turkish Earthquake Code 2006
- Nonlinear assessment according to the Turkish Earthquake Code 2006
- Nonlinear assessment described in FEMA 273

In linear assessment, equivalent elastic lateral load analysis method was employed. For nonlinear assessments, nonlinear displacements of the frame elements obtained from the pushover analysis were used. Linear and nonlinear assessment procedures used in this study were summarized in Chapter II. Moreover, a step by step procedure for an approximate pushover analysis is also presented in Chapter IV. For both linear and nonlinear assessment, the structural system is analyzed by using SAP 2000 analysis program. Model outputs of existing and retrofitted structures are given in Figure 3.9 through Figure 3.16.

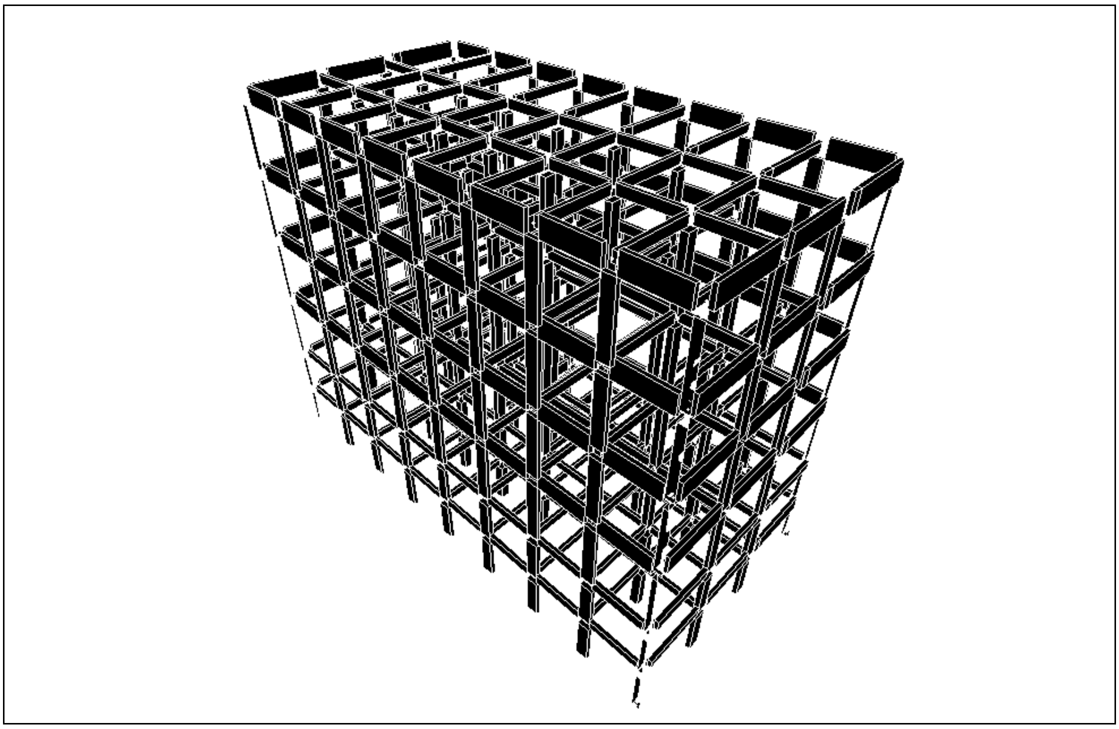


Figure 3.9 3-D View of the Existing System Model

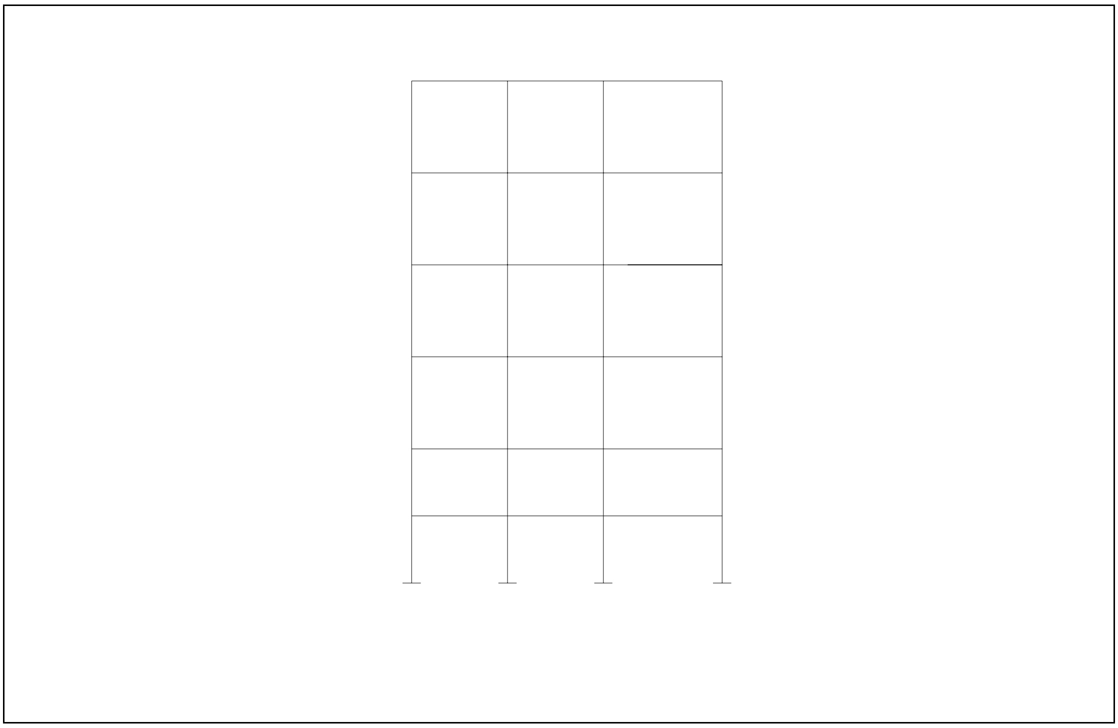


Figure 3.10 A Typical Frame in x – Direction of the Existing System Model

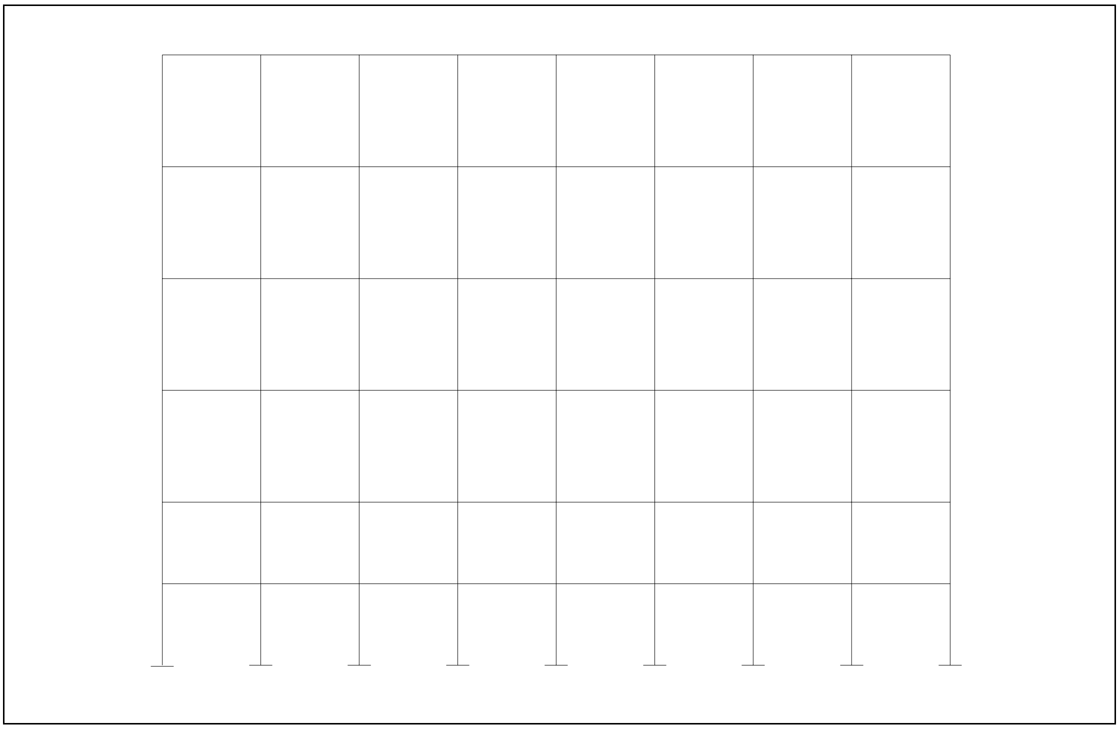


Figure 3.11 A Typical Frame in y – Direction of the Existing System Model

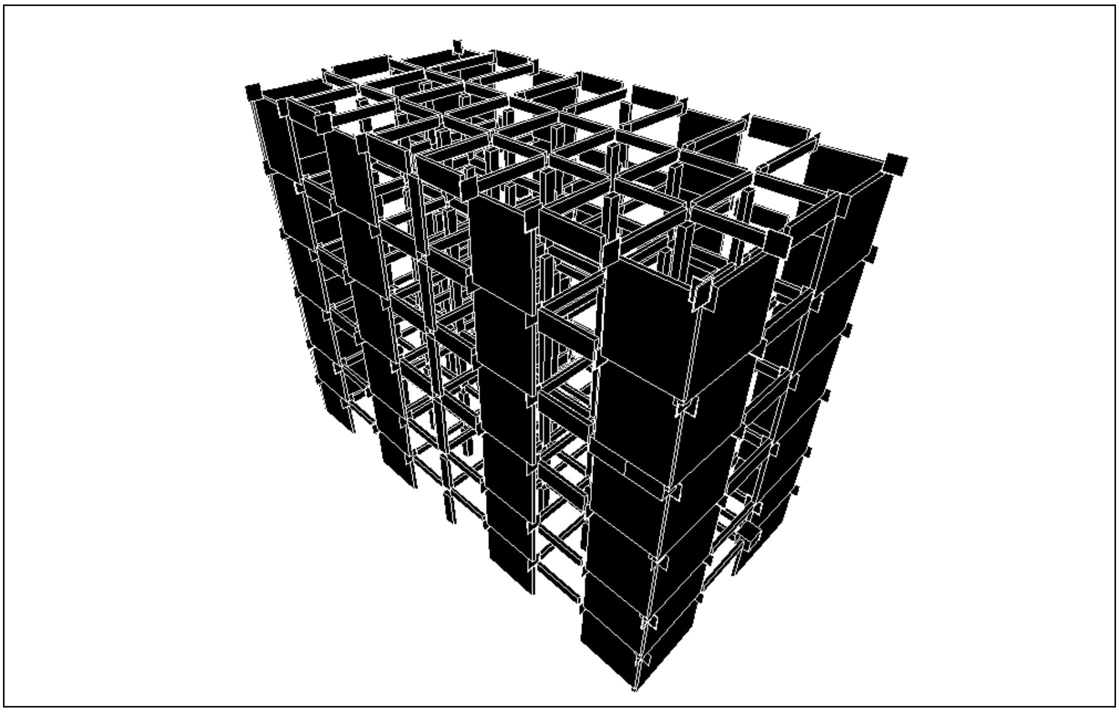


Figure 3.12 3-D View of the Retrofitted System Model

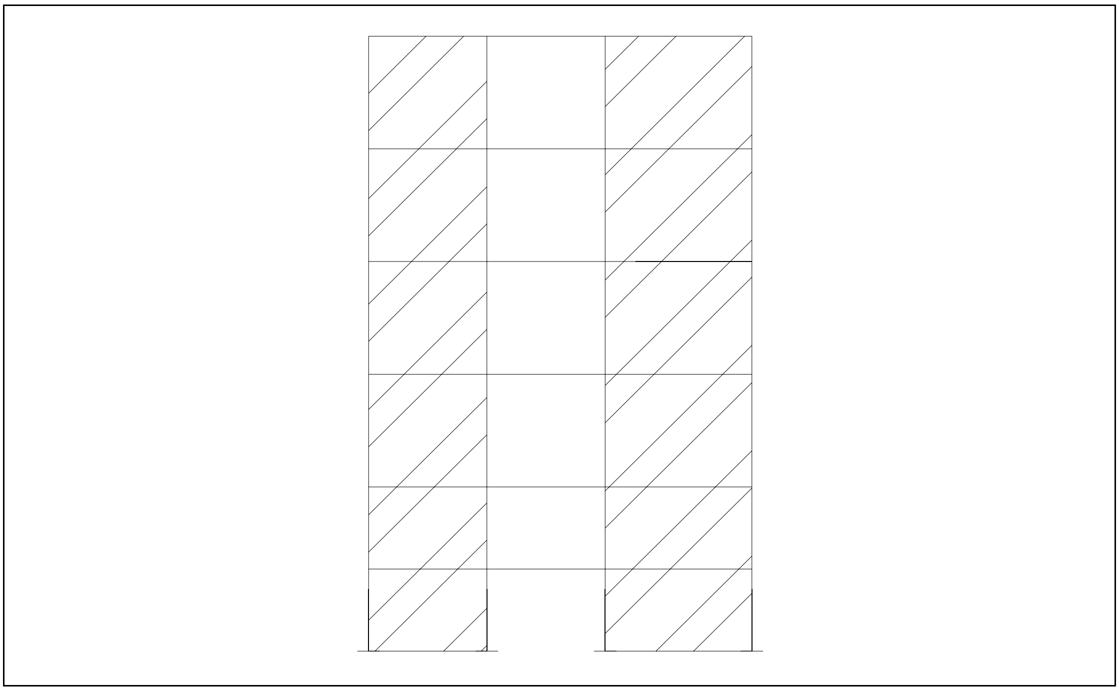


Figure 3.13 A Typical Exterior Frame in the x-Direction of the Retrofitted System Model

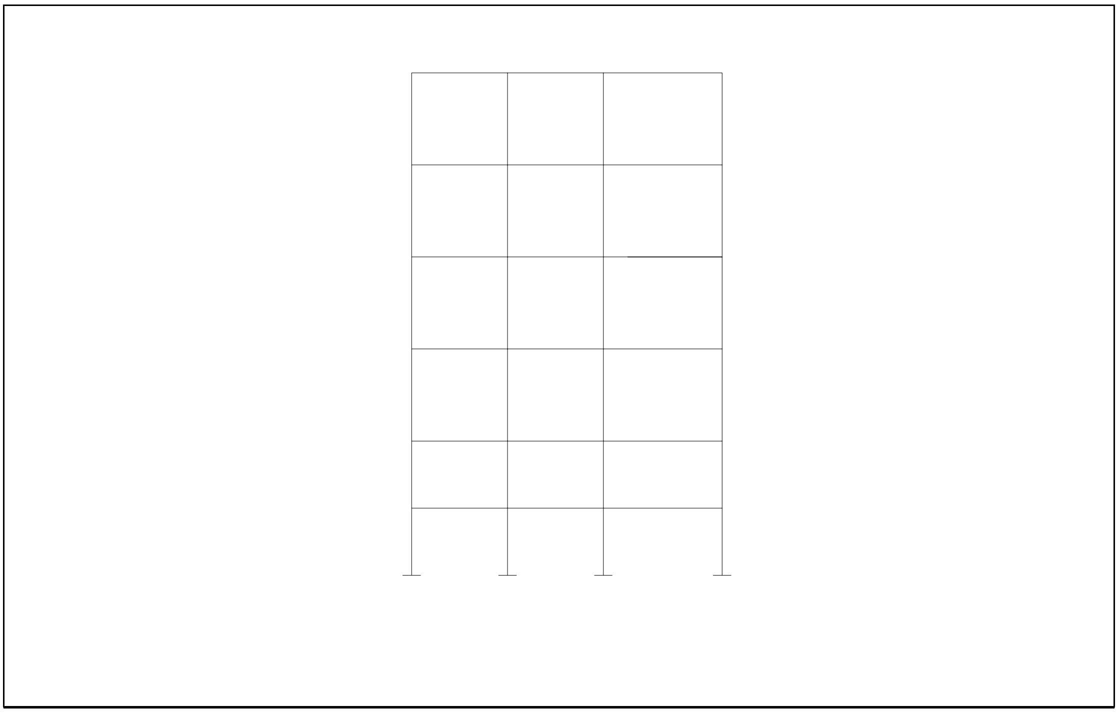


Figure 3.14 A Typical Interior Frame in the x-Direction of the Retrofitted System Model

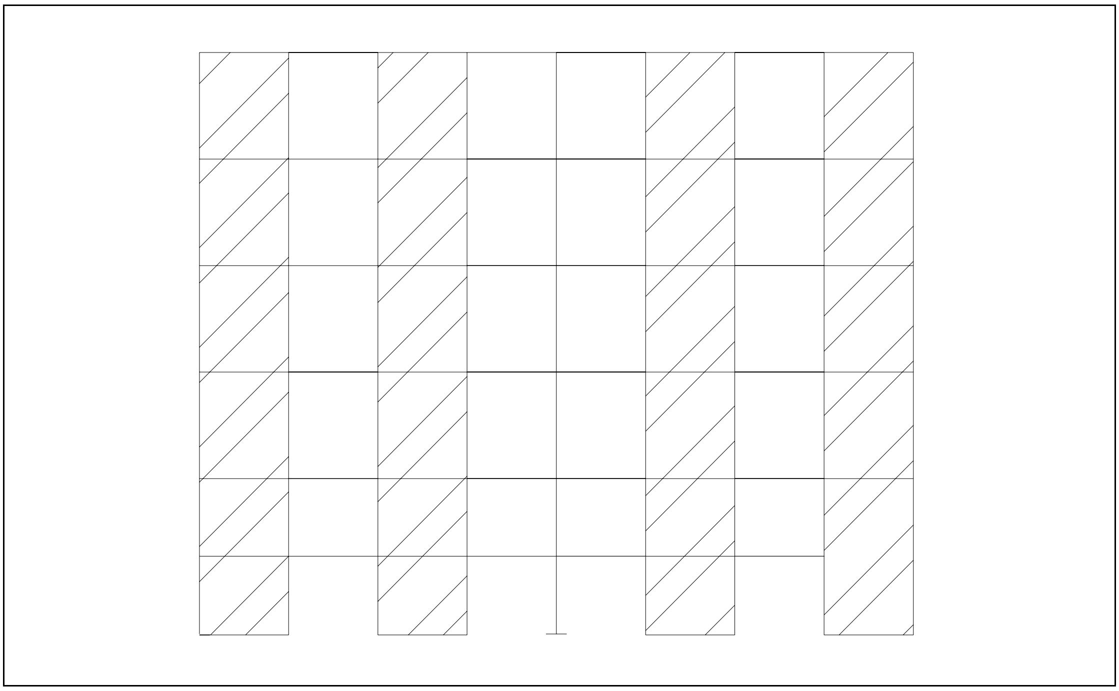


Figure 3.15 A Typical Exterior Frame in the y-Direction of the Retrofitted System Model

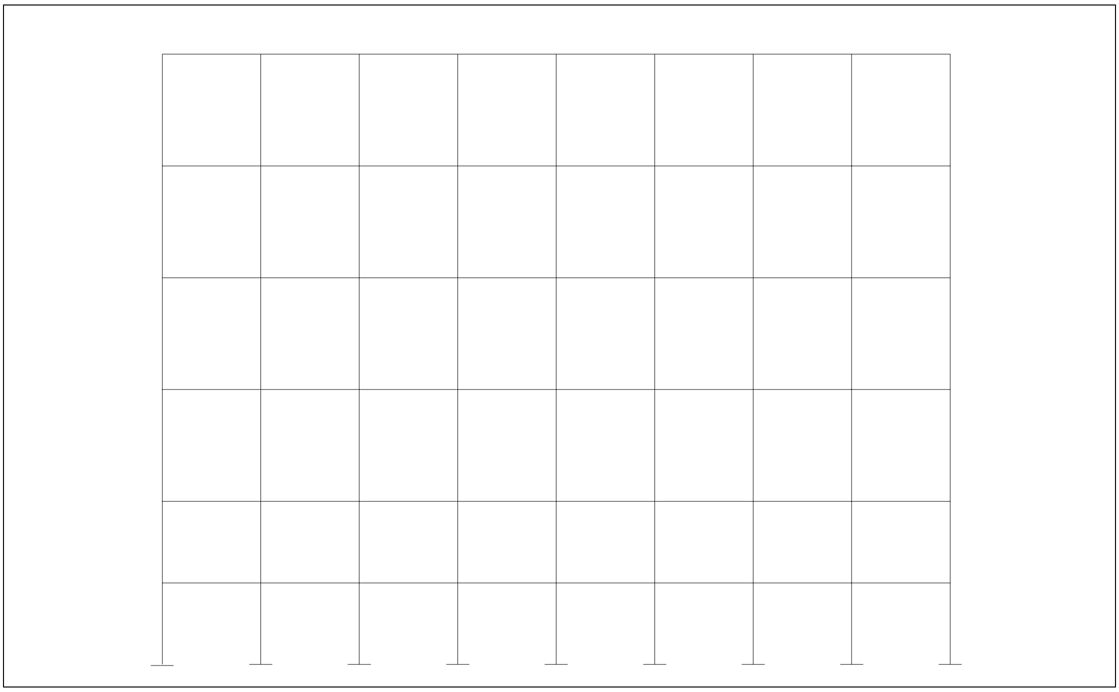


Figure 3.16 A Typical Interior Frame in the y-Direction of the Retrofitted System Model

3.8 FORCE – DISPLACEMENT RELATIONS FOR FRAME MEMBERS

Force-displacement relationships for the frame element cross sections were determined by RESPONSE 2000 (Bentz E. C., 2000) program. In both linear and nonlinear analysis, characteristic material properties were used to calculate capacities of cross sections.

As it was stated in Chapter II, in the Turkish Earthquake Code 2006, there are three knowledge levels and corresponding knowledge factors. For the case study building, all the engineering projects presenting all reinforced concrete details are available. Moreover, enough number of core samples was taken from the building. Therefore in the assessment of case study building the knowledge level was assumed as comprehensive and the corresponding knowledge factor of 1 was used.

In order to determine flexural capacities of columns, axial force versus moment interaction diagrams were calculated. For the nonlinear analysis, moment capacities were determined for axial load level calculated from gravity load analysis. That is, for nonlinear assessment moment capacity of each column was calculated from linear analysis applying only dead and live loads to the structural system. For the linear assessment on the other hand, firstly axial loads on columns were calculated from an analysis performed under the effect of gravity loads only. Then axial load under the effect of earthquake loads were determined from a limit analysis described in the Chapter 7A of the Turkish Earthquake Code 2006. In order to calculate axial load of each column for the linear assessment two axial loads which were calculated from the gravity load analysis and limit analysis were added. For the linear assessment, moment capacities of columns were determined at that axial load level.

In Appendix A, axial force-moment interaction diagrams for 3rd storey columns and newly added shear walls are presented.

For the beam capacities, moment curvature diagrams were calculated by using RESPONSE 2000 program. It is clear that, since the slabs were modeled as rigid diaphragms, axial load cannot be obtained on beams. Therefore, in deriving the moment curvature relation for beams, axial loads were neglected. For appropriate sections, T or L shaped sections were used in order to determine capacity of frame members more precisely.

In nonlinear assessment, inelastic displacement or deflection of a frame element determines the performance level of that element. In the Turkish Earthquake Code 2006, performance levels of the structural members are related to the strain capacity of concrete and reinforcement at the critical sections. In FEMA 273 on the other hand, performance criteria simply depends on plastic rotation angle obtained from analysis results at target displacement level. In nonlinear assessment according to the Turkish Earthquake Code 2006, firstly plastic rotations at the critical sections were determined as a result of the pushover analysis. Then, strain at top and bottom of a section were obtained from the RESPONSE 2000 program as a sub product of moment curvature diagram. Finally, strains at desired internal layers were calculated by linear interpolation from top and bottom strain values.

CHAPTER IV

AN APPROXIMATE ITERATIVE PUSHOVER ANALYSIS METHOD AND ITS VERIFICATION

4.1 GENERAL

In this study, efficiency of a widely used approximate pushover analysis was also verified on the case study building. The approximate pushover analysis procedure consists of a series of linear static earthquake analysis. This procedure is iterative, giving the sequence of hinging mechanisms and the displacement history of the building at different force levels of ground excitation. In this study, force controlled pushover analysis method was verified by using the SAP2000 linear elastic analysis program.

4.2 SIMPLIFIED PUSHOVER ANALYSIS PROCEDURE

The step-by-step procedure of the applied method can be summarized as follows;

1. Calculate the flexural capacities of the frame members by using existing beam and column dimensions and existing reinforcement configuration. Moment-curvature relation for beams and moment-axial force interaction

diagrams for columns should be evaluated for more accurate capacity values.

2. Construct a 3-D structural model and apply dead and live loads to the system.
3. Calculate total weight of the structure. Total weight of the structure is dead weight of the structure and a proportion of live load on the structure. In the case study building, the live load participation factor is 0.6.
4. Choose a load pattern to be applied among the load patterns described in Section 2.4.1 of this study. In analyzing the case study building, a load pattern proportional to the story heights h_i and weights w_i, is applied to the structure. The load pattern that is used in this study is shown in Figure 4.1.

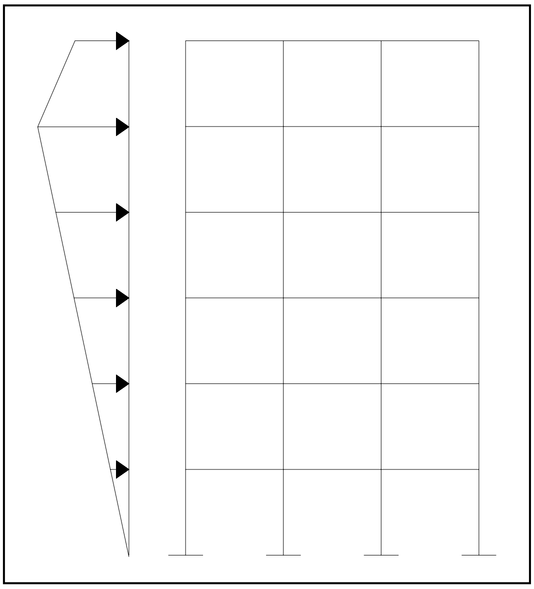


Figure 4.1 Load Pattern for Pushover Analysis

5. Apply lateral loads at floor levels in proportion to the selected load pattern.
6. Obtain the analysis results and compare flexural demands on members to member capacities for each frame elements. Demand to capacity ratios must be calculated for all critical sections.

7. Adjust the applied load so that only one member or a group of members reaches its flexural capacity (see Figure 4.2) and record the top story displacement at that load level.

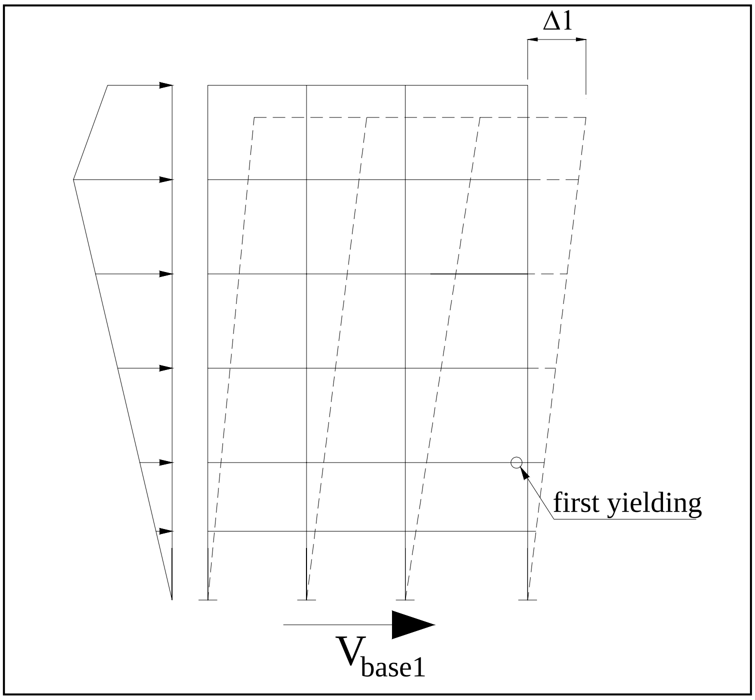


Figure 4.2 Adjust Total Base Shear for First Yielding

8. By placing hinges to the yielded member ends re-construct the model. In SAP2000 Analysis Program plastic hinges can be introduced by placing an internal hinge to the yielded end of the section and by applying an external moment equal to moment capacity of the member at that section. Also, in order to maintain the system in equilibrium, an external moment equal to the moment capacity of the section must be applied (see Figure 4.3). In this study, pushover curve is not constructed by cumulative load and displacement values. That is, in order to determine a point on pushover curve, total load is applied to the structure and total displacement corresponding to the applied base shear is obtained from the analysis results.

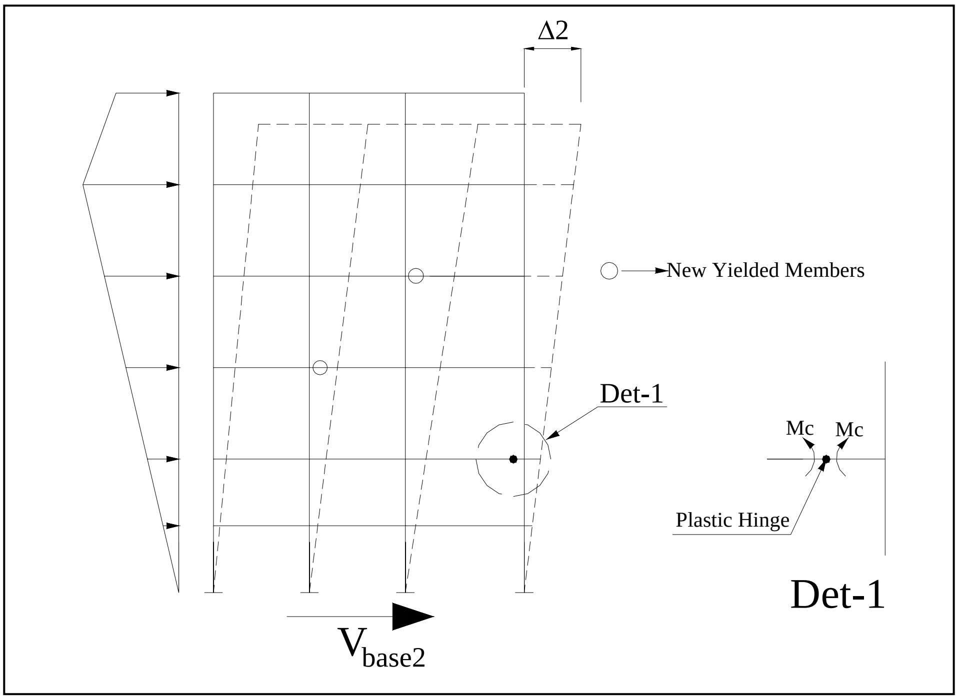


Figure 4.3 Introducing the Plastic Hinges

9. Incrementally increase the applied lateral load to yield some more elements. Record the total base shear applied and corresponding top story displacement at each step. At the end of each cycle, go to step 8 and modify the structural model. It is obvious that in order to get more realistic hinging mechanisms and top story drift values, applied lateral load must be increased with small increments as possible.
10. Continue this iterative analysis until the structure displaces infinitely under the effect of external forces and becomes unstable.

As it was stated previously, pushover curve is not constructed for cumulative values of base shear and displacement. Any point on the pushover curve is obtained directly. This method is called the secant method. At each step of this method, a larger amount of base shear from the previous step is applied in order to construct the pushover curve correctly. However, at further steps of the pushover curve, the structure may displace infinitely at a constant load level.

That is, even though the applied base shear remains constant, top floor may displace further at each step. As a result of this situation, after yielding of sufficient amount of elements, pushover curve becomes flat. In order to overcome this disadvantage, secant method was applied in force controlled region and incremental load method was applied in displacement controlled region. Construction of a pushover curve by employing above mentioned procedure is illustrated in Figure 4.4.

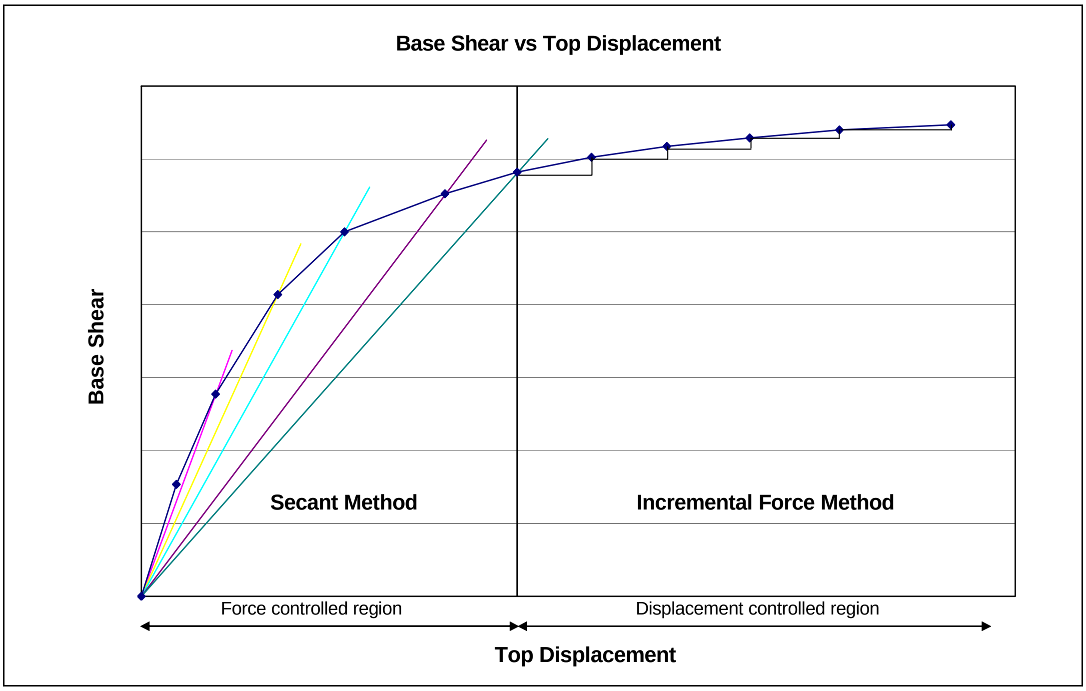


Figure 4.4 Constructing Capacity Curve

11. Plot the top displacement versus applied base shear force graph. This graph is called the capacity curve (see figure 4.5).
12. After constructing the capacity curve, determine the target displacement as explained in Chapter II.
13. Find the performance level of each element at the target displacement level.

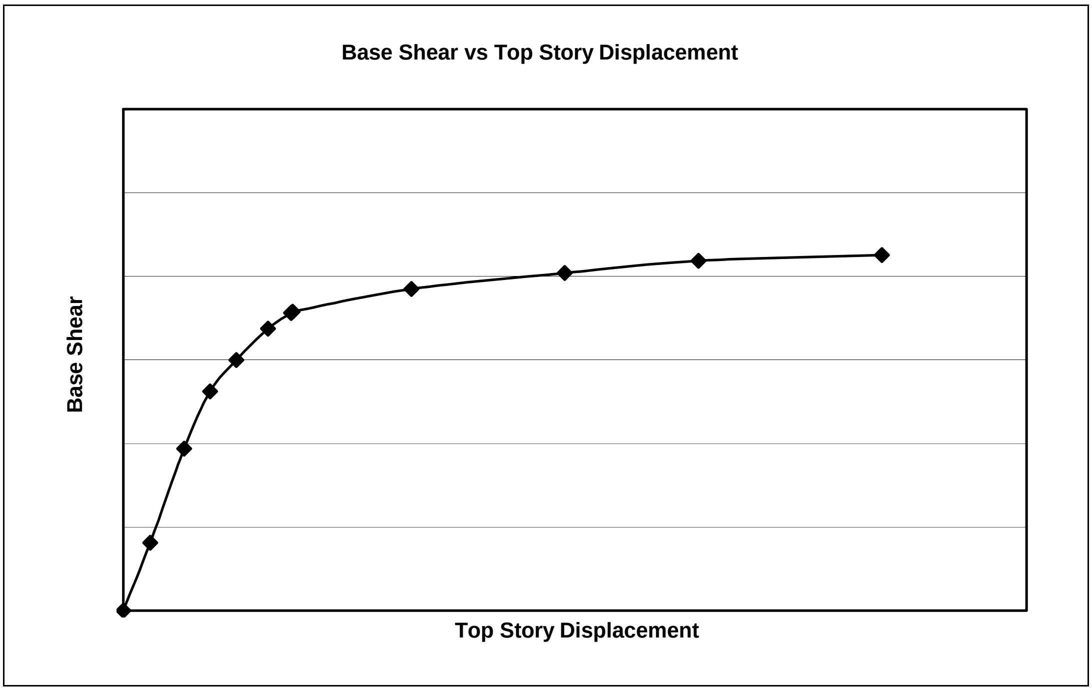


Figure 4.5 Capacity Curve

4.3 VERIFICATION OF THE SIMPLIFIED PUSHOVER ANALYSIS METHOD

In order to verify the simplified procedure, the analysis results were compared with the results obtained from SAP2000 nonlinear static (pushover) analysis.

The 3-dimensional model of the existing structure was subjected to pushover analysis by SAP2000 Analysis Program. In order to define the force displacement relation for the frame elements, moment-curvature relation for the beams and moment-axial force relation for the columns are used. By using characteristic material properties and reinforcement configuration, plastic hinge properties of each frame element are defined.

For columns, in order to precisely define hinge properties by accounting biaxial bending, five axial force-moment diagrams for five different bending angles were calculated by using formula given in the Code of Practice for Structural Use of Concrete (British Standard Institution, 1972). For 0^o, 22.5^o, 45^o, 67.5^o and 90^o angles, axial force-moment interaction diagrams were calculated and introduced to the analysis program. For 0^o, moment values about x axis are maximum, and moment values about y axis are all 0 for 90^o, moment values about y axis are maximum, and moment values about x axis are all 0, respectively. These two cases are pure bending cases about x or y directions. For the intermediate angles on the other hand, moment resistance about both axes exist. As the angle approaches from 0^o to 90^o, moment values about x axis decrease, and moment values about y axis increase.

For beams, moment-curvature relations are defined for all critical sections. In the simplified method, the effect of strain hardening is not considered. Therefore, in this more exact analysis, effect of strain hardening is also ignored.

Plastic hinge properties are introduced to the frame element ends. The structure is subjected to pushover analysis in both directions.

Pushover curves obtained from these analyses are compared with that of simplified method. In Figure 4.6 and Figure 4.7 pushover curves obtained from these analyses in both directions are presented.

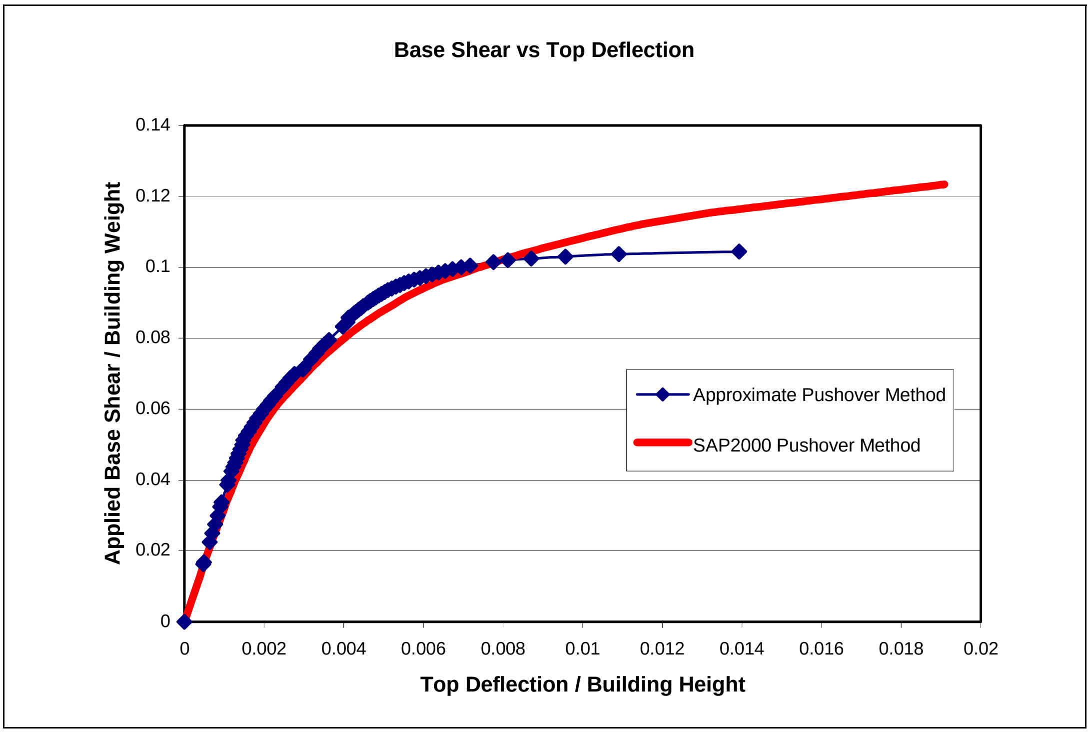


Figure 4.6 Existing System Verification of Pushover Curve in x - Direction

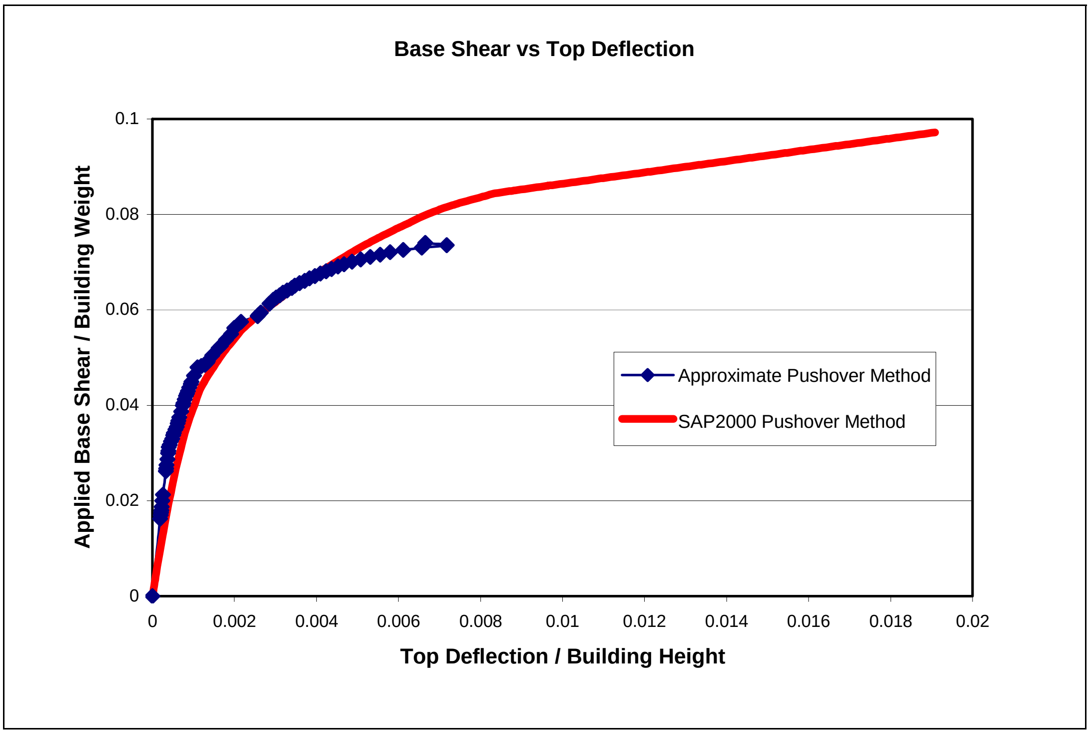


Figure 4.7 Existing System Verification of Pushover Curve in y - Direction

As it is seen from the given graphs, pushover curves obtained by using different analysis methods are close to each other. There are some discrepancies between the curves. In the simplified method, axial forces on the columns are assumed to remain constant during ground excitation and axial loads obtained from vertical load analysis were used. Moment capacities of the columns are calculated at that vertical load level, by considering pure bending about x or y axis. However, in the SAP2000 pushover analysis, column capacities are calculated at each step by considering the biaxial flexural effect on columns. In addition, in SAP 2000 method, applied loads are increased in very small increments at each step. This leads to more precise results.

From the given graphs it is clear that, the approximate method predicts, the displacement history of the building during a ground motion, in acceptable ranges. Therefore, this approximate method may be used for practical purposes, in order to determine the nonlinear static behavior of the structure.

CHAPTER V

COMPARATIVE ASSESSMENT OF THE EXISTING BUILDING

5.1. GENERAL

In order to determine the performance level of the case study building, linear and nonlinear analysis procedures were employed. As it was stated in the previous chapters, the Turkish Earthquake Code 2006 and FEMA 273 guideline were used in order to assess the case study building.

In linear analysis, equivalent static earthquake load procedure given in the Turkish Earthquake Code 2006 was used. Evaluation of the structural performance by linear procedure was performed only according to the Turkish Earthquake Code 2006.

For nonlinear analysis, static pushover analysis of SAP 2000 analysis program was used. Rotation angles of yielded elements were obtained as a result of pushover analysis, performed by using SAP 2000 analysis program.

As it was stated in the previous chapters, there are some differences for the nonlinear procedures, in the Turkish Earthquake Code 2006 and FEMA 273 in the evaluation of structural performance level. Therefore, nonlinear procedures were employed by using both the Turkish Earthquake Code 2006 and FEMA 273.

5.2. EIGENVALUE ANALYSIS

Modal shapes and corresponding periods of the structural system were obtained in both linear and nonlinear analysis methods. For the nonlinear analysis, the modal shapes and periods were evaluated at the initial linear elastic step of the nonlinear analysis.

As the current version of Turkish Code 2006 proposes, if there is no damage in the structural system, unreduced stiffness of the frame elements are used in linear analysis. As it was stated in the previous chapters, there was no damage observed in the structural system. Therefore, in linear analysis undamaged stiffness of frame elements was used.

In nonlinear analysis on the other hand, in order to represent the linear behavior of flexural elements before yielding, reduced stiffness values were used. In this study, for nonlinear assessment according to the Turkish Earthquake Code 2006, 40% of beam initial stiffness values were used. Columns or shear walls stiffness values were calculated according to axial load on columns under the effect of vertical loads. Columns stiffness-axial load relation according to the Turkish Earthquake Code 2006 can be defined by following equations:

For $N_D/A_cf_{cm} \leq 0.10$ 40% of column or shear wall initial stiffness   (5.1)
For $N_D/A_cf_{cm} \geq 0.40$ 80% of column or shear wall initial stiffness   (5.2)

In above equations;
N_D: Axial load calculated under the effect of vertical loads
A_c: Gross cross sectional area of column or shear wall
f_{cm}: Compressive strength of concrete

For nonlinear assessment according to FEMA 273, 70% of column initial stiffness and 50% of beam initial stiffness values were used.

Eigenvalue analysis results are summarized in Table 5.1. From the analysis in which cracked section properties were used, longer natural periods were obtained as expected. It is obvious that using cracked sectional properties, structure becomes more flexible and displacements are larger. Similar to other international earthquake codes, the Turkish Earthquake Code 2006 requires that, sum of the participated mass ratios in each principal direction should not be less than 90 percent. This criterion can be satisfied in the ninth mode of vibration.

In Figure 5.1 modal shapes for the first three modes are given. As it can also be followed from Table 5.1 first mode of vibration is translation in x direction, the second mode is torsion, and the third mode of vibration is translation in y direction.

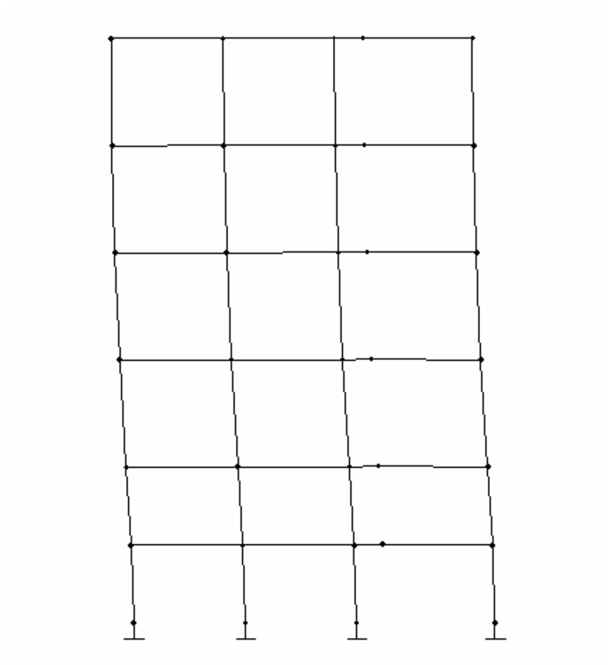


Figure 5.1 a) First Mode of Vibration

(T_{unred}=1.055 sec., T_{redTEC}=1.522 sec., $T_{redFEMA}$=1.353 sec.) (Elevation)

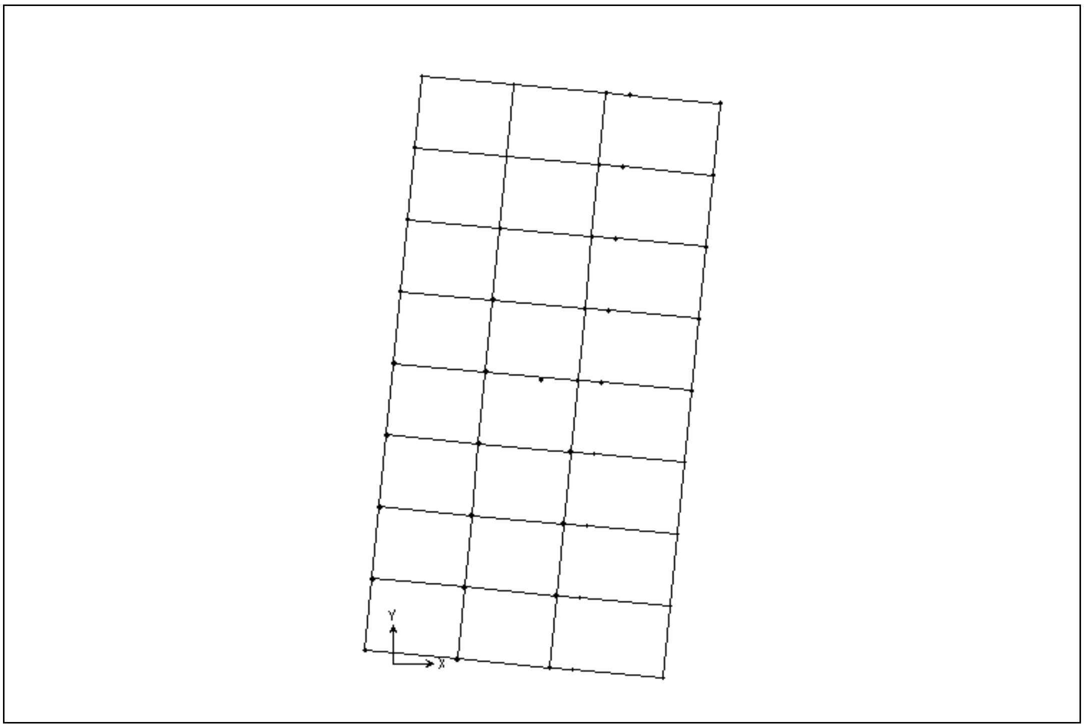


Figure 5.1 b) Second Mode of Vibration

(T_{unred}=1.027 sec., T_{redTEC}=1.466 sec., $T_{redFEMA}$=1.302 sec.) (Elevation)

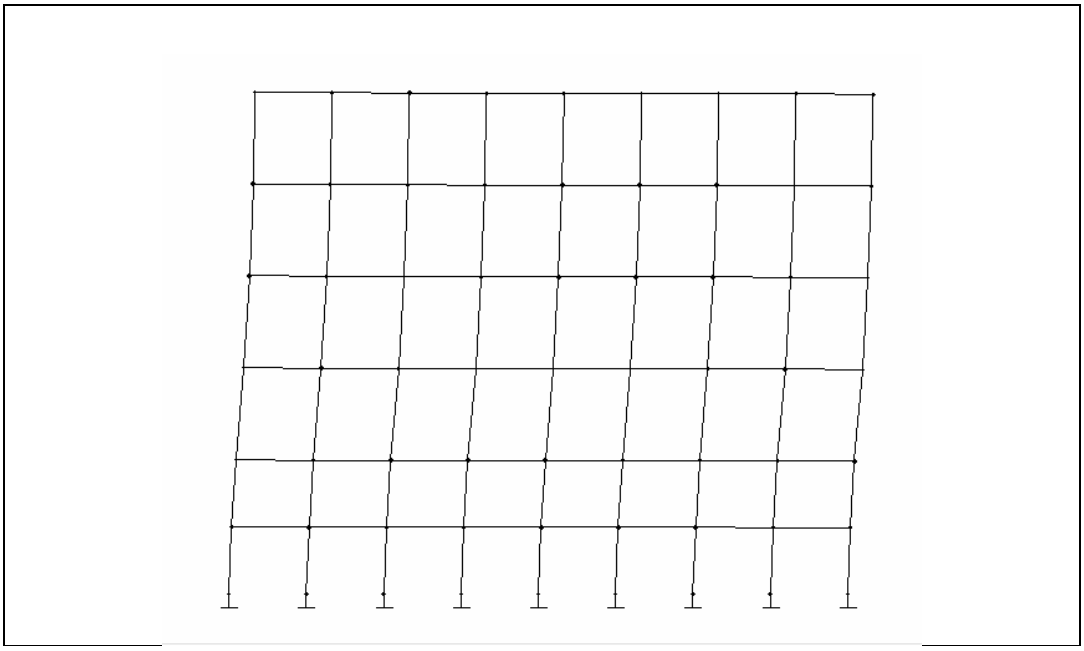


Figure 5.1 c) Third Mode of Vibration

(T_{unred}=0.861 sec., T_{redTEC}=1.230 sec., $T_{redFEMA}$=1.086 sec.) (Plan)

Figure 5.1 Existing System Vibration Modes

Table 5.1 Existing System Eigenvalue Analysis Results

Mode	Unreduced Stiffness			Reduced Stiffness According to the Turkish Earthquake Code 2006			Reduced Stiffness According to FEMA 273		
	T (sec)	Total Participated Mass Ratio in x – Direction	Total Participated Mass Ratio in y – Direction	T (sec)	Total Participated Mass Ratio in x – Direction	Total Participated Mass Ratio in y – Direction	T (sec)	Total Participated Mass Ratio in x – Direction	Total Participated Mass Ratio in y – Direction
1	1.055	74.2	0.0	1.522	72.7	0.0	1.353	74.6	0.0
2	1.027	74.4	0.0	1.466	72.9	0.0	1.302	74.7	0.0
3	0.861	74.4	77.0	1.230	72.9	76.0	1.086	74.7	77.8
4	0.349	87.5	77.0	0.511	85.6	76.0	0.440	87.3	77.8
5	0.333	87.6	77.0	0.488	85.7	76.0	0.413	87.3	77.8
6	0.284	87.6	88.7	0.419	85.7	87.2	0.352	87.3	88.8
7	0.213	94.4	88.7	0.304	92.5	87.2	0.264	94.1	88.8
8	0.207	94.5	88.7	0.294	92.6	87.2	0.254	94.2	88.8
9	0.178	94.5	94.9	0.255	92.6	93.1	0.218	94.2	94.7

5.3. ASSESSMENT OF EXISTING STRUCTURAL SYSTEM USING LINEAR ELASTIC PROCEDURE

5.3.1. Equivalent Static Lateral Load Analysis

For linear elastic procedure, equivalent static lateral load analysis proposed in the Turkish Earthquake Code 2006 was applied to determine structural performance of the case study building. General information on this method was given in the previous chapters. Analysis parameters used in the calculation of unreduced or elastic equivalent lateral load can be summarized as follows;

- Earthquake region II according to the Turkish Earthquake Code 2006,
- Soil class Z2 according to the code,
- The case study building is a six storey building; therefore λ coefficient given in Eq. (2.1) was taken as 0.85.

In Table 5.2, equivalent elastic static lateral load applied at each floor level are given for design spectra having different exceedance probabilities. Calculated equivalent lateral loads for design spectrum having 2% exceedance probability are 1.5 times that of design spectrum having 10% exceedance probability in 50 years. As it was stated in the previous chapters, the existing structural system is expected to satisfy immediate occupancy performance level for the earthquake spectrum having 10% exceedance probability and life safety performance level for the spectrum having 2% exceedance probability.

Table 5.2 Existing System Earthquake Loads at the Floors Level

			Earthquake loads for spectrum having 10% exceedance probability in 50 years		Earthquake loads for spectrum having 2% exceedance probability in 50 years	
Floor #	**H_i (m)**	**W_i (kN)**	**F_{ix} (kN)**	**F_{iy} (kN)**	**F_{ix} (kN)**	**F_{iy} (kN)**
1	3.50	6563	500.0	587.2	750.0	880.8
2	7.00	8476	1291.6	1516.7	1937.4	2275.1
3	11.80	6995	1796.8	2110.0	2695.2	3165.0
4	16.60	6995	2527.7	2968.4	3791.6	4452.6
5	21.40	6995	3258.5	3826.7	4887.8	5740.1
6	26.20	4277	2439.3	2864.6	3659.0	4296.9
V_{base} (kN)			11813.9	13873.6	17721.0	20810.5

5.3.2. Structural Irregularity and Drift Ratio Check

According to the Turkish Earthquake Code 2006, the equivalent static lateral load procedure can be used if the torsional irregularity coefficient calculated by applying the earthquake loads directly to gravity center of each floor, is less than 1.4. In Table 5.3 and 5.4 torsional irregularity coefficients calculated at each floor are summarized. These tables were prepared for the earthquake load calculated by using the earthquake spectrum having 10% exceedance probability in 50 years.

Table 5.3 Existing System Torsional Irregularity Check in x - Direction

Floor #	**δ_{imin} (m)**	**δ_{imax} (m)**	**δ_{iort} (m)**	**$\delta_{imax}/\delta_{iort}$**	**Check**
1	0.010	0.010	0.010	1.00	<1.4, ok
2	0.018	0.018	0.018	1.00	<1.4, ok
3	0.033	0.033	0.033	1.00	<1.4, ok
4	0.026	0.026	0.026	1.00	<1.4, ok
5	0.018	0.019	0.018	1.00	<1.4, ok
6	0.010	0.010	0.010	1.01	<1.4, ok

Table 5.4 Existing System Torsional Irregularity Check in y - Direction

Floor #	**δ_{imin} (m)**	**δ_{imax} (m)**	**δ_{iort} (m)**	**$\delta_{imax}/\delta_{iort}$**	**Check**
1	0.009	0.009	0.009	1.01	<1.4, ok
2	0.013	0.013	0.013	1.01	<1.4, ok
3	0.024	0.024	0.024	1.00	<1.4, ok
4	0.017	0.017	0.017	1.00	<1.4, ok
5	0.012	0.012	0.012	1.00	<1.4, ok
6	0.006	0.006	0.006	1.01	<1.4, ok

Since the existing system is very regular and symmetrical in both directions, torsional irregularity is negligible.

In Table 5.5 and 5.6, interstorey drift ratio for both earthquake loading cases are given. In the Turkish Earthquake Code 2006, limiting values for the interstorey drift ratio differ according to expected performance levels. Therefore, in Table 5.5 and 5.6, summary of the interstorey drift ratios for immediate occupancy performance level and life safety performance level are given separately.

Table 5.5 Existing System Interstorey Drift Ratio Check in x - Direction

		Immediate Occupancy			Life Safety		
Floor #	**hi (m)**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**
1	3.50	0.0101	0.00289	<0.008 ok	0.0151	0.00431	<0.02 ok
2	3.50	0.0181	0.00516	<0.008 ok	0.0270	0.00770	<0.02 ok
3	4.80	0.0334	0.00696	<0.008 ok	0.0499	0.01040	<0.02 ok
4	4.80	0.0262	0.00546	<0.008 ok	0.0391	0.00815	<0.02 ok
5	4.80	0.0185	0.00386	<0.008 ok	0.0276	0.00574	<0.02 ok
6	4.80	0.0099	0.00207	<0.008 ok	0.0147	0.00307	<0.02 ok

Table 5.6 Existing System Interstorey Drift Ratio Check in y - Direction

		Immediate Occupancy			Life Safety		
Floor #	**hi (m)**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**
1	3.50	0.0088	0.00250	<0.008 ok	0.0131	0.00375	<0.02 ok
2	3.50	0.0132	0.00378	<0.008 ok	0.0199	0.00568	<0.02 ok
3	4.80	0.0242	0.00504	<0.008 ok	0.0363	0.00756	<0.02 ok
4	4.80	0.0166	0.00346	<0.008 ok	0.0249	0.00519	<0.02 ok
5	4.80	0.0117	0.00245	<0.008 ok	0.0176	0.00367	<0.02 ok
6	4.80	0.0058	0.00120	<0.008 ok	0.0086	0.00180	<0.02 ok

As it can be observed from the above tables, although the existing structural system is very flexible, it satisfies the code requirements for the interstorey drift. Therefore, it can be said that overall rigidity of the existing structural system is in the acceptable ranges.

5.3.3. Performance Levels of the Frame Elements

In order to determine overall structural performance level, performance level of each individual element has to be determined. As the Turkish Earthquake Code 2006 proposes, first the elements are classified as ductile or brittle according to their failure mechanisms. Then the limiting demand to capacity ratio for each element is calculated. For beams, limiting demand to capacity ratio depends on confinement, shear force at the section and reinforcement ratio. For columns, limiting value of demand to capacity ratio depends on reinforcement ratio and axial load on them.

Demand to capacity ratios of columns and beams were calculated for two different performance levels. In order to determine the performance levels of each individual element, demand/capacity ratios were compared with the limiting values. In calculating the performance levels of elements, limiting values were determined assuming that the frame elements are unconfined, since they cannot satisfy the confinement rules given in the code.

As a result of code based assessment using linear elastic method, it is seen that, for the beams which were designed by using outdated standards or codes, bottom reinforcement amount at the beam supports are insufficient. In addition, beam yielding occurs before column yielding. It should be stated that, this type of structural behavior, strong column-weak beam, is expected from newly designed buildings in most of the modern earthquake codes.

In Tables 5.7 through 5.14, summary results of the code based linear elastic procedure are presented.

Table 5.7 Existing System Linear Procedure Immediate Occupancy Performance Level x – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	27	27	100	>10%
2	27	27	100	>10%
3	27	27	100	>10%
4	27	27	100	>10%
5	27	27	100	>10%
6	27	11	41	>10%

Table 5.8 Existing System Linear Procedure Immediate Occupancy Performance Level x – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	36	22	11805.4	9853.6	83.5	>0%
2	36	15	11305.9	5900.7	52.2	>0%
3	36	16	10015.0	4567.3	45.6	>0%
4	36	8	8219.9	3847.0	46.8	>0%
5	36	8	5693.7	2531.3	44.5	>0%
6	36	4	2437.6	791.3	32.5	>0%

Table 5.9 Existing System Linear Procedure Life Safety Performance Level x – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	27	27	100	>20%
2	27	27	100	>20%
3	27	27	100	>20%
4	27	27	100	>20%
5	27	27	100	>20%
6	27	10	37	>20%

Table 5.10 Existing System Linear Procedure Life Safety Performance Level x – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	36	13	17708.1	9931.2	56.1	>20%
2	36	13	16958.8	8205.1	48.4	>20%
3	36	21	15022.5	12587.4	83.8	>20%
4	36	10	12329.8	5764.1	46.7	>20%
5	36	6	8540.5	3210.1	37.6	>20%
6	36	4	3656.4	1187.8	32.5	>20%

Table 5.11 Existing System Linear Procedure Immediate Occupancy Performance Level y – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	32	32	100	>10%
2	32	32	100	>10%
3	32	32	100	>10%
4	32	32	100	>10%
5	32	16	50	>10%
6	32	2	6	<10%

Table 5.12 Existing System Linear Procedure Immediate Occupancy Performance Level y – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	36	28	13863.0	12146.5	87.6	>0%
2	36	24	13276.4	10726.1	81.1	>0%
3	36	25	11760.5	10644.1	90.5	>0%
4	36	16	9652.5	7272.0	75.3	>0%
5	36	16	6686.0	4995.1	74.7	>0%
6	36	10	2862.5	1605.9	56.1	>0%

Table 5.13 Existing System Linear Procedure Life Safety Performance Level y – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	32	32	100	>20%
2	32	32	100	>20%
3	32	32	100	>20%
4	32	32	100	>20%
5	32	16	50	>20%
6	32	1	3	<20%

Table 5.14 Existing System Linear Procedure Life Safety Performance Level y – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	36	23	20794.5	15383.8	74.0	>20%
2	36	16	19914.6	10516.2	52.8	>20%
3	36	18	17640.8	13870.8	78.6	>20%
4	36	16	14478.8	10913.8	75.4	>20%
5	36	14	10029.1	7172.3	71.5	>20%
6	36	0	4293.7	0.0	0.0	<20%

According to the Turkish Earthquake Code 2006, for immediate occupancy performance level, maximum 10% of beams may go beyond the minimum damage range, and all the columns must be within the minimum damage range. For life safety performance level on the other hand, 20% of beams may be beyond the life safety range, and shear force resisted by the columns exceeding the life safety range must not exceed 20% of total shear force on columns at that storey.

In both directions, almost all of the beams exceeded the acceptable range for both target performance levels. At all stories, some of the columns are beyond the target performance level. Shear forces at each floor, resisted by unsatisfying columns exceeded the limiting ratio.

According to linear assessment results, it can be concluded that, beams were designed only for vertical loads. Especially beam support section bottom reinforcements which are needed during cyclic loading were inadequate. Lack of confinement in frame sections results in smaller member capacities.

As a result of linear procedures, it is clear that the case study building cannot satisfy the subjected performance level criteria and it needs to be strengthened.

5.4. ASSESSMENT OF EXISTING STRUCTURAL SYSTEM USING NONLINEAR PROCEDURES

5.4.1. Pushover Analysis

In order to determine the structural performance level by nonlinear procedure, firstly static pushover analysis was employed in order to determine inelastic behavior of the structural system.

The case study building was assessed according to Turkish Earthquake Code 2006 and FEMA 273 guideline. As it was stated in previous parts, for nonlinear assessment according to Turkish Earthquake Code 2006 and FEMA 273, different initial stiffness values of frame elements were used. Therefore, pushover analysis was performed for the Turkish Earthquake Code 2006 and FEMA 273 separately. The pushover curves calculated by using different stiffness values are presented in Figure 5.2 and 5.3. Lateral load capacity of the structural system according to the Turkish Earthquake Code 2006 is less than that FEMA 273, as expected. Nonlinear behavior of the structure begins at the very early stages of the analysis. Overall lateral load carrying capacity is very limited. The structure loses its stability, at a very low base shear force level.

When the pushover curves obtained in both directions are compared, it is observed that lateral load capacity of the structure is higher in the x direction. This situation is mainly because of very low flexural capacities of beams in the y direction. In x direction the structure becomes inelastic at a load level of about 6-7% of total weight. In y direction on the other hand, inelastic behavior initiates at about 5-6% of total weight.

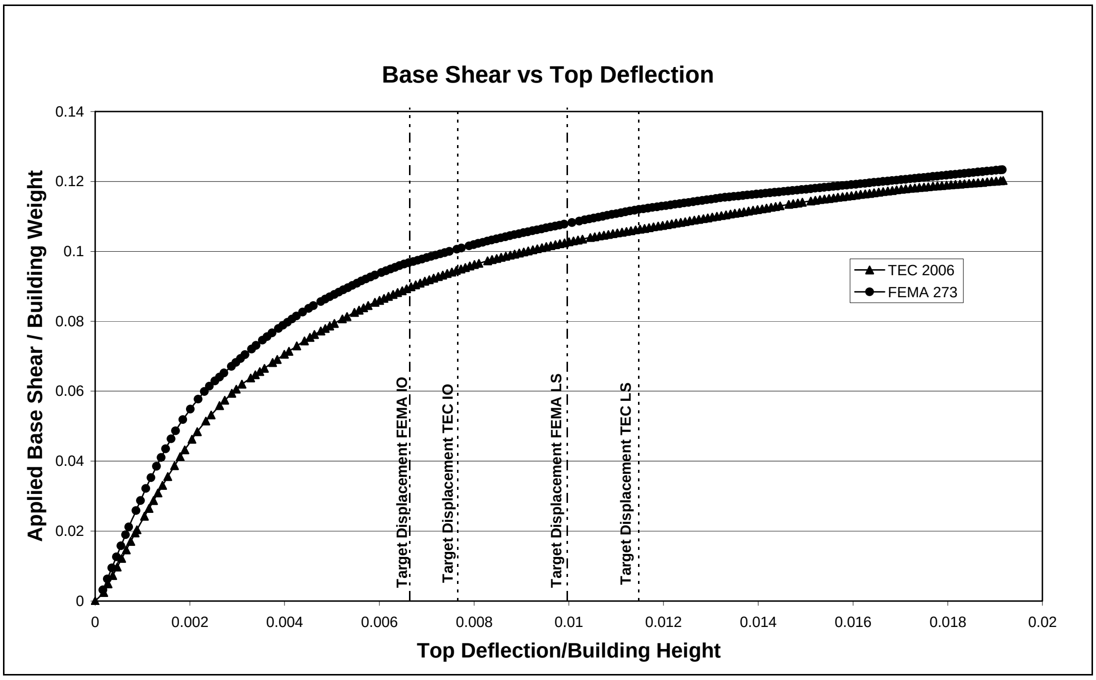


Figure 5.2 Existing System Pushover Analysis Result in x – Direction

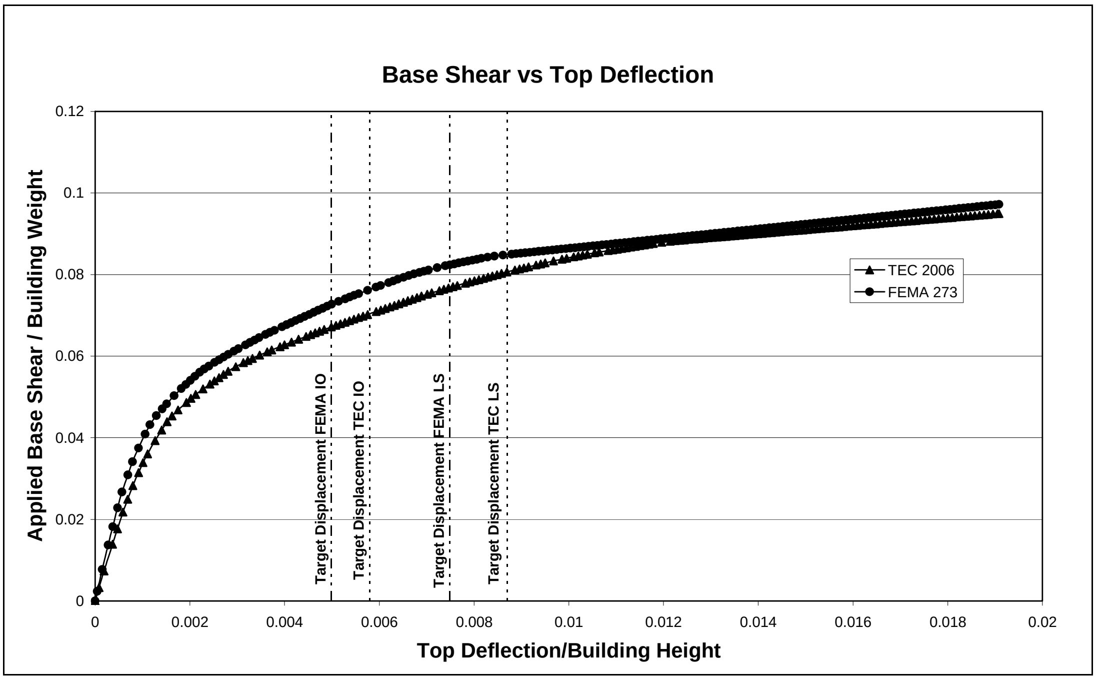


Figure 5.3 Existing System Pushover Analysis Result in y – Direction

Similar to linear procedure, the case study building was assessed for two target performance levels in the nonlinear assessment. Therefore, two target displacements were calculated using different earthquake spectra having different exceedance probabilities. Since the structure is more flexible in x direction, displacement demand of the structure in this direction is higher as compared to y direction. Inelastic deflection of the individual members were calculated at these target displacement levels and compared with the limiting values given in the Turkish Earthquake Code 2006 and FEMA 273.

Displacement demands of the structure in both directions were determined by using the methods described in Chapter II. Displacement demands or target displacements in each principal direction were calculated by using the Turkish Earthquake Code 2006 and FEMA 273. In Figure 5.2 and 5.3 target displacements for different performance levels calculated according to used codes or guidelines are also indicated. As it can be observed from these figures, target displacements calculated for the Turkish Earthquake Code 2006, are higher.

5.4.2. Performance Levels of the Frame Elements

Similar to linear procedure, firstly damage ranges of each frame elements were determined, in order to decide overall structural performance level.

From the nonlinear procedure results, rotation of each plastic hinge was obtained at target displacement. For the assessment according to FEMA 273 guideline, these plastic rotation angles were directly compared with the limiting value given for each damage range. For the assessment according to the Turkish Earthquake Code 2006 on the other hand, strain at the concrete extreme fiber and reinforcement was calculated. Damage range of each element was determined by comparing the calculated strain values to the limiting values.

Above mentioned procedure was repeated for two target performance levels. In Tables 5.15 through 5.22 summary results of the nonlinear procedure are presented.

As it can be observed from the below tables, the existing system of the case study building cannot satisfy the target performance levels according to nonlinear analysis results.

Table 5.15 Existing System Nonlinear Procedure Immediate Occupancy Performance Level x – Direction Beams, Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	27	0	0	<10%	8	29.6	>10%
2	27	27	100	>10%	27	100	>10%
3	27	27	100	>10%	27	100	>10%
4	27	27	100	>10%	27	100	>10%
5	27	0	0	<10%	0	0	<10%
6	27	0	0	<10%	0	0	<10%

Table 5.16 Existing System Nonlinear Procedure Immediate Occupancy Performance Level x – Direction Columns, Performance Level Summary

Floor #	Total Number of Columns	Total Shear Force on Columns (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	36	11805.4	0	0	0	≤0	0	0	0	≤0
2	36	11305.9	0	0	0	≤0	0	0	0	≤0
3	36	10015.0	0	0	0	≤0	0	0	0	≤0
4	36	8219.9	0	0	0	≤0	0	0	0	≤0
5	36	5693.7	0	0	0	≤0	0	0	0	≤0
6	36	2437.6	0	0	0	≤0	0	0	0	≤0

Table 5.17 Existing System Nonlinear Procedure Life Safety Performance Level x – Direction Beams, Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	27	0	0	≤20%	0	0	≤20%
2	27	0	0	≤20%	27	100	>20%
3	27	14	51.9	>20%	27	100	>20%
4	27	7	25.9	>20%	27	100	>20%
5	27	0	0	≤20%	0	0	≤20%
6	27	0	0	≤20%	0	0	≤20%

Table 5.18 Existing System Nonlinear Procedure Life Safety Performance Level x – Direction Columns, Performance Level Summary

Floor #	Total Number of Columns	Total Shear Force on Columns (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	36	17708.1	0	0	0	≤20%	6	4936	27.9	>20%
2	36	16958.8	0	0	0	≤20%	0	0	0	≤20%
3	36	15022.5	0	0	0	≤20%	0	0	0	≤20%
4	36	12329.8	0	0	0	≤20%	0	0	0	≤20%
5	36	8540.5	0	0	0	≤20%	2	139.4	1.6	≤20%
6	36	3656.4	0	0	0	≤20%	0	0	0	≤20%

Table 5.19 Existing System Nonlinear Procedure Immediate Occupancy Performance Level y – Direction Beams, Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	32	20	62.5	>10%	31	96.9	>10%
2	32	32	100	>10%	32	100	>10%
3	32	32	100	>10%	32	100	>10%
4	32	4	12.5	>10%	9	28.1	>10%
5	32	1	3.1	≤10%	0	0	≤10%
6	32	0	0	≤10%	0	0	≤10%

Table 5.20 Existing System Nonlinear Procedure Immediate Occupancy Performance Level y – Direction Columns, Performance Level Summary

Floor #	Total Number of Columns	Total Shear Force on Columns (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	36	13863.0	0	0	0	≤0	0	0	0	≤0
2	36	13276.4	0	0	0	≤0	0	0	0	≤0
3	36	11760.5	0	0	0	≤0	0	0	0	≤0
4	36	9652.5	0	0	0	≤0	0	0	0	≤0
5	36	6686.0	0	0	0	≤0	0	0	0	≤0
6	36	2862.5	0	0	0	≤0	0	0	0	≤0

Table 5.21 Existing System Nonlinear Procedure Life Safety Performance Level y – Direction Beams, Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	32	0	0	≤20%	0	0	≤20%
2	32	0	0	≤20%	32	100	>20%
3	32	0	0	≤20%	32	100	>20%
4	32	0	0	≤20%	3	9.4	≤20%
5	32	0	0	≤20%	0	0	≤20%
6	32	0	0	≤20%	0	0	≤20%

Table 5.22 Existing System Nonlinear Procedure Life Safety Performance Level y – Direction Columns, Performance Level Summary

Floor #	Total Number of Columns	Total Shear Force on Columns (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	36	20794.5	0	0	0	≤20	0	0	0	≤20
2	36	19914.6	0	0	0	≤20	0	0	0	≤20
3	36	17640.8	0	0	0	≤20	0	0	0	≤20
4	36	14478.8	0	0	0	≤20	0	0	0	≤20
5	36	10029.1	0	0	0	≤20	0	0	0	≤20
6	36	4293.7	0	0	0	≤20	0	0	0	≤20

In x direction, for immediate occupancy performance level, at 2nd, 3rd and 4th stories all of the beams yielded and plastic deformation of the yielded beams are beyond the acceptable ranges according to both applied nonlinear procedures. However, columns are within the acceptable damage range. For life safety performance level on the other hand, according to the code nonlinear assessment procedure, only beams at 3rd and 4th stories do not obey permitted ratios. But plastic deflections of yielded columns are within the acceptable ranges. According to FEMA 273 guideline on the other hand, all of the beams at 2nd, 3rd and 4th stories exceed the limiting damage range. In addition, at the 1st storey, shear force resisted by the yielded columns is more than 20% of total shear force applied at that storey.

In y direction, for immediate occupancy performance level, almost all of beams at 1st, 2nd and 3rd stories exceed the limiting damage range. In addition, beams at 4th storey do not satisfy the limiting conditions. For life safety performance level, plastic deflections of all beams are within the acceptable ranges according to the Turkish Earthquake Code 2006. For FEMA 273 on the other hand, all of beams at 2nd and 3rd stories exceed the limiting plastic deflections. Columns in y direction obey the plastic deflection criteria for both target performance levels.

As a result of nonlinear procedure, it is clear that existing case study building cannot satisfy the code or FEMA 273 requirements. In most cases columns satisfy the limiting conditions. However because of very low flexural capacities of beams, overall structural performance cannot obey the target performance levels criteria.

5.5. COMPARISON OF THE ASSESSMENT PROCEDURES FOR THE EXISTING STRUCTURAL SYSTEM

The existing structural system was assessed by using different codes and analysis methods. Firstly, the system was subjected to linear elastic analysis and assessed by using Turkish Earthquake Code 2006. Then, by using pushover analysis, inelastic deformation capacity of the structure was determined. At target displacement level, which is calculated by using inelastic structural properties, plastic deformation of each yielded members were compared with the acceptable limits given in FEMA 273 and Turkish Earthquake Code 2006. Assessment results of all these methods are given separately in the above parts of this chapter.

In Appendix B assessment results of existing structural system for life safety performance level are presented in graphical forms.

In addition to graphical representations given in Appendix B, comparative assessment results are presented on typical frames which are given in Figures 5.4 through 5.7. In these figures, element sections are marked according to their damage levels. Legends used in these figures are presented in Table 5.23.

Table 5.23 Legend for Frame Section Performance Levels

Used Mark	Corresponding Member Performance Level
	Performance Level<IO
○	IO< Performance Level <LS
●	LS< Performance Level <CP
X	Performance Level >CP

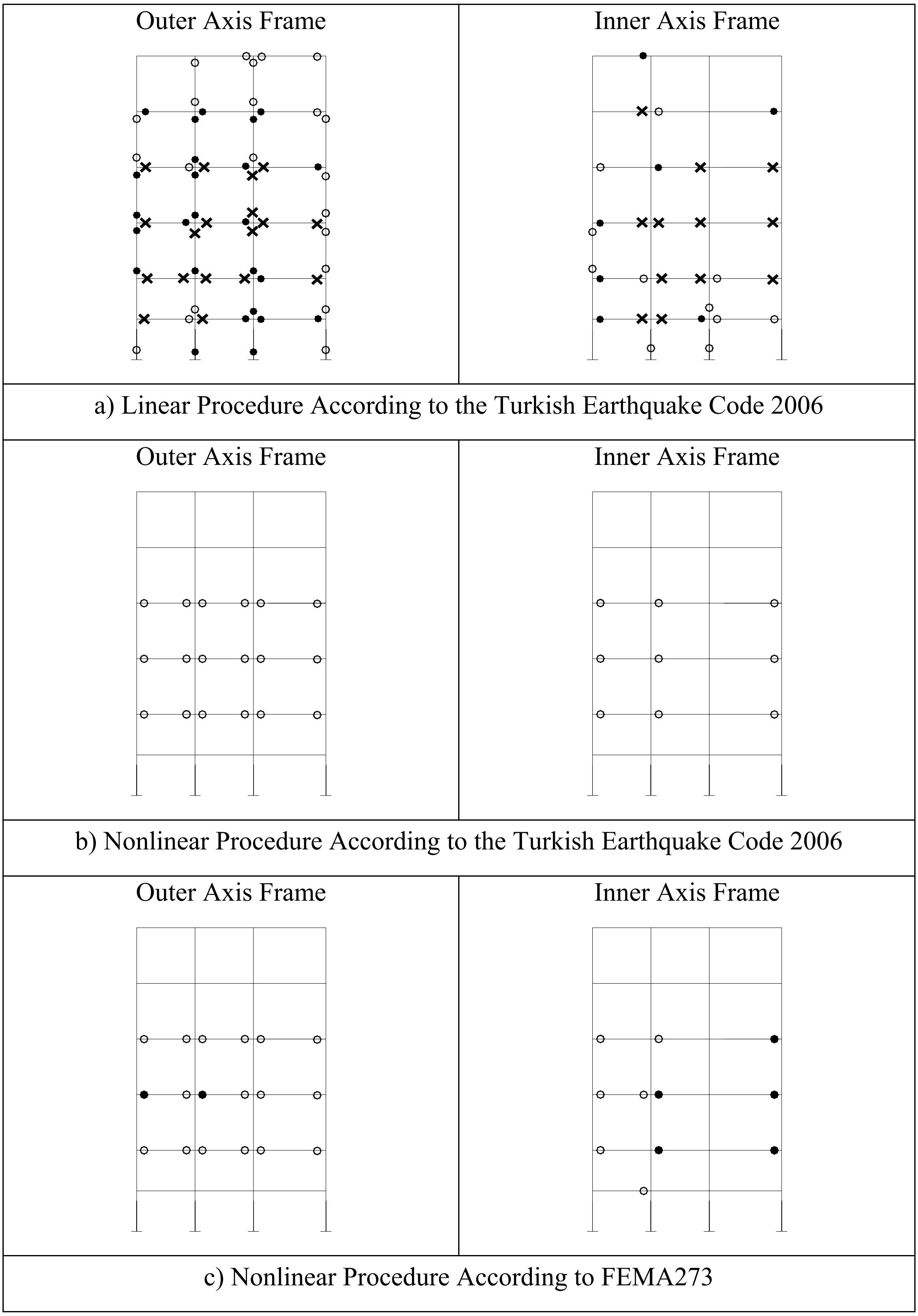


Figure 5.4 Existing System: Comparison of Element Sections Performance Levels, Immediate Occupancy Performance Level in x – Direction

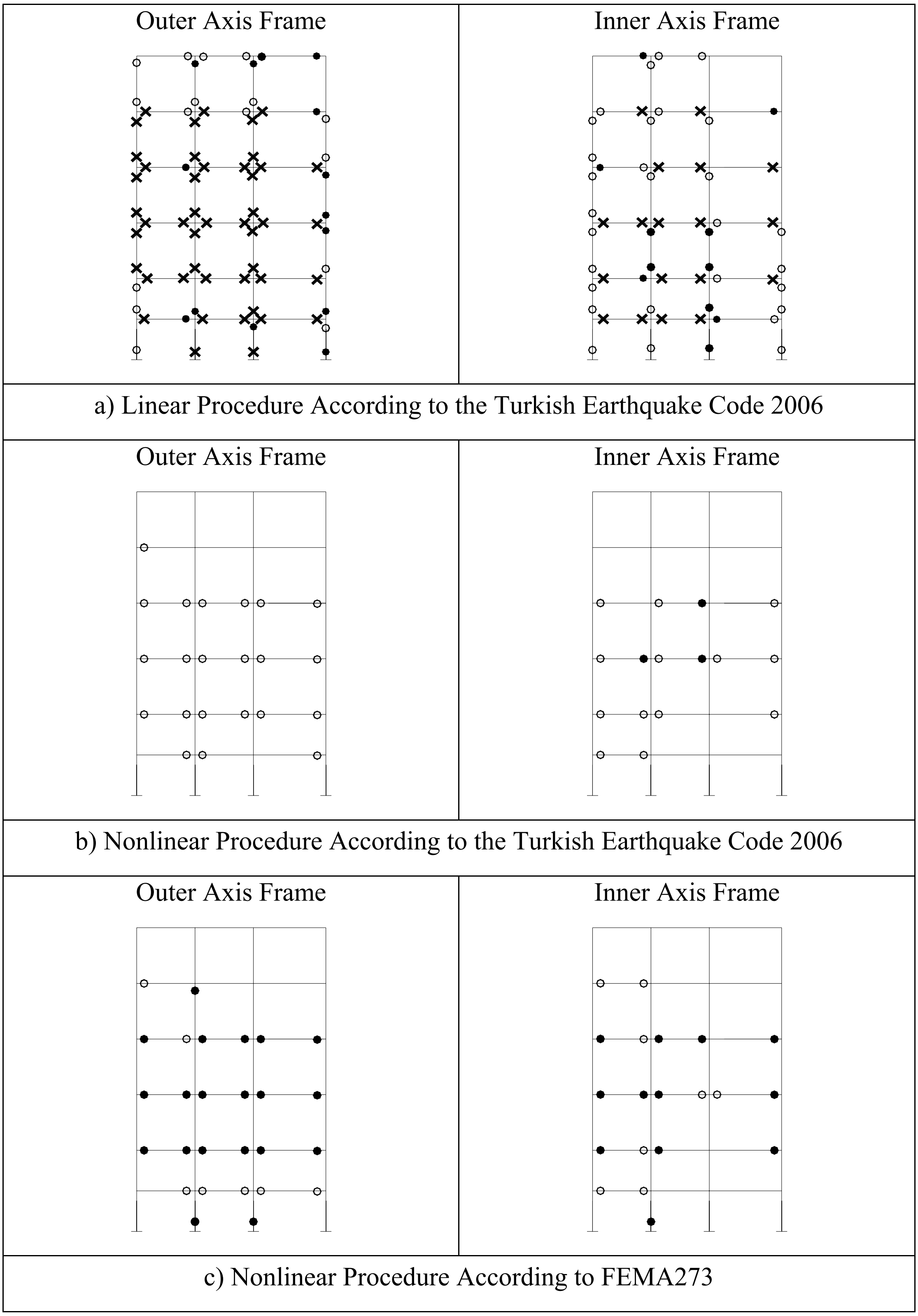


Figure 5.5 Existing System: Comparison of Element Sections Performance Levels, Life Safety Performance Level in x Direction

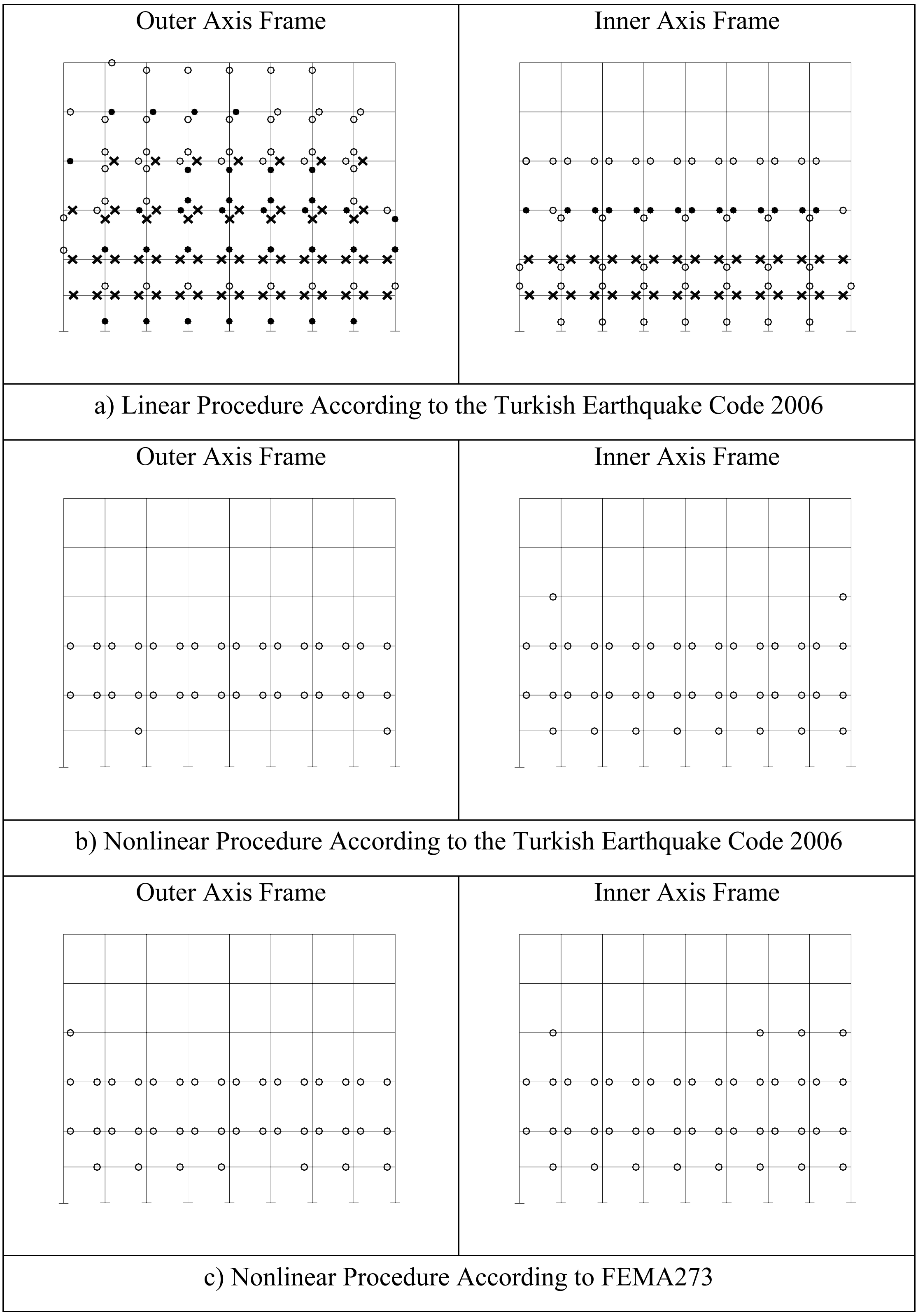


Figure 5.6 Existing System: Comparison of Element Sections Performance Levels, Immediate Occupancy Performance Level in y Direction

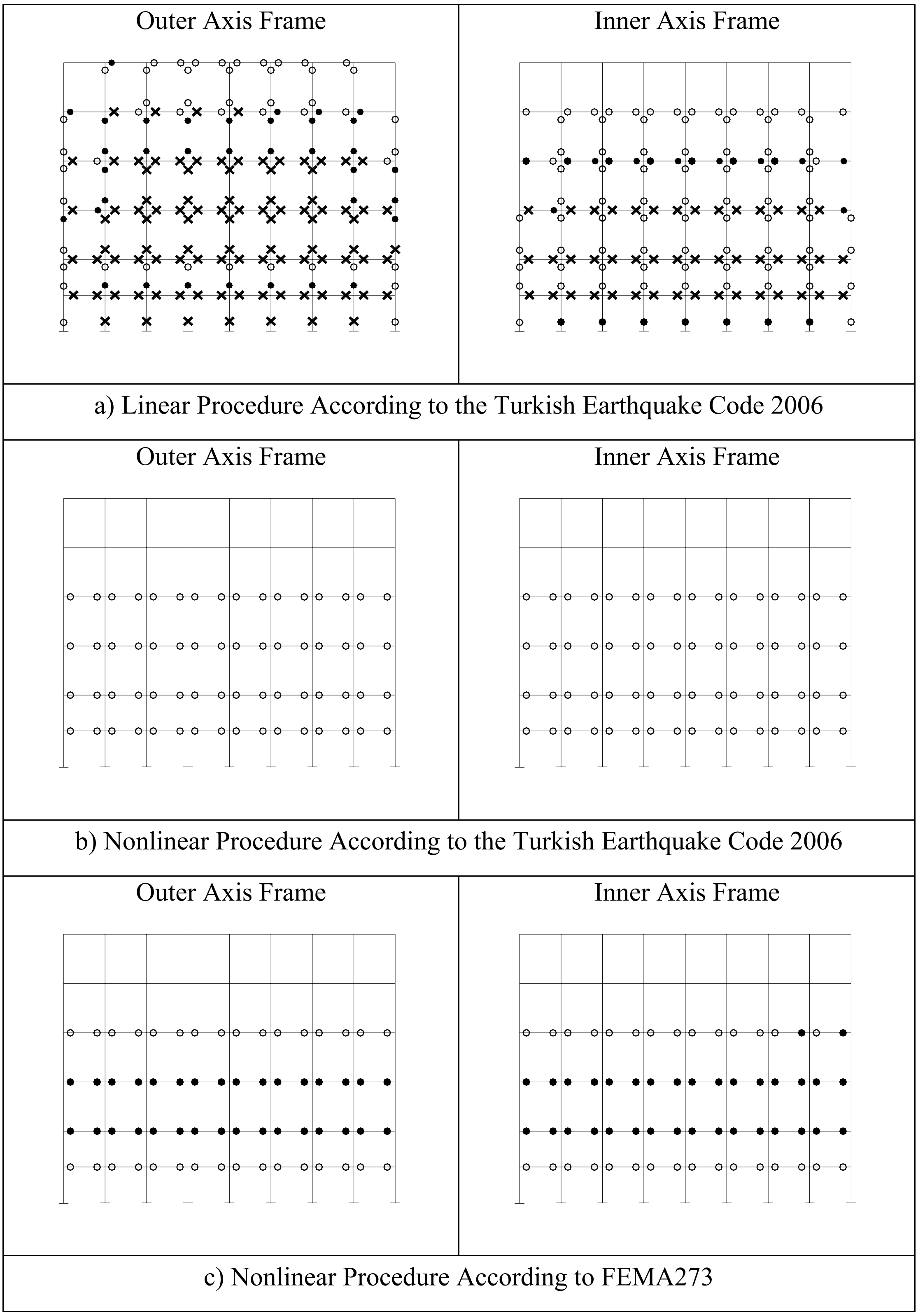


Figure 5.7 Existing System: Comparison of Element Sections Performance Levels, Life Safety Performance Level in y Direction

From the graphical representation given in Appendix B and the figures presented above, it is obvious that there exist some differences between the linear and nonlinear assessment results.

The main reason for the difference is based on the fact that, applied elastic earthquake load is too large as compared with lateral load capacity of overall structure calculated by using approximate pushover analysis. For immediate occupancy performance level, total elastic lateral forces applied in principal directions are 40% and 50% of total weight of the structure in x and y directions respectively. For life safety performance level, 150% more loads than those applied in immediate occupancy performance level are applied to the structure. As can be seen from the capacity curves obtained as a result of pushover analysis, in the x direction, the existing system reaches its displacement demand for immediate occupancy performance level at the load level of about 11% of total weight of the structure. In the y direction on the other hand, the target displacement is reached at the load level of about 9% of total weight. As compared to equivalent elastic earthquake loads applied to the structure for linear assessment method, these loads are very low. Furthermore, in linear assessment procedure it is not possible to observe the effect of redistribution. However, in the nonlinear assessment procedure, because of redistribution effect considered implicitly in the analysis, inelastic deformations of the yielded elements are limited.

There are some minor differences between nonlinear assessment results of the Turkish Earthquake Code 2006 and FEMA 273. Generally, FEMA 273 gives more conservative results as compared to the Turkish Earthquake Code 2006.

As a result of both linear and nonlinear assessment performed by using different codes or guidelines, it is clear that the existing system of the case study building is incapable of satisfying the performance level criteria and it needs to be strengthened.

CHAPTER VI

COMPARATIVE ASSESSMENT OF THE RETROFITTED BUILDING

6.1 GENERAL

In order to retrofit the existing system, shear walls are added to existing moment resisting frames. This method is basic and widely used to improve the overall structural behavior under the effect of lateral loads. Main strategy of this method is that, newly added shear walls are placed and designed so as to resist almost all of lateral loads acting on the structure. Accordingly, the existing frame system is mainly responsible for the gravity loads only.

In the case study building, shear walls were placed in peripheral frames. Effectiveness of a shear wall at peripheral axes is more than that of close to center. Symmetrical placement of shear walls is also important in order to avoid additional eccentricity. Area of shear walls and their ratio to floor area is given in Table 3.2. As it can be observed from this table, area of shear walls is approximately 1% and 1.5% of floor area in x and y directions respectively. Retrofitted scheme of the case study building was also presented in Chapter III.

The retrofitted system was assessed by following the same procedure with the assessment of the existing system. Firstly the structural system was subjected to eigenvalue analysis in order to determine the modal shapes and corresponding periods. Using the equivalent elastic lateral load analysis results, the structural system was assessed according to the linear procedure described in the Turkish Earthquake Code 2006. Then, the retrofitted system was subjected to pushover analysis in order to determine the inelastic capacity. According to the results of pushover analysis, the retrofitted structural system was assessed by using nonlinear procedure given in the Turkish Earthquake Code 2006 and FEMA 273 guideline. Finally, linear and nonlinear assessment results were compared in order to make a reasonable judgment about the effectiveness of retrofitting.

6.2 EIGENVALUE ANALYSIS

As it was stated in the previous chapter, modal shapes and periods were determined by using three different stiffness values of the frame members. In the linear assessment method, unreduced stiffness of the frame sections was used throughout the entire analysis. In the assessment using nonlinear analysis on the other hand, two different reduced stiffness values of reinforced concrete sections were used according to the Turkish Earthquake Code 2006 and FEMA 273 as described in Chapter V.

Eigenvalue analysis results are given in Table 6.1. Modal shapes for the first three modes are presented in Figure 6.1. As it can be followed from Table 6.1 and Figure 6.1, the first mode of vibration is translation in x direction. The second mode is translation in y direction and the third mode is torsion or rotation about z axis. In the analysis of the existing system, the second mode was torsion and the third mode was translation in y direction. Because of the implemented strengthening, periods became shorter. In other words, applied strengthening method has increased the overall structural stiffness considerably.

Table 6.1 Retrofitted System Eigenvalue Analysis Results

Mode	Unreduced Stiffness			Reduced Stiffness According to the Turkish Earthquake Code 2006			Reduced Stiffness According to FEMA 273		
	T (sec)	Total Participated Mass Ratio in x – Direction	Total Participated Mass Ratio in y – Direction	T (sec)	Total Participated Mass Ratio in x – Direction	Total Participated Mass Ratio in y – Direction	T (sec)	Total Participated Mass Ratio in x – Direction	Total Participated Mass Ratio in y – Direction
1	0.426	68.9	0.00	0.592	68.2	0.00	0.529	68.7	0.00
2	0.394	68.9	72.7	0.563	68.2	72.4	0.499	68.7	73.0
3	0.308	68.9	72.7	0.423	68.2	72.4	0.376	68.7	73.0
4	0.117	88.4	72.7	0.160	68.2	86.9	0.143	87.2	73.1
5	0.113	88.4	87.9	0.158	86.6	86.9	0.142	87.2	87.3
6	0.088	88.4	87.9	0.117	86.6	86.9	0.106	87.2	87.3
7	0.062	95.6	87.9	0.081	86.6	93.9	0.074	87.2	94.2
8	0.060	95.6	94.9	0.079	94.4	93.9	0.073	94.8	94.2
9	0.047	95.6	94.9	0.060	94.4	93.9	0.056	94.8	94.2

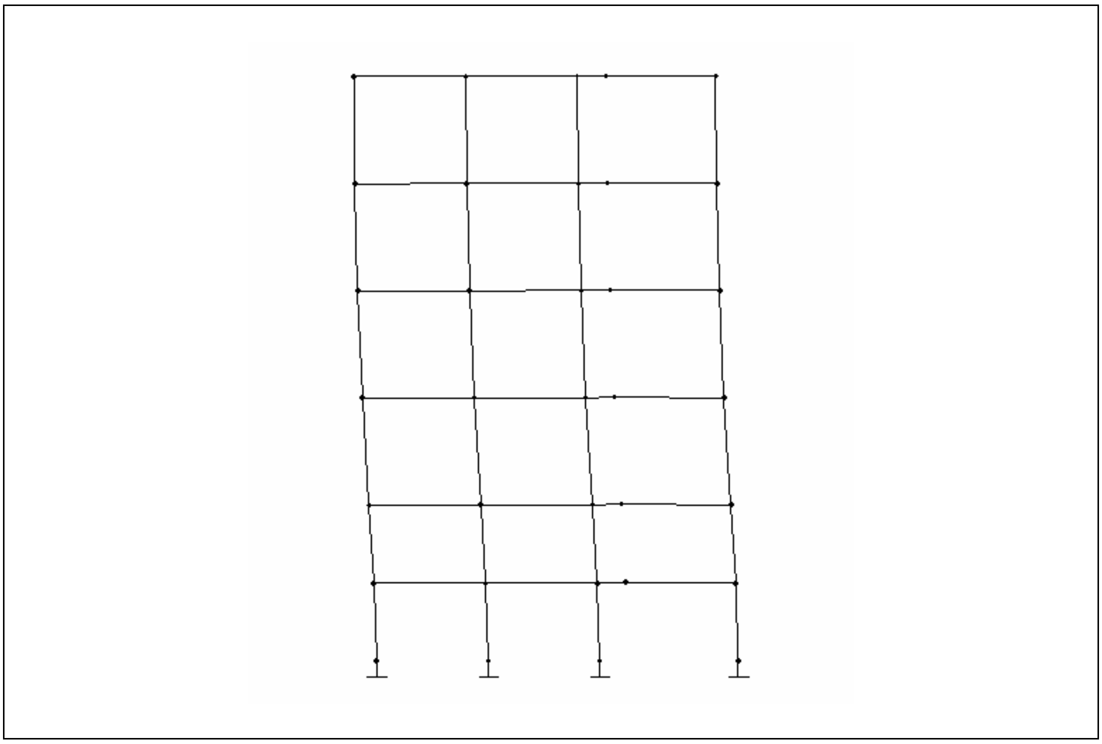


Figure 6.1 a) First Mode of Vibration

(T_{unred}=0.426 sec., T_{redTEC}=0.592 sec., $T_{redFEMA}$=0.529 sec.) (Elevation)

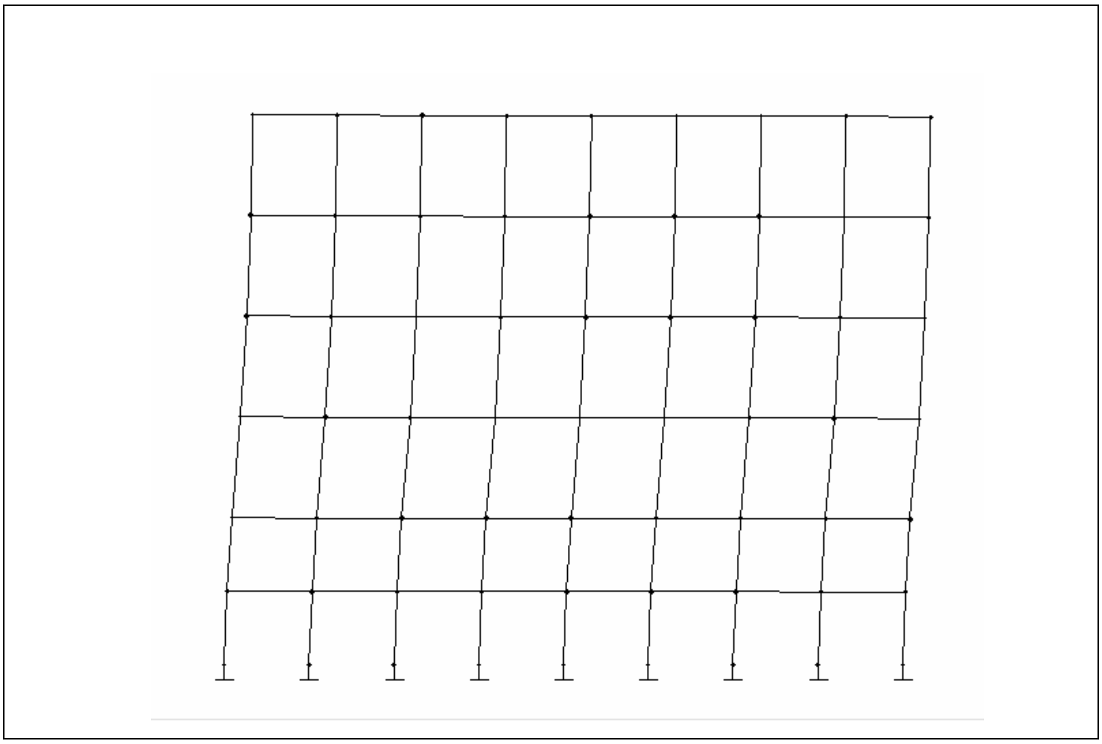


Figure 6.1 b) Second Mode of Vibration

(T_{unred}=0.394 sec., T_{redTEC}=0.563 sec., $T_{redFEMA}$=0.499 sec.) (Elevation)

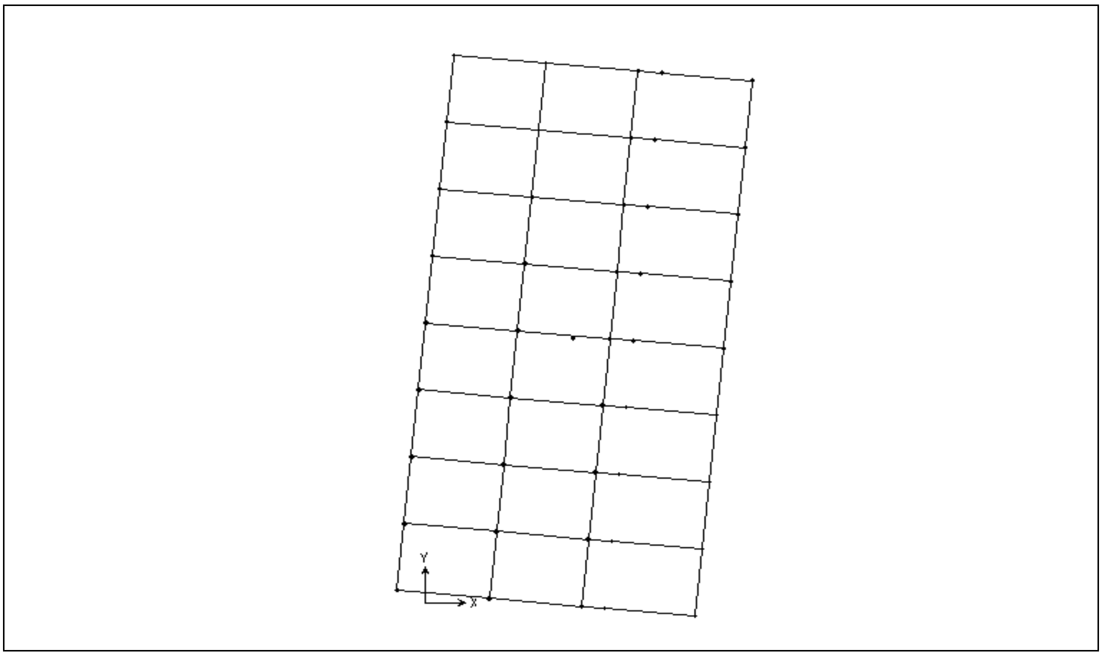


Figure 6.1 c) Third Mode of Vibration

(T_{unred}=0.308 sec., T_{redTEC}=0.423 sec., $T_{redFEMA}$=0.376 sec.) (Plan)

Figure 6.1 Retrofitted System Vibration Modes

6.3 ASSESSMENT OF RETROFITTED STRUCTURAL SYSTEM USING LINEAR ELASTIC PROCEDURE

6.3.1 Equivalent Static Lateral Load Analysis

Similar to assessment of existing system using linear elastic methods, in assessment of retrofitted structure equivalent elastic lateral load analysis method was employed. Using the analysis results, performance level of the retrofitted structure was determined by employing the linear assessment procedure proposed in the Turkish Earthquake Code 2006.

In Table 6.2, equivalent elastic static lateral load applied at each floor level are given for design spectra having different exceedance probabilities. Expected or target performance levels of the retrofitted structure are the same as that of existing structure.

Table 6.2 Retrofitted System Earthquake Loads at the Floors Level

			Earthquake Loads for Spectrum having 10% Exceedance Probability in 50 Years		**Earthquake Loads for Spectrum Having 2% Exceedance Probability in 50 Years**	
Floor #	**H_i (m)**	**W_i (kN)**	**F_{ix} (kN)**	**F_{iy} (kN)**	**F_{ix} (kN)**	**F_{iy} (kN)**
1	3.50	7088	1104.4	1161.5	1656.6	1742.2
2	7.00	9001	2805.0	2950.0	4207.5	4424.9
3	11.80	7683	4036.1	4244.6	6054.1	6366.9
4	16.60	7683	5677.9	5971.3	8516.8	8956.9
5	21.40	7683	7319.7	7697.9	10979.5	11546.8
6	26.20	4965	5791.2	6090.4	8686.8	9135.6
V_{base} (kN)			27892.7	28115.7	41839.1	42173.5

Total earthquake force applied to the structure and total shear force on shear walls at base level, are presented in Table 6.3 comparatively. As it can be observed from this table, as a result of implemented retrofit system, almost entire lateral load applied to the structure was resisted by shear walls.

Table 6.3 Comparison of Earthquake Loads to Shear Force on Shear Walls

x Direction			**y Direction**		
EQ Load F_x (kN)	**Shear Force on Shear Walls (kN)**	**% Ratio (Shear Force on Walls / EQ Load)**	**EQ Load F_y (kN)**	**Shear Force on Shear Walls (kN)**	**% Ratio (Shear Force on Walls / EQ Load)**
27892.7	25652.4	92	28115.7	26493.9	94

6.3.2 Structural Irregularity and Drift Ratio Check

Torsional irregularity and drift ratio of the retrofitted structure was also checked as the existing case. In Table 6.3 and 6.4 torsional irregularity coefficients calculated at each floor in the principal directions are summarized. In Table 6.5 and 6.6 interstorey drift ratio for two principal directions are presented.

Table 6.4 Retrofitted System Torsional Irregularity Check in the x – Direction

Floor #	δ_{imin} (m)	δ_{imax} (m)	δ_{iort} (m)	$\delta_{imax}/\delta_{iort}$	Check
1	0.0022	0.0027	0.0025	1.10	<1.4, ok
2	0.0055	0.0068	0.0061	1.11	<1.4, ok
3	0.0104	0.0131	0.0117	1.12	<1.4, ok
4	0.0114	0.0147	0.0130	1.13	<1.4, ok
5	0.0110	0.0143	0.0126	1.13	<1.4, ok
6	0.0100	0.0130	0.0115	1.13	<1.4, ok

Table 6.5 Retrofitted System Torsional Irregularity Check in the y - Direction

Floor #	δ_{imin} (m)	δ_{imax} (m)	δ_{iort} (m)	$\delta_{imax}/\delta_{iort}$	Check
1	0.002	0.003	0.002	1.03	<1.4, ok
2	0.005	0.006	0.006	1.03	<1.4, ok
3	0.009	0.010	0.009	1.03	<1.4, ok
4	0.009	0.009	0.009	1.03	<1.4, ok
5	0.007	0.007	0.007	1.03	<1.4, ok
6	0.005	0.006	0.005	1.04	<1.4, ok

Table 6.6 Retrofitted System Interstorey Drift Ratio Check in the x - Direction

		Immediate Occupancy			Life Safety		
Floor #	**hi (m)**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**
1	3.50	0.0027	0.00078	<0.008 ok	0.0041	0.00116	<0.02 ok
2	3.50	0.0068	0.00195	<0.008 ok	0.0102	0.00291	<0.02 ok
3	4.80	0.0131	0.00273	<0.008 ok	0.0196	0.00408	<0.02 ok
4	4.80	0.0147	0.00306	<0.008 ok	0.0220	0.00458	<0.02 ok
5	4.80	0.0143	0.00298	<0.008 ok	0.0214	0.00446	<0.02 ok
6	4.80	0.0130	0.00271	<0.008 ok	0.0194	0.00405	<0.02 ok

Table 6.7 Retrofitted System Interstorey Drift Ratio Check in the y - Direction

		Immediate Occupancy			Life Safety		
Floor #	**hi (m)**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**	**δ_{imax} (m)**	**δ_{imax}/h_i**	**Check**
1	3.50	0.0025	0.00073	<0.008 ok	0.0038	0.00109	<0.02 ok
2	3.50	0.0058	0.00166	<0.008 ok	0.0087	0.00249	<0.02 ok
3	4.80	0.0097	0.00203	<0.008 ok	0.0146	0.00305	<0.02 ok
4	4.80	0.0091	0.00189	<0.008 ok	0.0137	0.00285	<0.02 ok
5	4.80	0.0074	0.00155	<0.008 ok	0.0112	0.00233	<0.02 ok
6	4.80	0.0055	0.00115	<0.008 ok	0.0083	0.00173	<0.02 ok

Torsional irregularity and interstorey drift ratio requirements are both satisfied as expected. Torsional irregularity coefficient calculated for the retrofitted structure is a little higher than that of existing structure. Under the effect of lateral loads, nodes connected to the shear walls displace less than the nodes connected to the columns and this causes the difference between the maximum and minimum displacements at a floor.

6.3.3 Performance Levels of the Frame Elements

Performance levels of each frame element were determined under the effect of elastic earthquake loads. Then overall structural performance level was determined. The procedure which was employed to determine the performance level of the retrofitted structure was the same as the existing structure. However, in the retrofitted system there exist shear walls in addition to columns and beams. In order to determine performance level of shear walls, firstly the shear walls are classified as brittle or ductile according to their geometric properties and lateral reinforcement used. Then, their performance levels are decided, whether confinement exists or not. In the case study building, the shear walls were satisfying the geometric properties of ductile requirements and they were designed according to the Turkish Earthquake Code 1998 rules. Therefore, these shear walls were treated as ductile and confined. As in the existing structure, the remaining frame elements, i.e. beams and columns, were accepted as unconfined.

As described in the previous chapter, performance level of each element was decided by applying two different elastic earthquake loads. That is, under the effect of elastic earthquake load, which was calculated by using elastic spectrum having exceedance probability 10% in 50 years, all the elements were checked for immediate occupancy performance level. Then, they were checked for life safety performance level under the effect of earthquake load, which was calculated by using elastic spectrum having exceedance probability 2% in 50 years.

In Tables 6.8 through 6.15, summary results of the code based linear elastic procedure are presented. Column and shear walls summary results are given in same tables. As can be observed from these tables none of shear walls exceed the limiting values of target performance levels.

Table 6.8 Retrofitted System Linear Procedure Immediate Occupancy Performance Level x – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	23	4	17	>10%
2	23	3	13	>10%
3	23	9	39	>10%
4	23	8	35	>10%
5	23	9	39	>10%
6	23	13	57	>10%

Table 6.9 Retrofitted System Linear Procedure Immediate Occupancy Performance Level x – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns and Shear Walls	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	28	0	27515.3	0	0	≤0%
2	28	0	26319.0	0	0	≤0%
3	28	0	23371.5	0	0	≤0%
4	28	0	19129.3	0	0	≤0%
5	28	0	13202.7	0	0	≤0%
6	28	1	5533.7	41.0	0.7	>0%

Table 6.10 Retrofitted System Linear Procedure Life Safety Performance Level x – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	23	2	9	>20%
2	23	15	65	>20%
3	23	22	96	>20%
4	23	18	78	>20%
5	23	20	87	>20%
6	23	12	52	>20%

Table 6.11 Retrofitted System Linear Procedure Life Safety Performance Level x – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns and Shear Walls	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	28	0	41757.6	0	0	<20%
2	28	0	40025.5	0	0	<20%
3	28	0	35476.5	0	0	<20%
4	28	0	29124.0	0	0	<20%
5	28	0	20165.3	0	0	<20%
6	28	1	8632.6	82.0	0.95	<20%

Table 6.12 Retrofitted System Linear Procedure Immediate Occupancy Performance Level y – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	24	24	100	>10%
2	24	24	100	>10%
3	24	22	92	>10%
4	24	22	92	>10%
5	24	22	92	>10%
6	24	17	71	>10%

Table 6.13 Retrofitted System Linear Procedure Immediate Occupancy Performance Level y – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns and Shear Walls	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	28	0	27779.8	0	0	≤0%
2	28	0	26902.2	0	0	≤0%
3	28	0	23838.2	0	0	≤0%
4	28	2	19564.7	110.0	0.6	>0%
5	28	1	13548.5	48.5	0.4	>0%
6	28	0	5789.7	0	0	≤0%

Table 6.14 Retrofitted System Linear Procedure Life Safety Performance Level y – Direction Beams, Performance Level Summary

Floor#	Total Number of Beams	Number of beams not satisfying the performance level	Number Ratio (%) (Unsatisfied / Total)	Check
1	24	24	100	>20%
2	24	24	100	>20%
3	24	22	92	>20%
4	24	22	92	>20%
5	24	21	88	>20%
6	24	15	63	>20%

Table 6.15 Retrofitted System Linear Procedure Life Safety Performance Level y – Direction Columns, Performance Level Summary

Floor#	Total Number of Columns and Shear Walls	Number of columns not satisfying the performance level	Total Shear Force on Columns (kN)	Shear Force on Unsatisfied Columns (kN)	Shear Force Ratio % (Unsatisfied / Total)	Check
1	28	0	42124.5	0	0	<20%
2	28	0	40353.2	0	0	<20%
3	28	2	35757.3	195.0	0.5	<20%
4	28	2	29347.0	164.3	0.6	<20%
5	28	1	20322.8	73.8	0.4	<20%
6	28	0	8684.6	0	0	<20%

It is observed from the given tables that the retrofitted system cannot satisfy the target performance levels. Especially, flexural capacities of beams are insufficient in both directions.

In the x direction, approximately 30% to 80% of beams are beyond the expected limit, for both performance levels. In addition, at the 6^{th} storey, one column does not satisfy the immediate occupancy performance level. As it was stated, for the immediate occupancy performance level, the Turkish Earthquake Code 2006 does not allow any column beyond the minimum damage range. The same column cannot satisfy life safety performance level also. However, shear force resisted by the non confirming columns are less than 20% of total shear force on columns at that floor. Therefore, for life safety performance level, columns are in the acceptable ranges.

In the y direction, almost all of the beams are beyond the acceptable damage ranges for both expected performance levels. For both performance levels, E-5 and E-8 axes columns do not satisfy the objective criteria at some stories. But for the life safety performance level, shear force on these columns are less than 20% of total shear force at that floor. As a result, similar to the x direction, columns satisfy the life safety performance level criteria.

As a result, in both directions, the retrofitted structural system does not obey the target performance level criteria. This condition is mostly because of very low flexural capacities of beams. Moreover, some columns are also beyond the acceptable limits. For the life safety performance level, these columns satisfy the shear force requirements. However, for immediate occupancy performance level the Turkish Earthquake Code 2006 does not allow any column to be beyond the acceptable damage range. In both directions columns cannot satisfy the allowed limits of immediate occupancy performance level, because of this strict criterion. Finally, it can be concluded that, in spite of the applied retrofitting system, the structure is not safe enough according to the Turkish Earthquake Code 2006 requirements of linear elastic procedure.

6.4 ASSESSMENT OF RETROFITTED STRUCTURAL SYSTEM USING NONLINEAR PROCEDURES

6.4.1 Pushover Analysis

Similar to the existing structural system, retrofitted system was also assessed by nonlinear procedures according to the Turkish Earthquake Code 2006 and FEMA 273. In the nonlinear assessment of the retrofitted system, static pushover analysis method was used for the analysis tool.

Pushover curves calculated in each direction are presented in Figure 6.2 and 6.3. In addition to pushover curves of the retrofitted system, pushover curves for the existing system are given in the same graph. As it can be observed from these comparative diagrams, lateral load carrying capacity of the system was improved drastically by retrofitting.

In both directions, the structure remains elastic until yielding of newly added shear walls. After failure of some shear walls, high top deflections were calculated under the effect of smaller increases in lateral forces. In other words, inelastic behavior followed by the yielding of sufficient number of shear walls.

When the applied retrofit system is compared, it is seen that, number of shear walls placed in the y direction is more than that of x direction. However, the shear walls placed in the x direction are longer than the shear walls in the y direction and the flexural capacity of the shear walls in the x direction is more than that in the y direction. That is, shear walls in x direction yielded at a higher base shear force level.

Similar to assessment of the existing system, retrofitted system was also assessed according to Turkish Earthquake Code 2006 and FEMA 273 guideline by using pushover analysis.

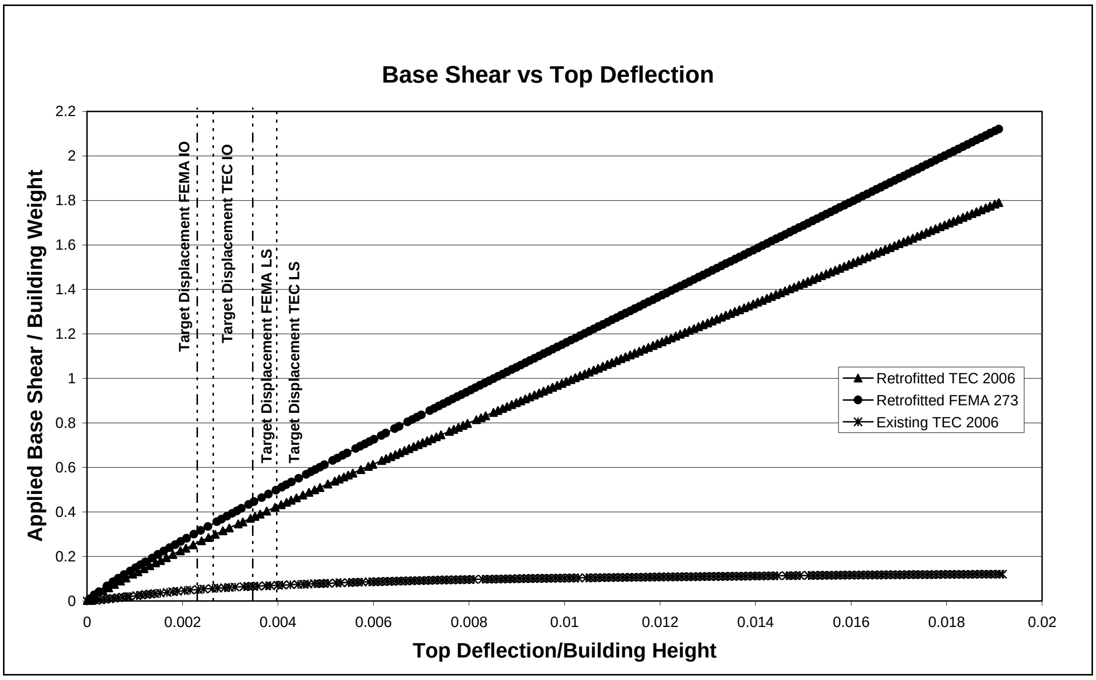


Figure 6.2 Retrofitted System Pushover Analysis Result in x-direction

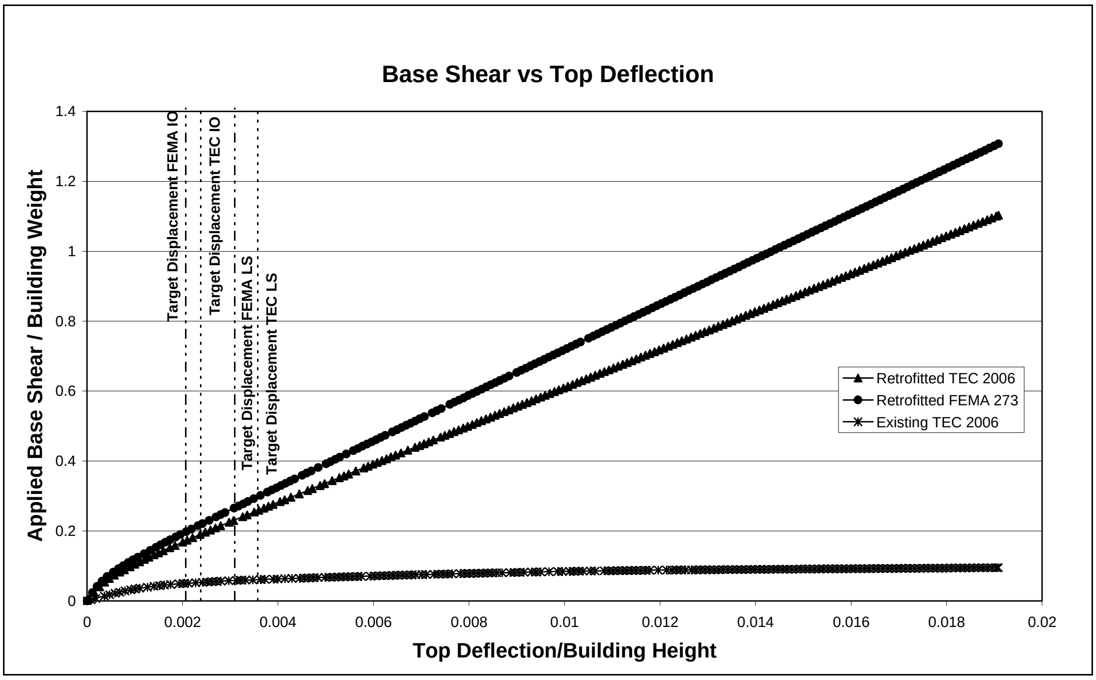


Figure 6.3 Retrofitted System Pushover Analysis Result in y-direction

Displacement demand, in other words target displacement for the retrofitted structure is determined by the same procedure given in the Chapter II. Displacement demand was calculated for two target performance levels and nonlinear deformation of each element was determined at these performance levels. In Figure 6.2 and 6.3 target displacements, calculated according to the Turkish Earthquake Code 2006 and FEMA 273 are also presented.

6.4.2 Performance Level of the Frame Elements

At target displacement load level, nonlinear deformations of yielded elements are calculated and compared with the limiting values of each damage range in order to determine the performance level of individual elements. Then overall structural performance level is determined according to the Turkish Earthquake Code 2006 and FEMA 273. This procedure is the same as that of followed in assessment of the existing system. In Tables 6.16 through 6.23 summary results of the nonlinear procedure are presented. Similar to linear assessment results, summary of shear walls and columns are given in same tables.

It is observed that, the retrofitted structure cannot satisfy the immediate occupancy performance level criteria because of some nonconforming beams in the y direction. According to the Turkish Earthquake Code 2006, at a storey, maximum ratio of beams not satisfying the immediate occupancy performance level cannot be more than 10% of total number of beams at that floor. However, in the y direction, this criterion has been exceeded.

For the life safety performance level on the other hand, all frame members are within the acceptable damage ranges and the structure satisfies requirements of this performance level according to applied nonlinear procedures.

Table 6.16 Retrofitted System Nonlinear Procedure Immediate Occupancy Performance Level x – Direction Beams, Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	23	0	0	<10%	0	0	<10%
2	23	0	0	<10%	0	0	<10%
3	23	0	0	<10%	0	0	<10%
4	23	0	0	<10%	0	0	<10%
5	23	1	4.3	<10%	1	4.3	<10%
6	23	1	4.3	<10%	1	4.3	<10%

Table 6.17 Retrofitted System Nonlinear Procedure Immediate Occupancy Performance Level x – Direction Columns Performance Level Summary

Floor #	Total Number of Columns and Shear Walls	Total Shear Force on Columns and Walls (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	28	27515.3	0	0	0	≤0	0	0	0	≤0
2	28	26319.0	0	0	0	≤0	0	0	0	≤0
3	28	23371.5	0	0	0	≤0	0	0	0	≤0
4	28	19129.3	0	0	0	≤0	0	0	0	≤0
5	28	13202.7	0	0	0	≤0	0	0	0	≤0
6	28	5533.7	0	0	0	≤0	0	0	0	≤0

Table 6.18 Retrofitted System Nonlinear Procedure Life Safety Performance Level x – Direction Beams, Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	23	0	0	≤20%	0	0	≤20%
2	23	0	0	≤20%	0	0	≤20%
3	23	0	0	≤20%	0	0	≤20%
4	23	0	0	≤20%	0	0	≤20%
5	23	0	0	≤20%	0	0	≤20%
6	23	0	0	≤20%	0	0	≤20%

Table 6.19 Retrofitted System Nonlinear Procedure Life Safety Performance Level x – Direction Columns, Performance Level Summary

Floor #	Total Number of Columns and Shear Walls	Total Shear Force on Columns and Walls (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	28	41757.6	0	0	0	≤20	0	0	0	≤20
2	28	40025.5	0	0	0	≤20	0	0	0	>20
3	28	35476.5	0	0	0	≤20	0	0	0	≤20
4	28	29124.0	0	0	0	≤20	0	0	0	≤20
5	28	20165.3	0	0	0	≤20	0	0	0	≤20
6	28	8632.6	0	0	0	≤20	0	0	0	≤20

Table 6.20 Retrofitted System Nonlinear Procedure Immediate Occupancy Performance Level y – Direction Beams, Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	24	0	0	≤10%	0	0	≤10%
2	24	0	0	≤10%	0	0	≤10%
3	24	4	16.7	>10%	4	16.7	>10%
4	24	4	16.7	>10%	4	16.7	>10%
5	24	4	16.7	>10%	4	16.7	>10%
6	24	4	16.7	>10%	4	16.7	>10%

Table 6.21 Retrofitted System Nonlinear Procedure Immediate Occupancy Performance Level y – Direction Columns, Performance Level Summary

Floor #	Total Number of Columns and Shear Walls	Total Shear Force on Columns and Walls (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	28	28083.0	0	0	0	≤0	0	0	0	≤0
2	28	26902.2	0	0	0	≤0	0	0	0	≤0
3	28	23838.2	0	0	0	≤0	0	0	0	≤0
4	28	19564.7	0	0	0	≤0	0	0	0	≤0
5	28	13548.5	0	0	0	≤0	0	0	0	≤0
6	28	5789.7	0	0	0	≤0	0	0	0	≤0

Table 6.22 Retrofitted System Nonlinear Procedure Life Safety Performance Level y – Direction Beams Performance Level Summary

Floor #	Total Number of Beams	Beams not Satisfying Performance Level					
		TEC 2006			FEMA 273		
		Number	Ratio %	Check	Number	Ratio %	Check
1	24	0	0	≤20%	0	0	≤20%
2	24	0	0	≤20%	0	0	≤20%
3	24	0	0	≤20%	0	0	≤20%
4	24	0	0	≤20%	0	0	≤20%
5	24	0	0	≤20%	0	0	≤20%
6	24	0	0	≤20%	0	0	≤20%

Table 6.23 Retrofitted System Nonlinear Procedure Life Safety Performance Level y – Direction Columns Performance Level Summary

Floor #	Total Number of Columns and Shear Walls	Total Shear Force on Columns and Walls (kN)	Columns not Satisfying Performance Level							
			TEC 2006				FEMA 273			
			No	Shear (kN)	Ratio %	Check	No	Shear (kN)	Ratio %	Check
1	28	42124.5	0	0	0	≤20	0	0	0	≤20
2	28	40353.2	0	0	0	≤20	0	0	0	≤20
3	28	35757.3	0	0	0	≤20	0	0	0	≤20
4	28	29347.0	0	0	0	≤20	0	0	0	≤20
5	28	20322.8	0	0	0	≤20	0	0	0	≤20
6	28	8684.6	0	0	0	≤20	0	0	0	≤20

6.5 COMPARISON OF THE ASSESSMENT PROCEDURES FOR THE RETROFITTED STRUCTURAL SYSTEM

Similar to the existing structural system, retrofitted system was assessed by using different codes and analysis procedures. Firstly, the system was subjected to linear elastic analysis and assessed by using Turkish Earthquake Code 2006. Then, by using approximate pushover analysis procedure, inelastic deformation capacity of the structure was determined. At target displacement level, which is calculated by using inelastic structural properties, plastic deformation of each yielded members were calculated and compared with the acceptable limits given in FEMA 273 and Turkish Earthquake Code 2006. Assessment results of all these methods are given separately in above parts of this chapter.

In Appendix C assessment results of retrofitted structural system for life safety performance level are given in graphical representations. In addition to these graphics, comparative assessment results are presented on typical frames which are given in Figures 6.4 through 6.7. In these figures, element sections are marked according to their damage levels. Legends used in these figures are presented in Table 6.24.

Table 6.24 Legend for Frame Section Performance Levels

Used Mark	Corresponding Member Performance Level
	Performance Level<IO
○	IO< Performance Level <LS
●	LS< Performance Level <CP
X	Performance Level >CP

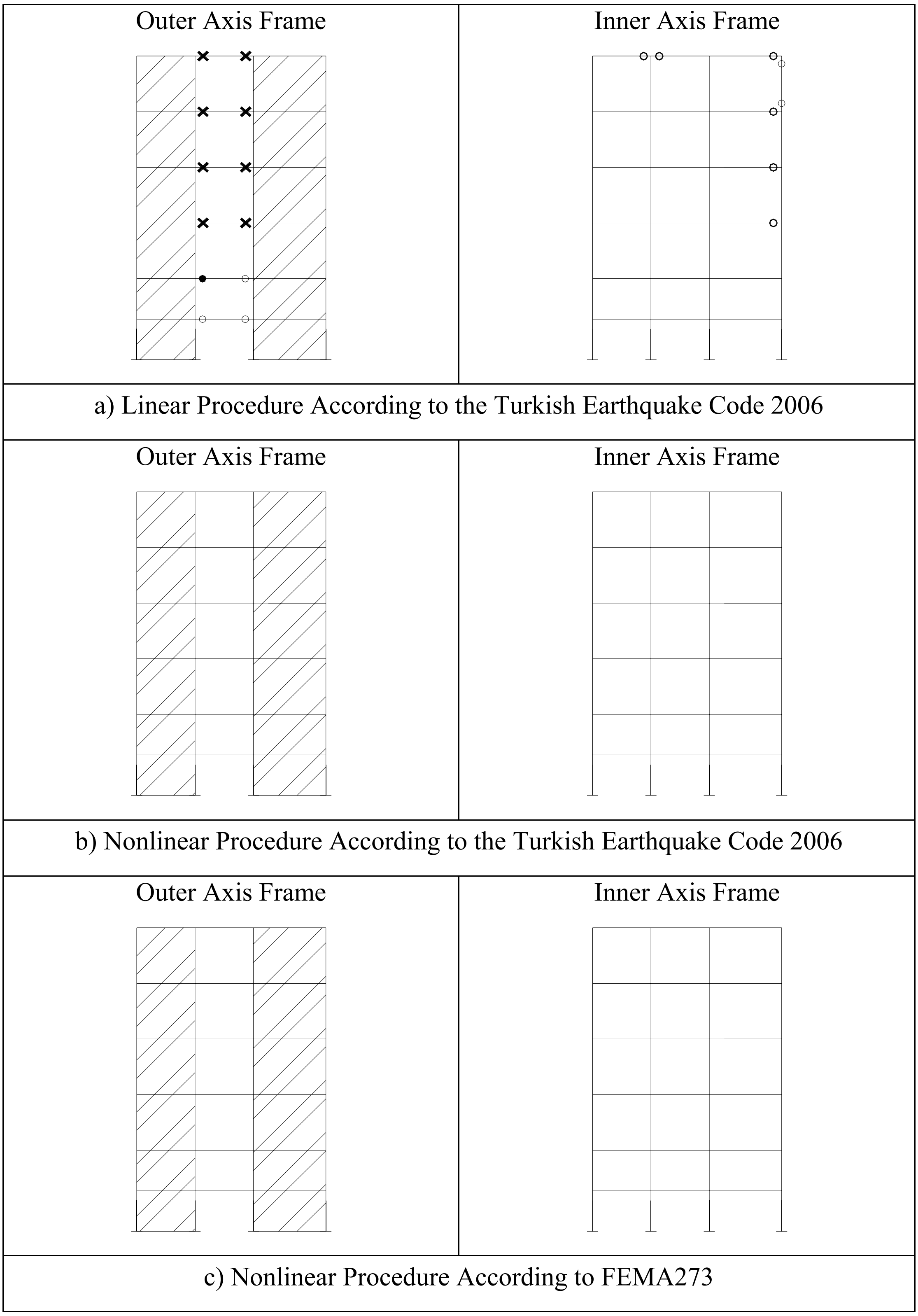


Figure 6.4 Retrofitted System: Comparison of Element Section Performance Levels, Immediate Occupancy Performance Level in x Direction

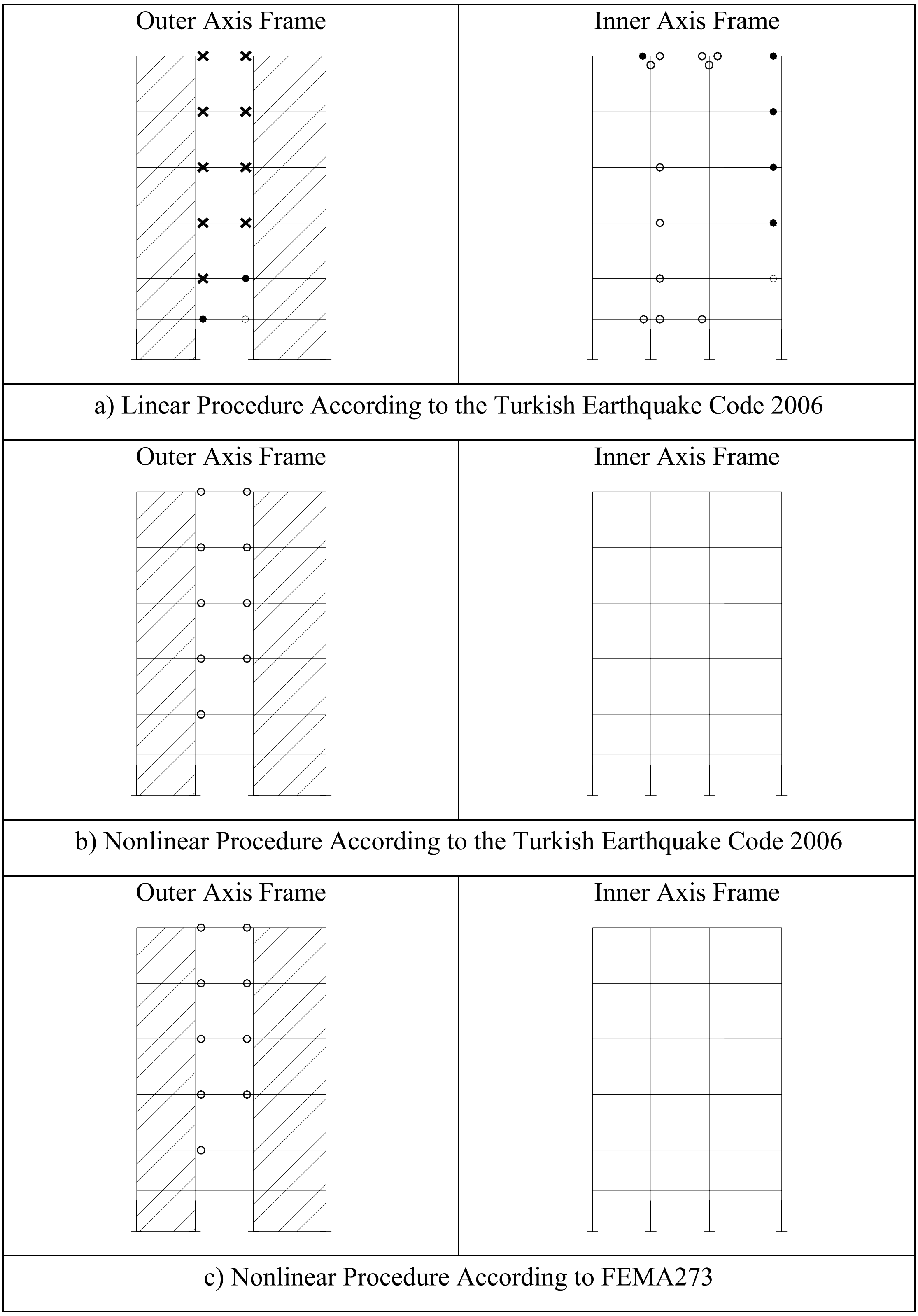
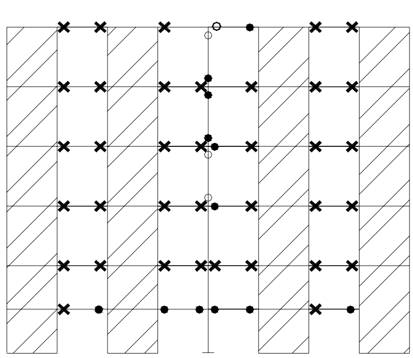
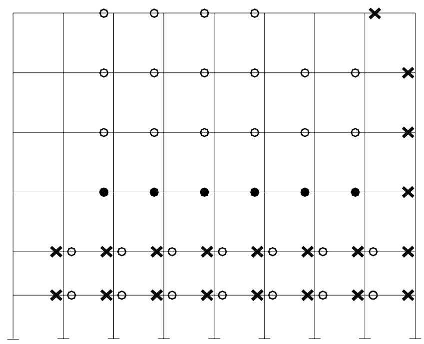
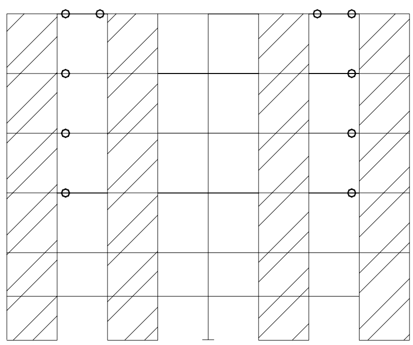
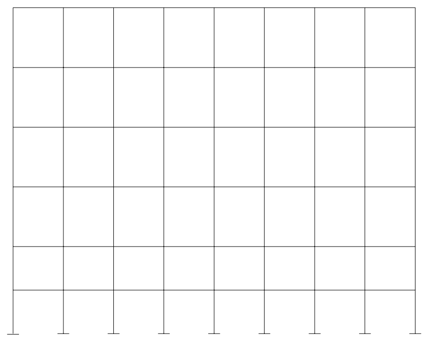


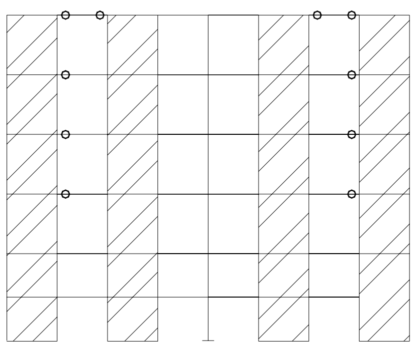
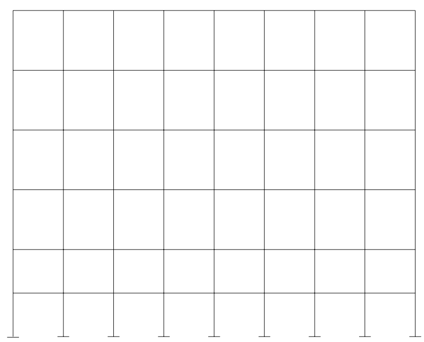
Figure 6.5 Retrofitted System: Comparison of Element Section Performance Levels, Life Safety Performance Level in x Direction

Outer Axis Frame	Inner Axis Frame
	

a) Linear Procedure According to the Turkish Earthquake Code 2006

Outer Axis Frame	Inner Axis Frame
	

b) Nonlinear Procedure According to the Turkish Earthquake Code 2006

Outer Axis Frame	Inner Axis Frame
	

c) Nonlinear Procedure According to FEMA273

Figure 6.6 Retrofitted System: Comparison of Element Section Performance Levels, Immediate Occupancy Performance Level in y Direction

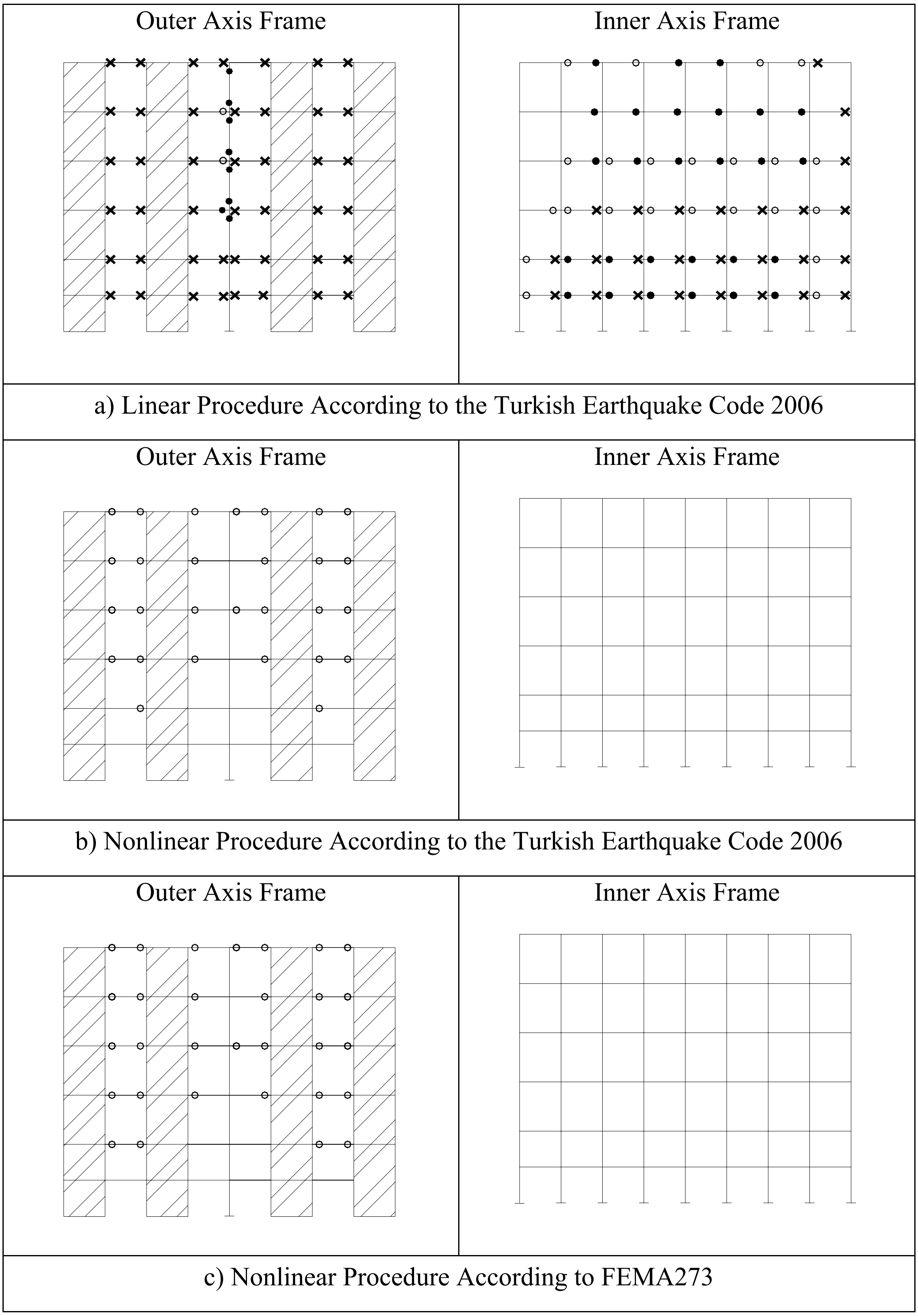


Figure 6.7 Retrofitted System: Comparison of Element Section Performance Levels, Life Safety Performance Level in y Direction

As it can be observed from these figures and given tables, like the assessment of existing system, there exist some differences between linear and nonlinear assessment results. However, difference between the linear and nonlinear procedures is not that much as in the linear procedure.

As the structure becomes rigid, the load level beyond which the structure undergoes nonlinear action increased and the target displacement load level comes closer to the equivalent elastic lateral load calculated according to the Turkish Earthquake Code 2006. However, since redistribution effect is considered explicitly during the nonlinear analysis, assessment results obtained by using linear method yields more conservative results.

When the graphics and the above figures are considered together, it is clear that, similar to the existing case, nonlinear assessment results obtained by employing the Turkish Earthquake Code 2006 and FEMA 273 are almost the same.

It is obvious from the applied assessment results, almost all of the applied lateral loads are resisted by newly added shear walls. Columns carry approximately 5% to 10% of total applied lateral loads. As can be seen from the given figures and graphics, almost all of the columns remain elastic or undamaged under the effect of lateral loads.

In spite of the applied retrofitting system, according to the linear assessment results, the structure cannot satisfy the immediate occupancy performance level criteria given in the Turkish Earthquake Code 2006. As it was stated in the previous chapters, this retrofitting system is directly adopted from an applied project. In this project the structure was analyzed using the equivalent lateral load method assuming the structure consists of ordinary moment resisting frame system. That is, ductility ratio defined in the Turkish Earthquake Code 1998 was assumed as 4. In addition, importance factor of 1.5 was also considered in earthquake load analysis. The newly added shear walls were designed according to inelastic equivalent lateral load analysis results. However, in this assessment, unlike the assessment procedures considered or discussed in this study, beam performance levels were not considered. Because, at the time when this retrofitted system had been designed, it was thought that, when the structure satisfies the drift limits and the columns have enough flexural and axial load capacity under the effect of lateral load, the structure can be considered as satisfactory. In other words, at that time when the retrofitted system was designed, flexural capacities of the beams were not being checked considering that if the structure satisfies the strong column weak beam action during a ground excitation, the structure may undergo some inelastic action but it will remain stable. And it was also accepted that yielding of one or two columns is not that important since almost all of the applied lateral loads were resisted by the newly added shear walls.

According to linear assessment results, the implemented retrofitting system is not sufficient. When the nonlinear assessment results are considered, it can be concluded that the retrofitted structure satisfy the target performance levels criteria. Although there are some unsatisfying beams in y direction, these beams can be accepted as in tolerable ranges.

CHAPTER VII

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDIES

7.1. SUMMARY

A reinforced concrete structure, which was designed in 1972, is assessed by employing different codes or guidelines and different analysis procedures in this study. The case study building, which is located in Gayrettepe, İstanbul is a six storey reinforced concrete structure and it serves as a telephone exchange building.

There was no damage observed in the building as a result of earthquakes occurred in the Marmara Region in the last decade. Although the structure has remained undamaged after the earthquakes, the existing structure was retrofitted since the structure has a primary importance after an earthquake. In addition to the assessment of the existing system, the retrofitted system of the structure is also assessed by following the same procedure for the existing system.

As it was stated in the previous chapters, both existing and retrofitted structures are first subjected to linear elastic analysis by employing the equivalent lateral load procedure. By using the analysis results, the structural systems were assessed according to the Turkish Earthquake Code 2006. Then, both structural systems were analyzed by employing the approximate pushover method which was briefly described in Chapter IV. Inelastic displacements were calculated as a result of this analysis and finally, both structural systems were assessed by using the nonlinear procedures proposed in the Turkish Earthquake Code 2006 and FEMA 273.

Placing the new shear walls in the existing moment resisting frames, the existing system was retrofitted. This retrofitting method is widely used because of its simplicity. In this method, the newly added shear walls were placed and designed to resist all the lateral loads applied to the structure.

7.2. CONCLUSIONS

The following conclusions and results can be drawn as a result of this comparative study:

- When the pushover curves calculated by using the approximate pushover analysis and by employing SAP 2000 pushover analysis are compared, it can be concluded that the approximate pushover method gives satisfactory results. Furthermore, the approximate pushover method is suitable for computer program to be a post processor of a readily available structural analysis programs.

- According to linear assessment procedure, the structure cannot satisfy the target performance levels. Beams at all stories exceeded the limiting values of corresponding performance levels. In addition, very weak columns are also beyond the acceptable limits.

- The existing system of the case study building cannot satisfy the target performance level according to the nonlinear assessment procedures. As a results of nonlinear analysis by employing the Turkish Earthquake Code 2006 and FEMA 273 almost the same performance levels of each individual frame members were obtained. However, it is clear that FEMA 273 procedure gives conservative results as compared with the Turkish Earthquake Code 2006.

- When the linear and nonlinear analysis results were compared, it is observed that there are several differences between the performance levels of frame members.

- According to linear assessment of the retrofitted structure, although performance levels of elements are improved considerably, the structure cannot satisfy the target performance level criteria given in the Turkish Earthquake Code 2006. This is mainly because of very low flexural capacities of the beams.

- As a result linear assessment procedure, it is concluded that the retrofitting system proposed for this building, which was designed by using the equivalent lateral load analysis procedure proposed in the Turkish Earthquake Code 1998, is insufficient according to the Turkish Earthquake Code 2006 regulations. In order to make it a satisfactory retrofitting system according to the Turkish Earthquake Code 2006, in addition to shear walls some individual member should also be strengthened.

- According to pushover analysis results, the lateral load capacity of the structure is increased considerably after retrofit.

- When the nonlinear assessment results are considered, performance levels of each element are improved. The structure can be accepted as satisfactory for both target performance levels. Although there are some nonconforming beams in y direction these beams can be accepted as in tolerable ranges.

- Nonlinear assessment results calculated according to the Turkish Earthquake Code 2006 and FEMA 273 are almost the same. However, similar to the existing case assessment, FEMA 273 gives more conservative results.

- When the nonlinear and linear assessment results of the retrofitted structure are compared, there are still some differences between both procedures. However, the difference is not that much as in the existing case because as the structure reaches its target displacement it still remains elastic. The differences on beam performance levels are mainly due to the redistribution effect which is considered in nonlinear analysis.

7.3. FUTURE RECOMMENDATIONS

In this limited study, the case study building of existing and retrofitted system were assessed by using different codes or guidelines and different methods.

In this study, efficiency of an approximate pushover analysis method which was briefly described in Chapter IV was also investigated. This approximate pushover method may be developed for a computer program which may be post processor of a readily available structural analysis program.

As it was stated in the previous paragraphs, linear and nonlinear assessment results of the existing system is too different from each other. In order to assess those kinds of very ductile structures, more realistic linear and nonlinear methods may be developed. Furthermore, in order to improve correlation between the linear and nonlinear assessment procedures, more buildings may be studied comparatively.

In addition, it is clear that generally the linear assessment procedures give conservative results as compared with the nonlinear assessment procedures. The linear assessment procedures are insufficient to consider the redistribution effect. Therefore, the linear procedures may further be improved in order to consider the redistribution effect for different kind of buildings. Moreover, in order to take consideration the location of the yielded members, acceptance criteria may be revised for especially immediate occupancy performance level.

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APPENDIX A

INTERACTION DIAGRAMS FOR COLUMNS AND SHEAR WALLS AT A TYPICAL FLOOR

Interaction diagrams for columns and newly added shear walls at a typical storey are presented in the next pages. These interaction diagrams were calculated by using RESPONSE 2000 (Bentz E. C., 2000) program. As it was stated in Chapter III, in calculating these interaction diagrams, characteristic values for concrete and reinforcement were used. Concrete characteristic strength value was determined from test results of core samples. In Table 3.1, core samples locations and test results are presented briefly.

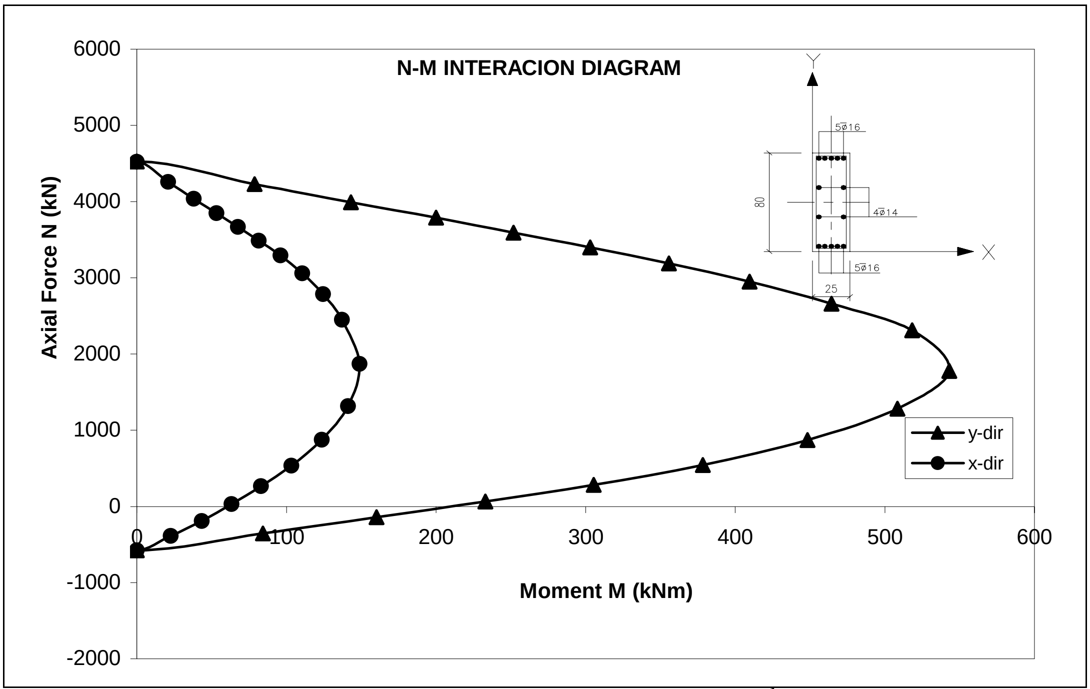


Figure A.1 Interaction Diagrams for Columns at 3rd Storey on Axes B-5/8, C-5/8, D-5/8, E-5/8, F-5/8, G-5/8, H-5/8

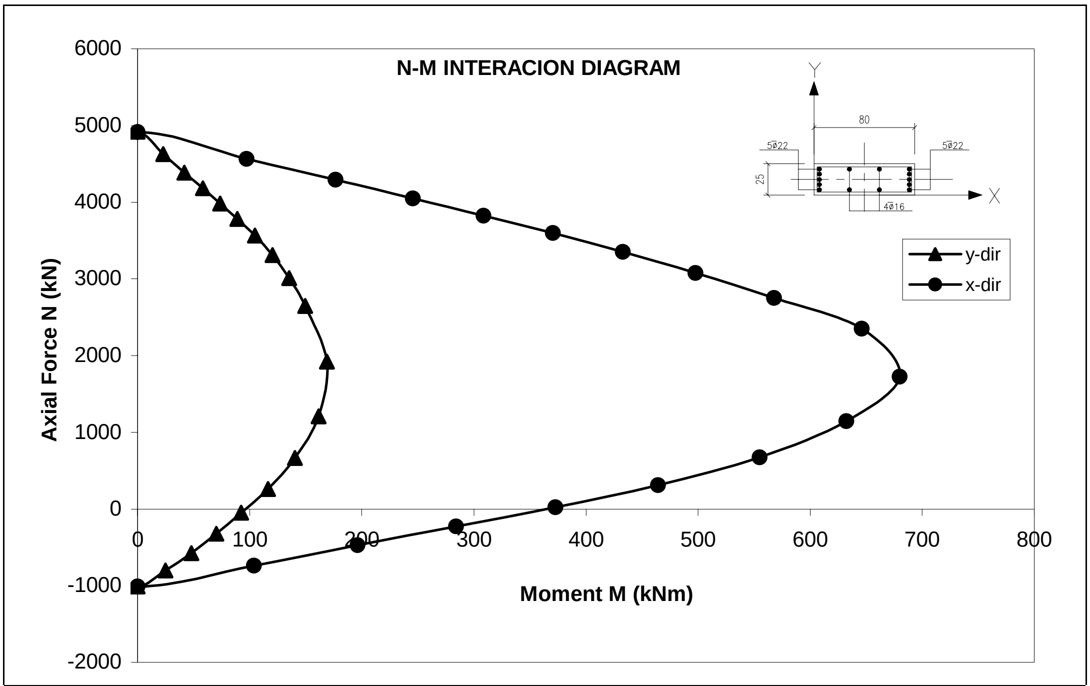


Figure A.2 Interaction Diagrams for Columns at 3rd Storey on Axes A-6/7, I-6/7

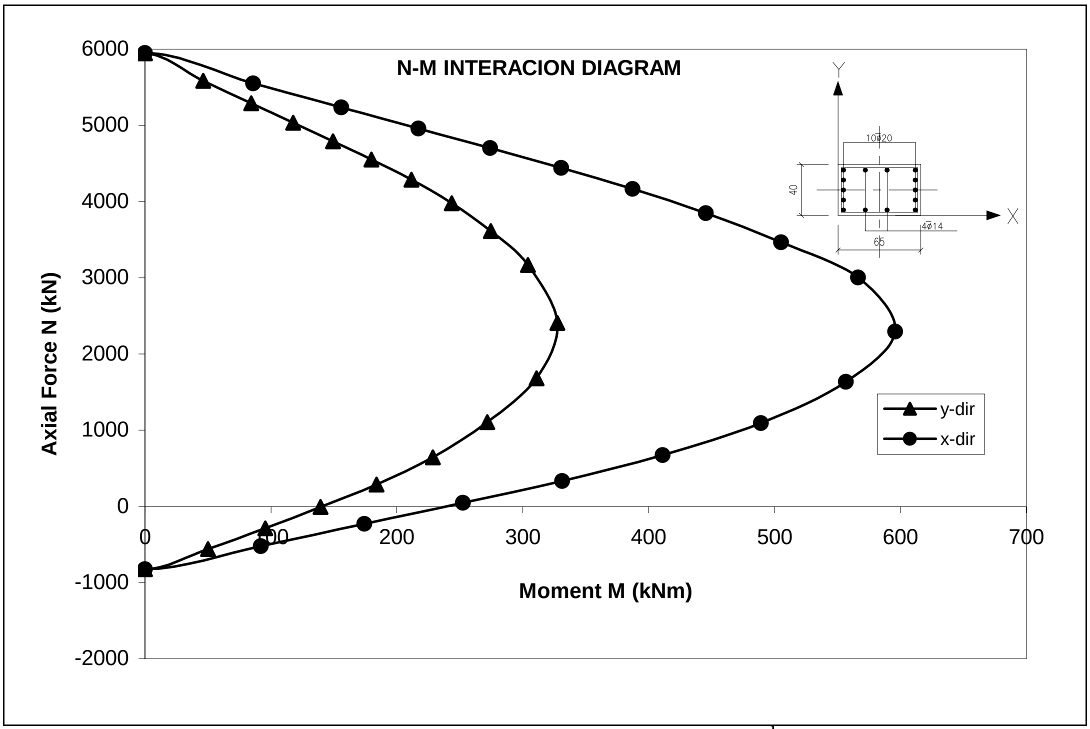


Figure A.3 Interaction Diagrams for Columns at 3rd Storey on Axes B-6, C-6, D-6, E-6, F-6, G-6, H-6

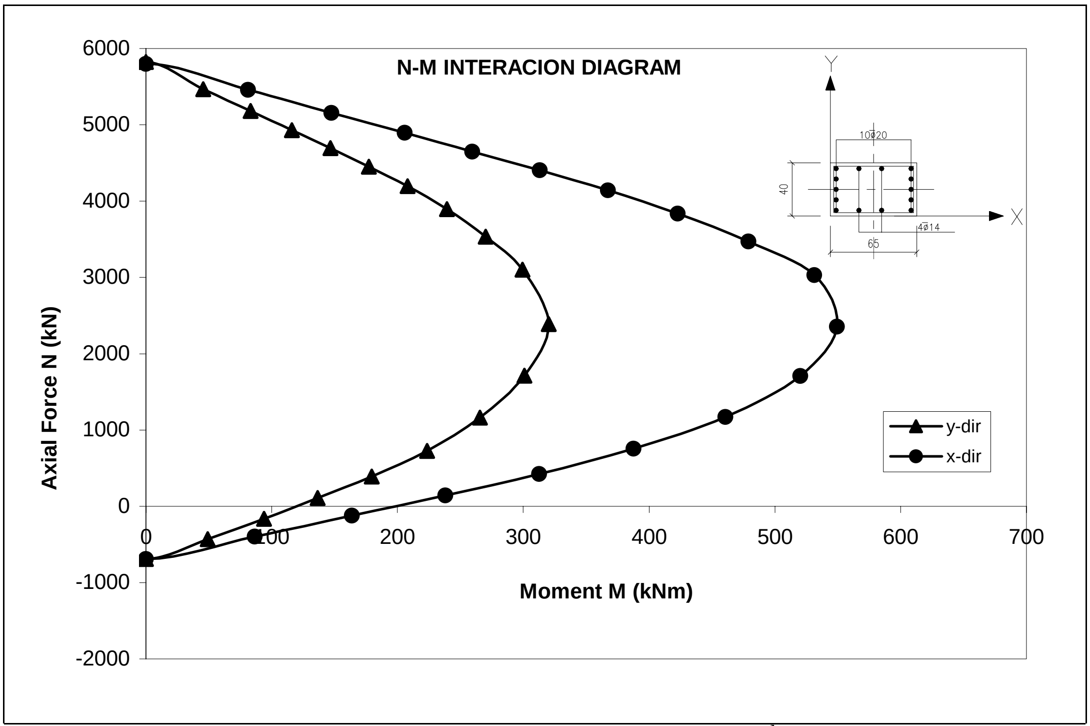


Figure A.4 Interaction Diagrams for Columns at 3rd Storey on Axes B-7, C-7, D-7, E-7, F-7, G-7, H-7

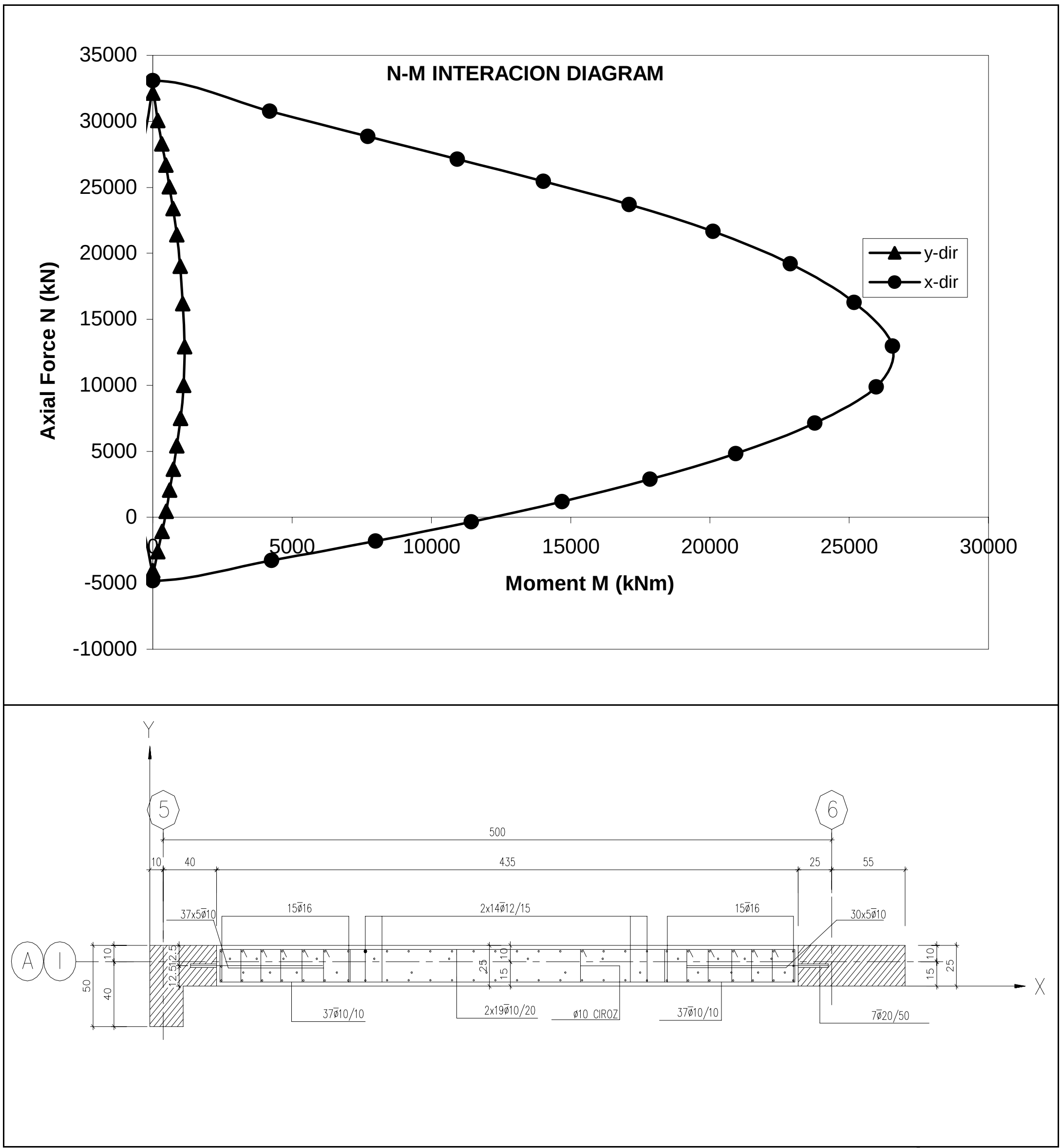
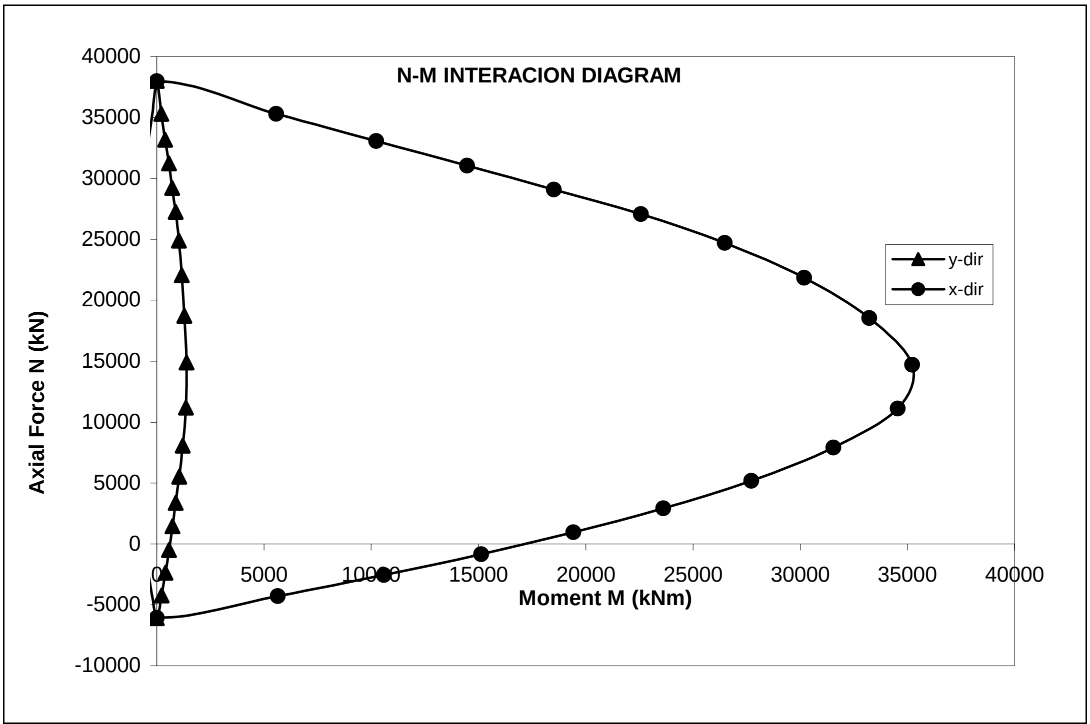


Figure A.5 Interaction Diagrams for Newly Added Shear Walls at 3rd Storey on Axes A-5/6, I-5/6



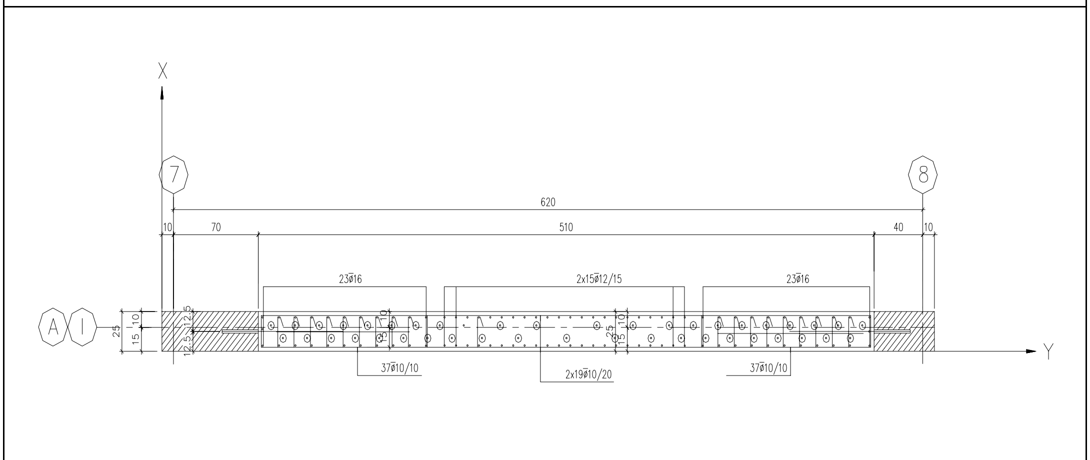


Figure A.6 Interaction Diagrams for Newly Added Shear Walls at 3rd Storey on Axes A-7/8, I-7/8

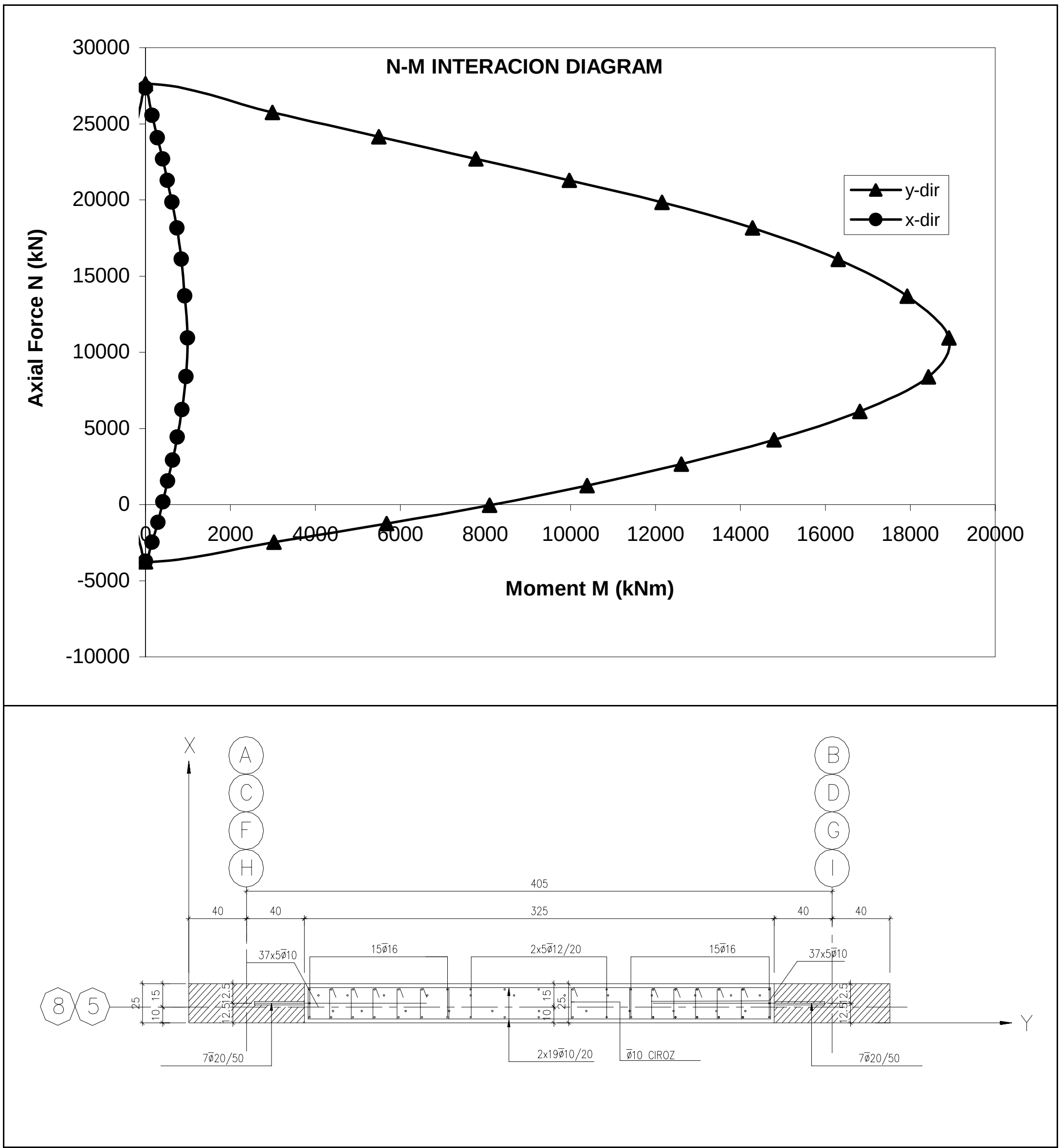


Figure A.7 Interaction Diagrams for Newly Added Shear Walls at 3rd Storey on Axes A/B-5, C/D-5, F/G-5, H/I-5, A/B-8, C/D-8, F/G-8, H/I-8

APPENDIX B

EXISTING SYSTEM, LIFE SAFETY PERFORMANCE LEVEL, GRAPHICAL REPRESENTATION OF THE COMPERATIVE ASSESSMENT RESULTS

In this appendix, existing system comparative assessment results of all frame elements for life safety performance level are presented in graphical forms. Comparative results summary for both immediate occupancy and life safety performance levels are also given in Chapter V.

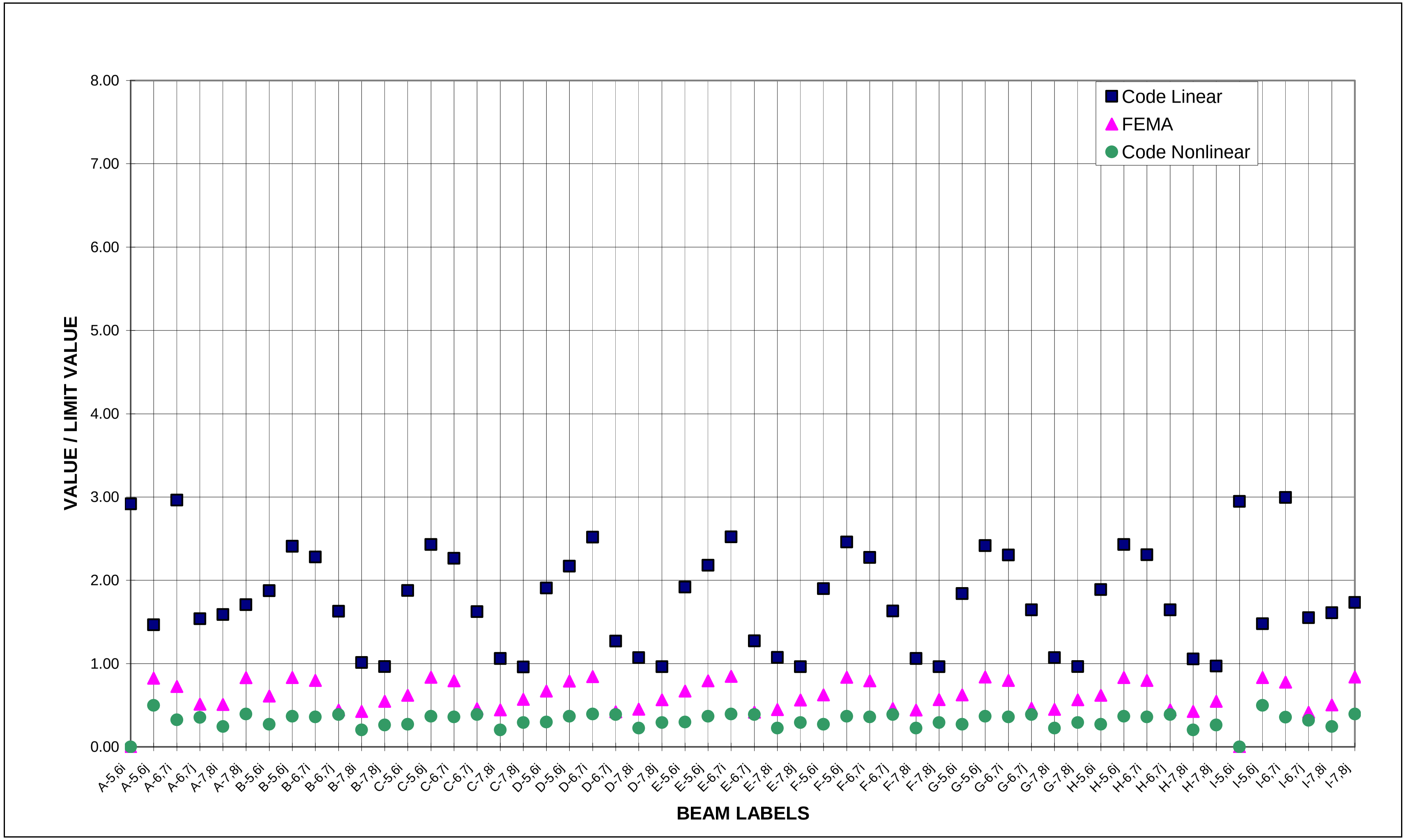


Figure B.1 Existing System Life Safety Performance Level 1st Storey x – Direction Beams Performance Levels

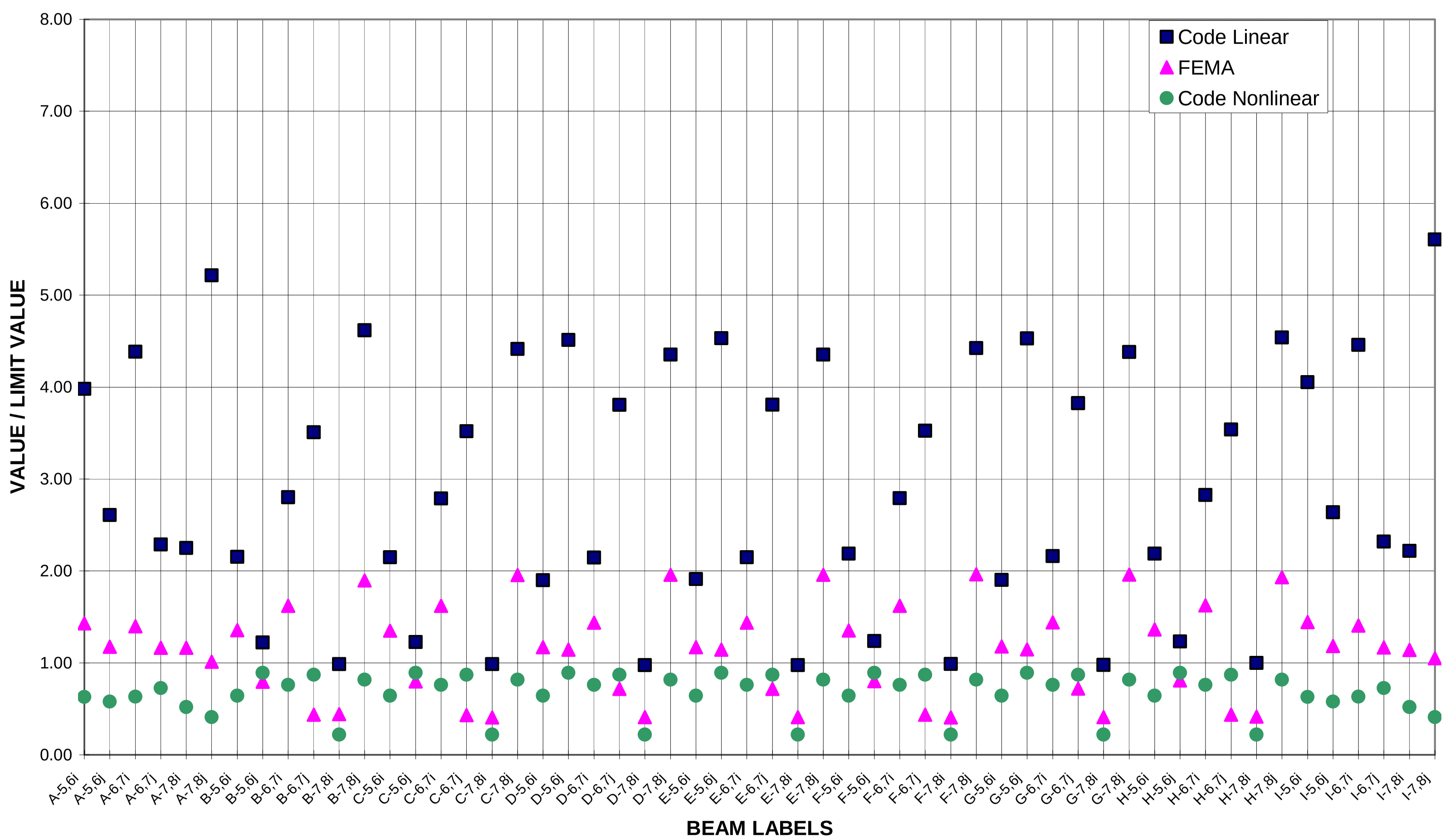


Figure B.2 Existing System Life Safety Performance Level 2nd Storey x – Direction Beams Performance Levels

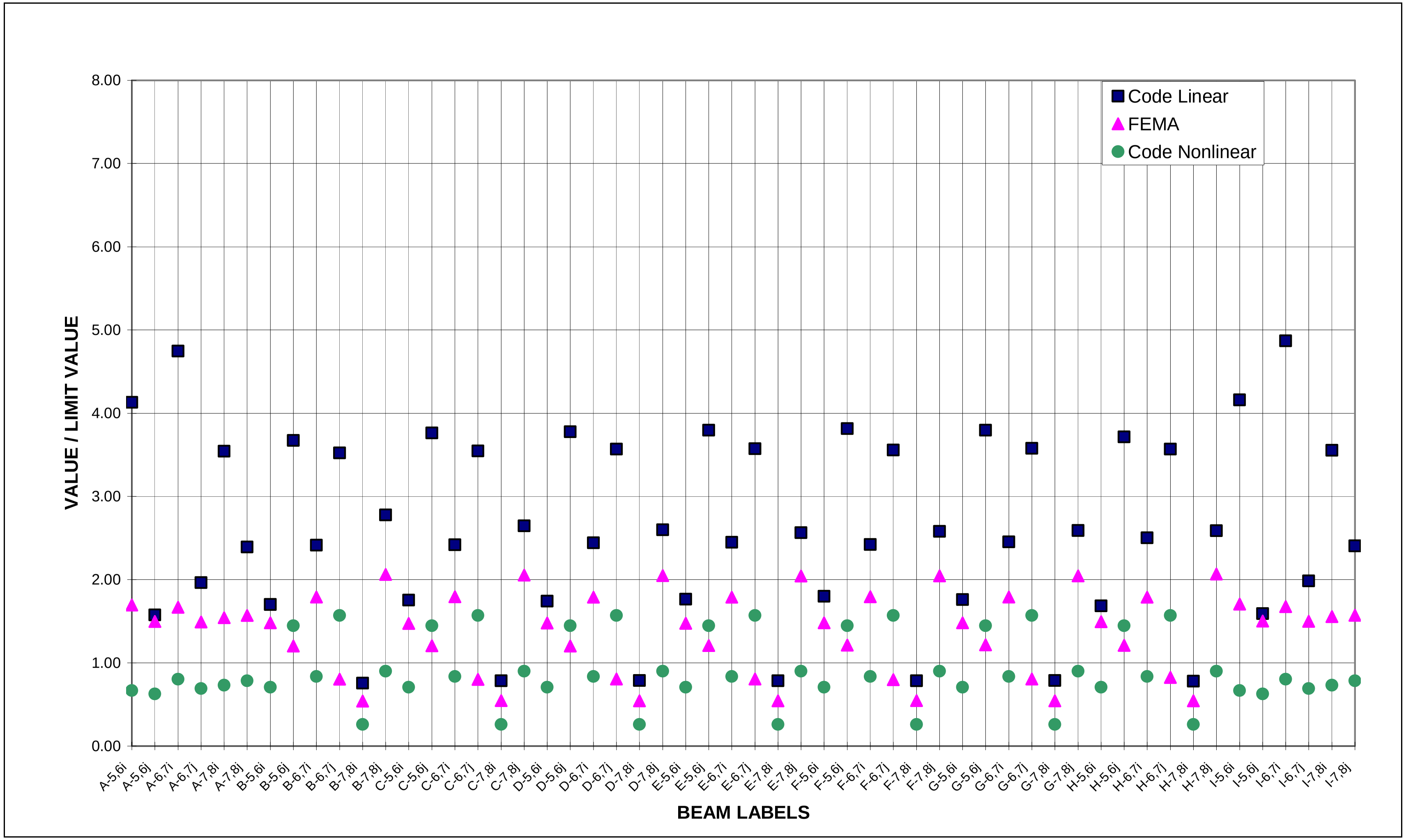


Figure B.3 Existing System Life Safety Performance Level 3rd Storey x – Direction Beams Performance Levels

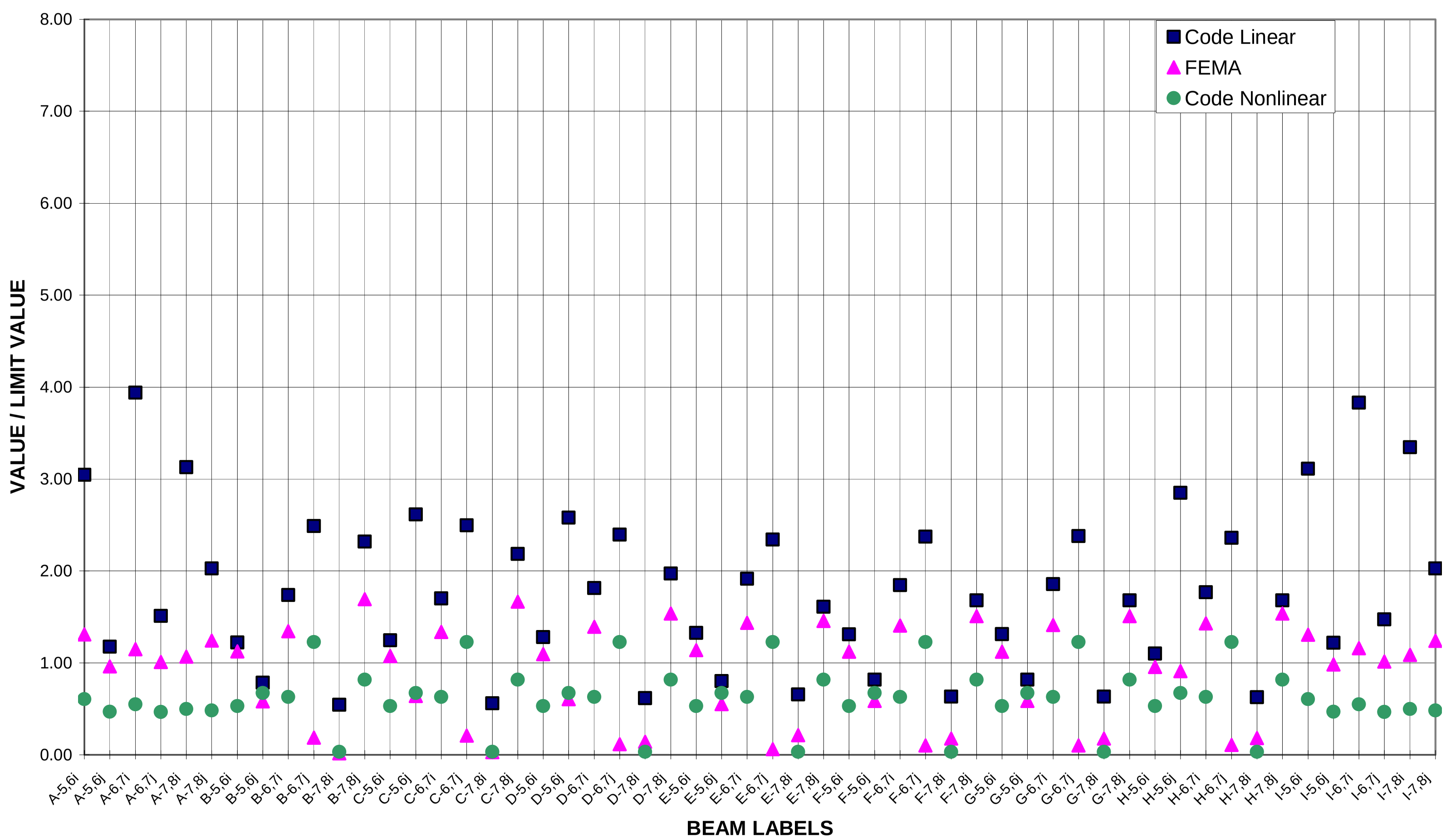


Figure B.4 Existing System Life Safety Performance Level 4th Storey x – Direction Beams Performance Levels

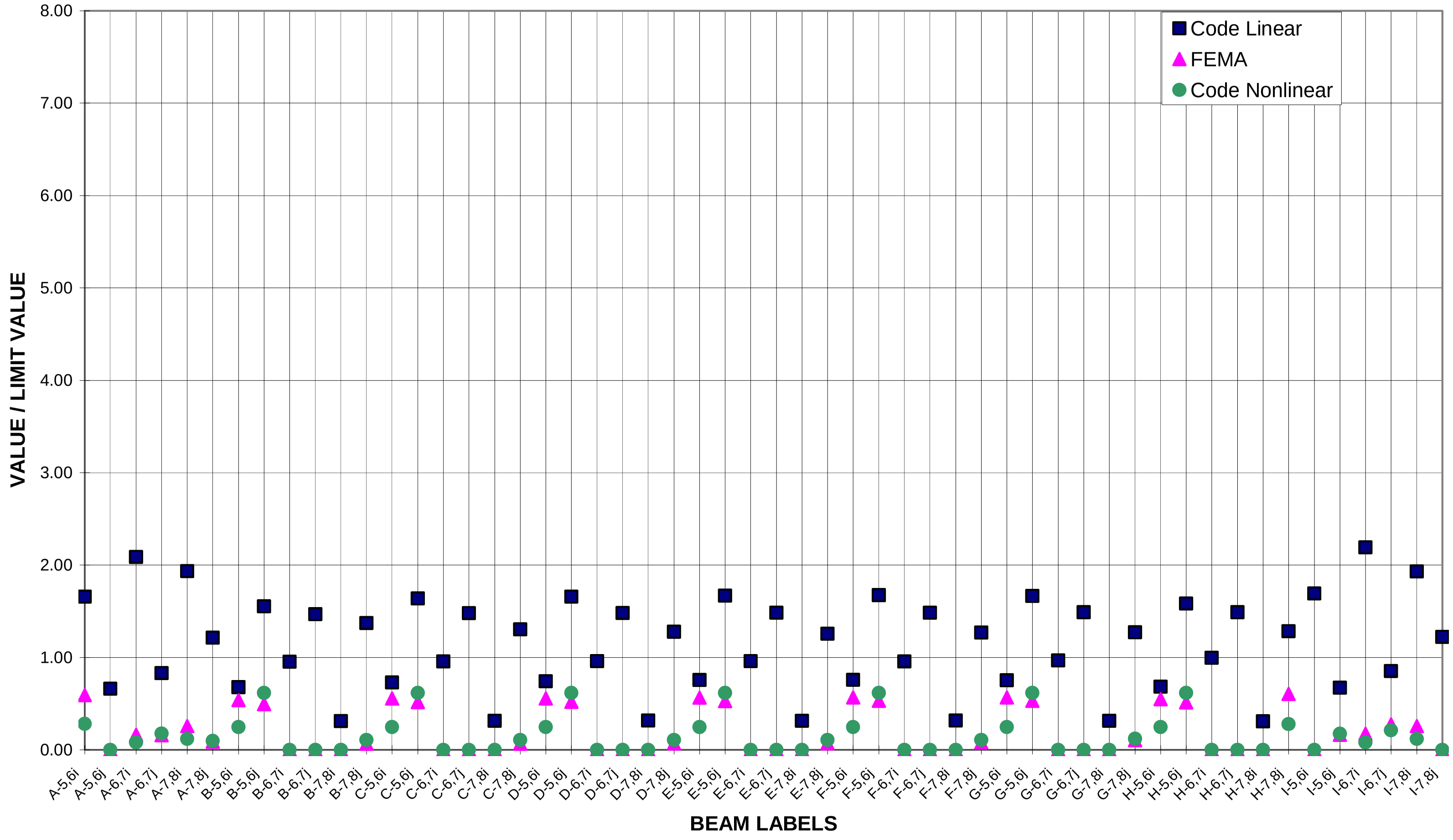


Figure B.5 Existing System Life Safety Performance Level 5th Storey x – Direction Beams Performance Levels

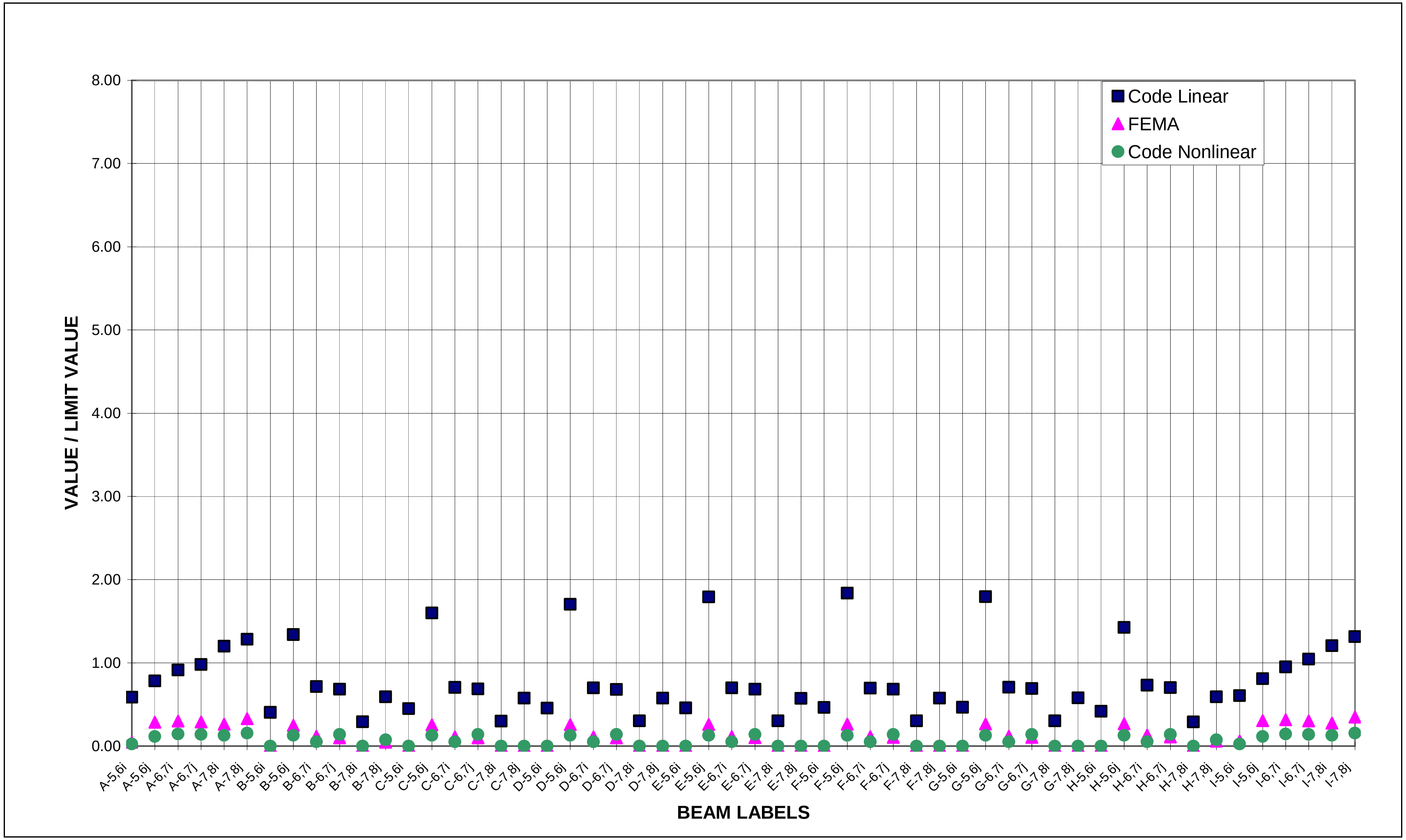


Figure B.6 Existing System Life Safety Performance Level 6th Storey x – Direction Beams Performance Levels

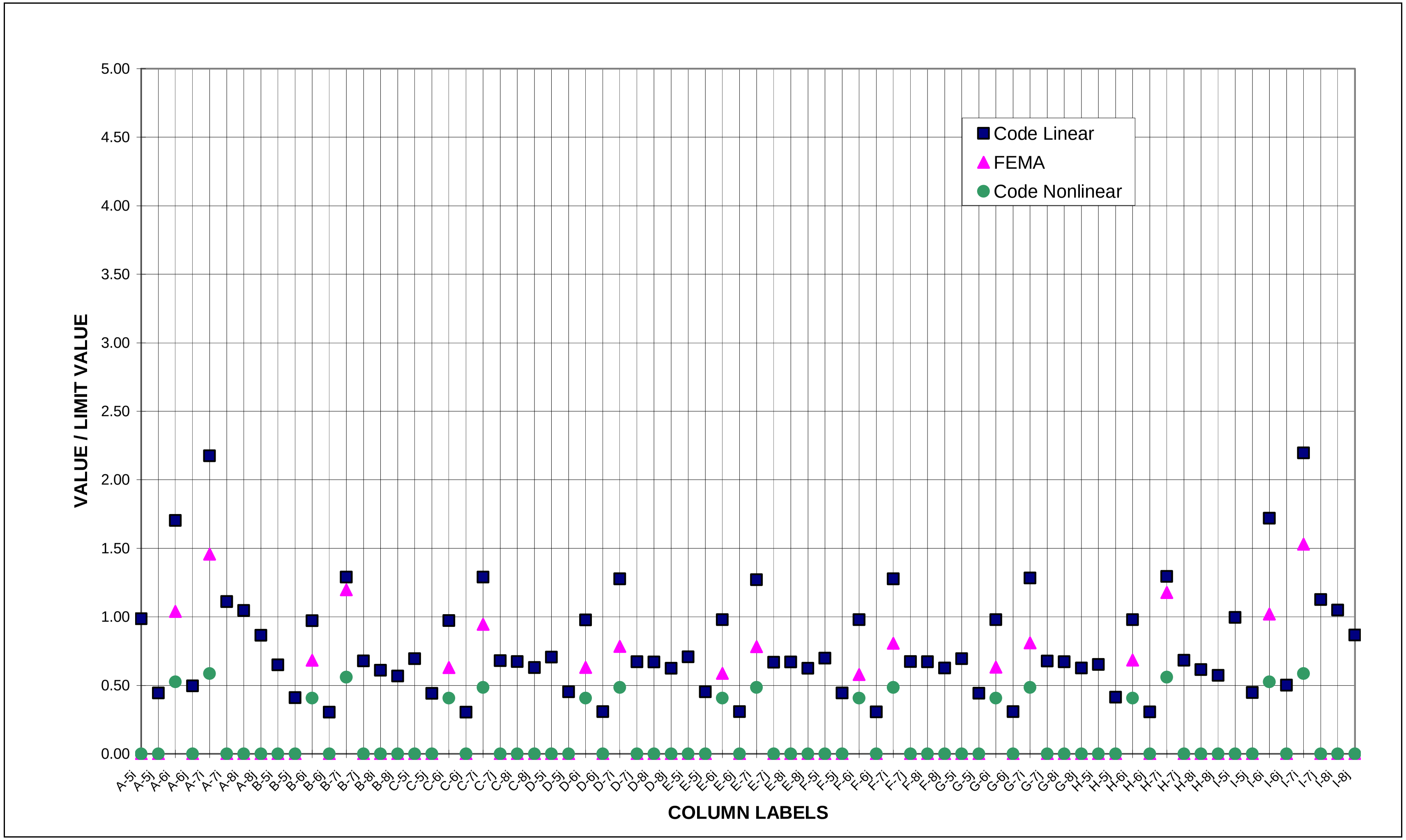


Figure B.7 Existing System Life Safety Performance Level 1st Storey x – Direction Columns Performance Levels

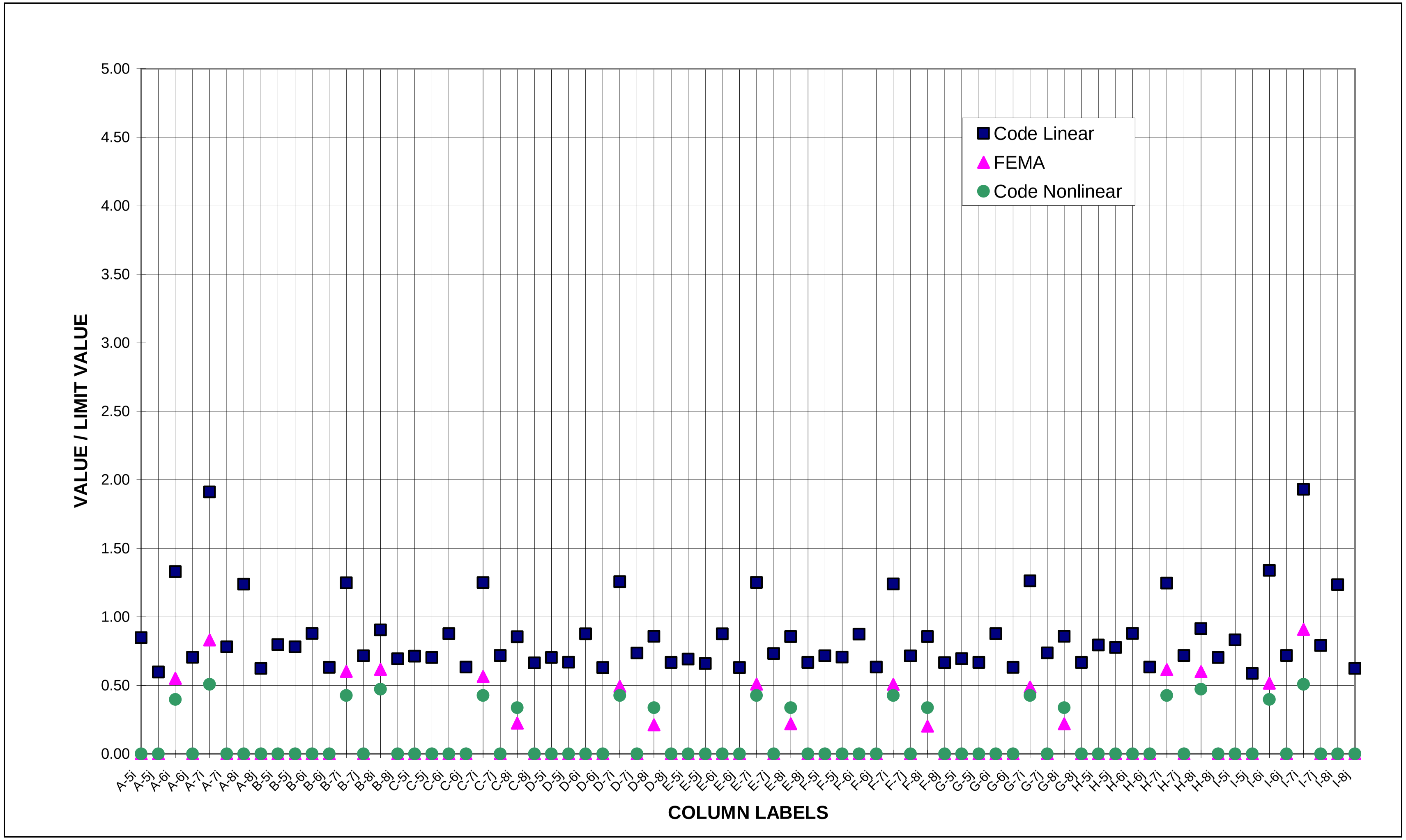


Figure B.8 Existing System Life Safety Performance Level 2nd Storey x – Direction Columns Performance Levels

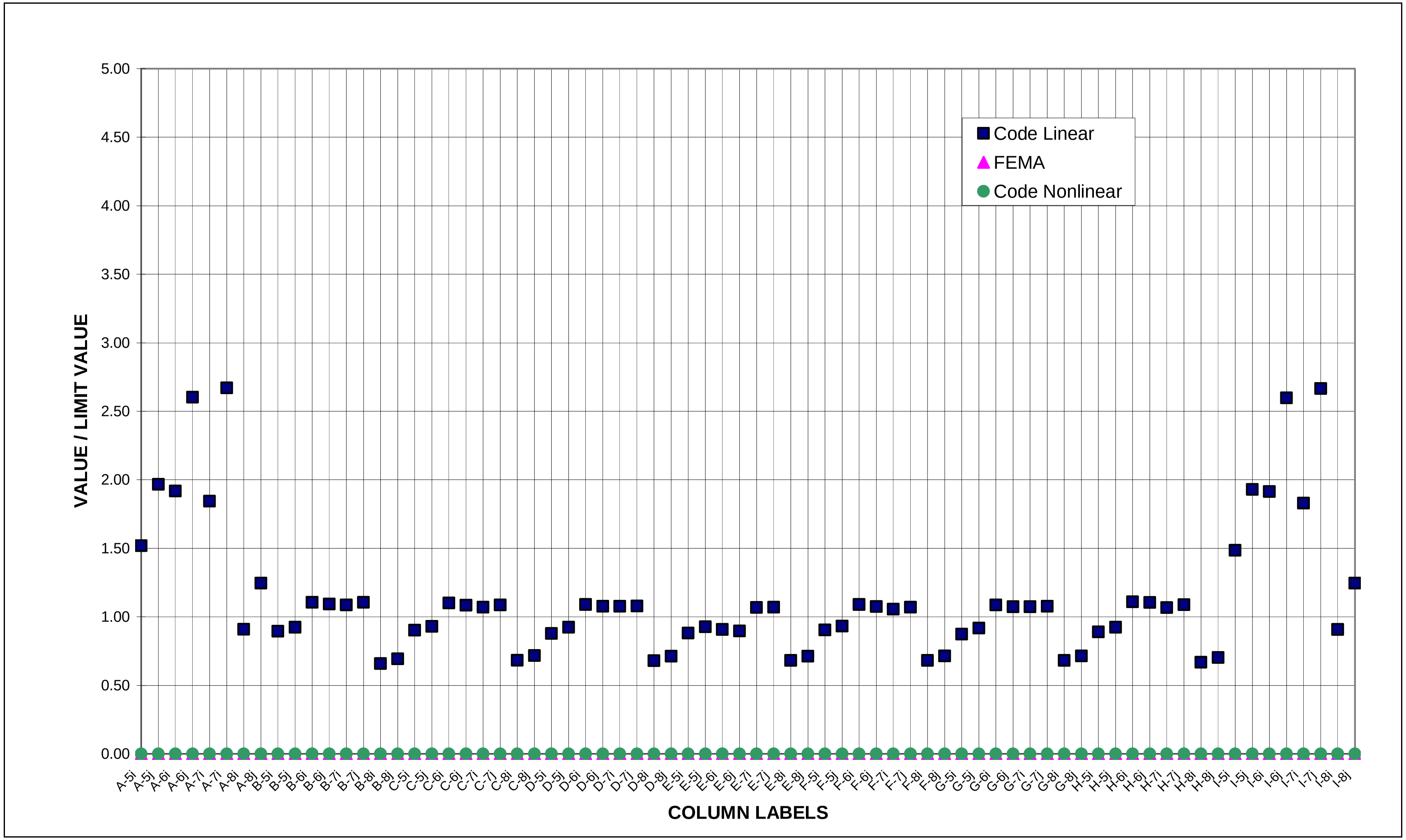


Figure B.9 Existing System Life Safety Performance Level 3rd Storey x – Direction Columns Performance Levels

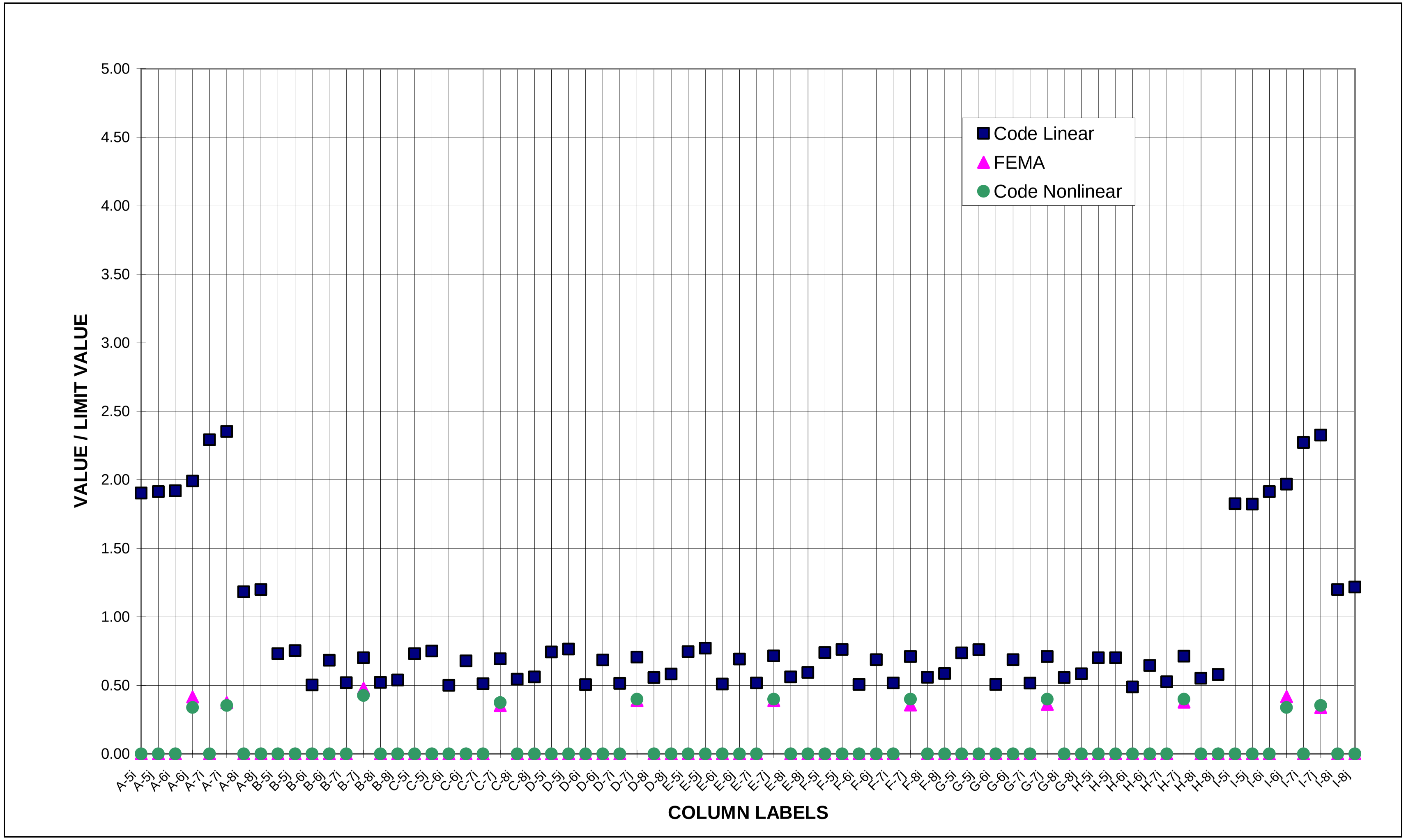


Figure B.10 Existing System Life Safety Performance Level 4th Storey x – Direction Columns Performance Levels

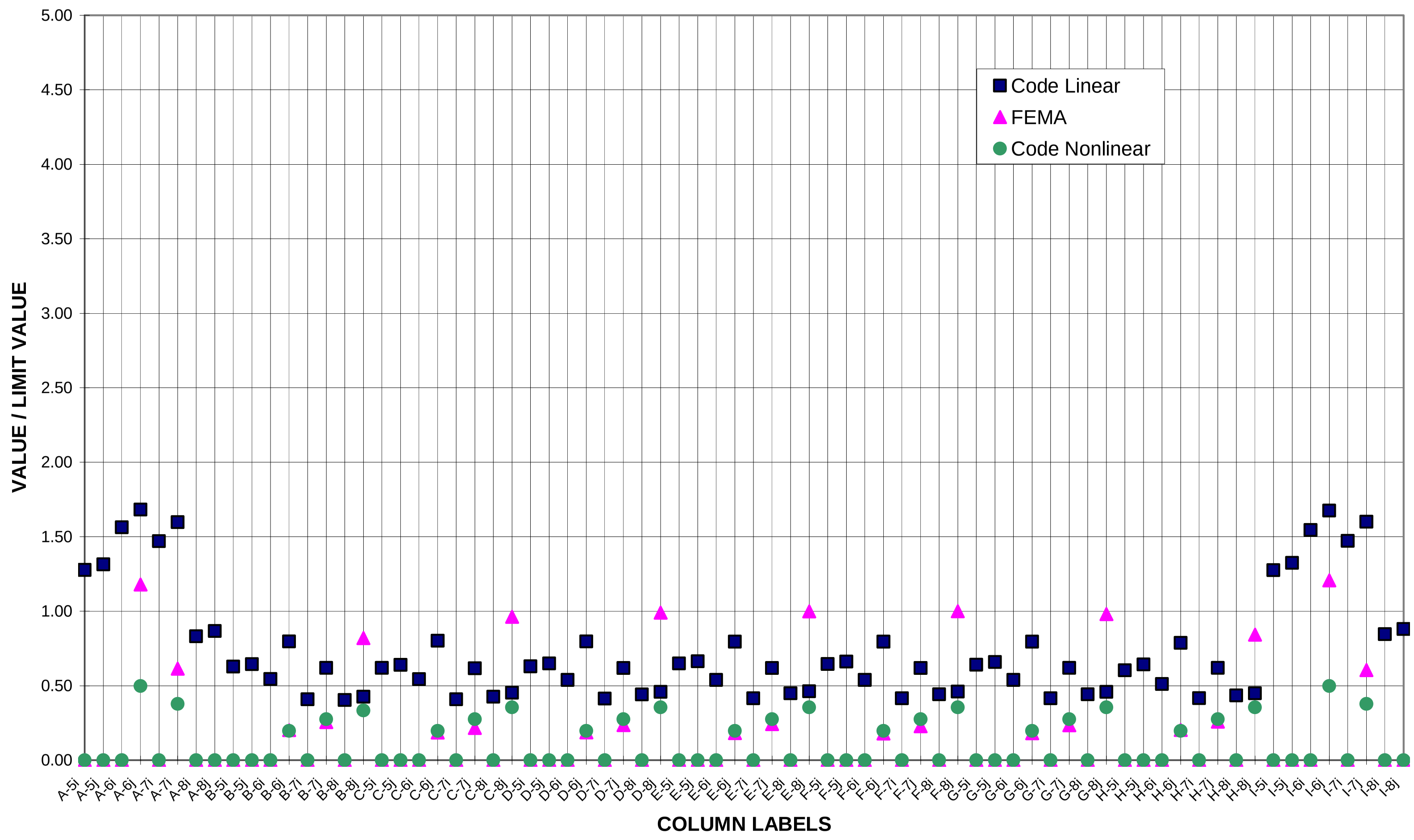


Figure B.11 Existing System Life Safety Performance Level 5th Storey x – Direction Columns Performance Levels

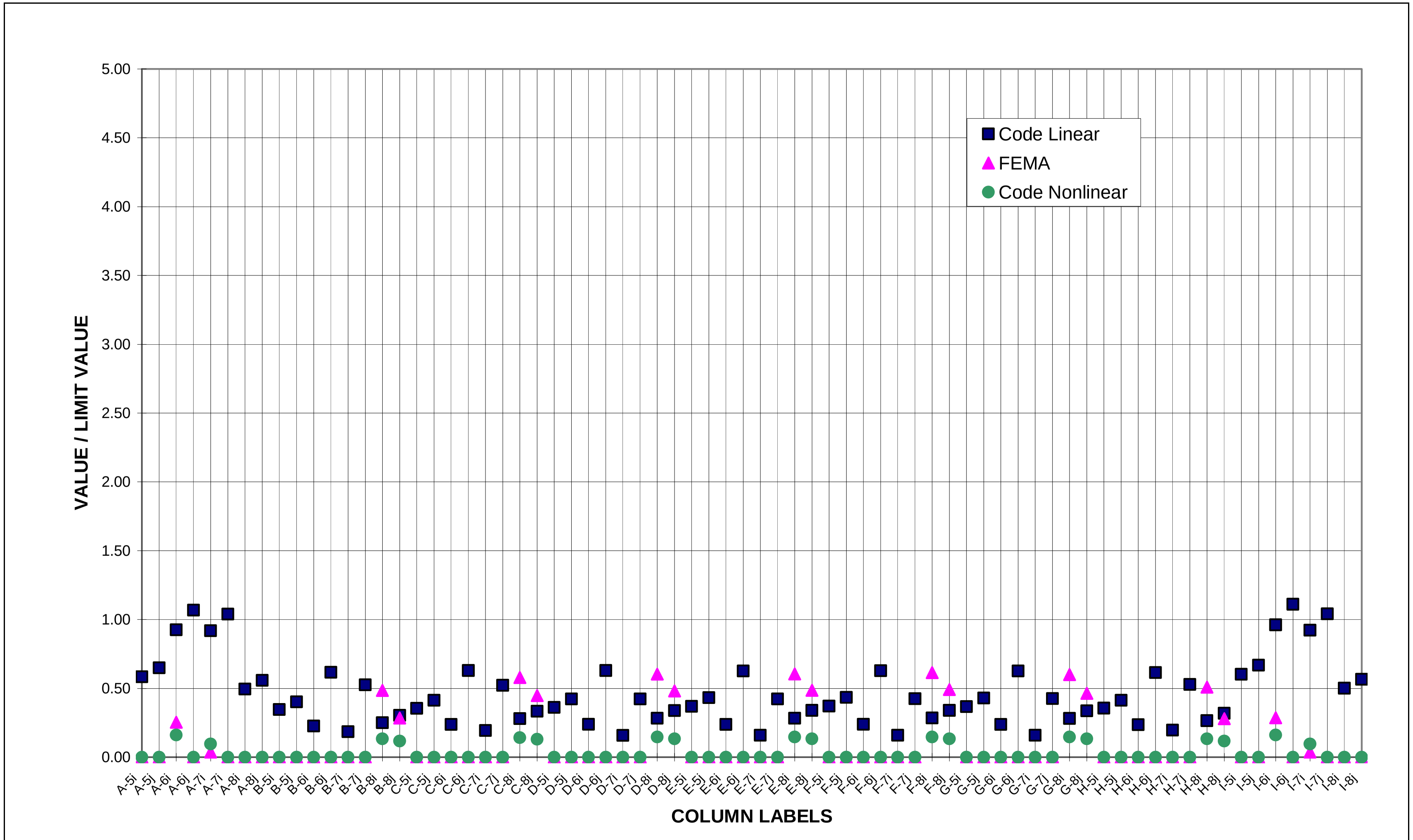


Figure B.12 Existing System Life Safety Performance Level 6th Storey x – Direction Columns Performance Levels

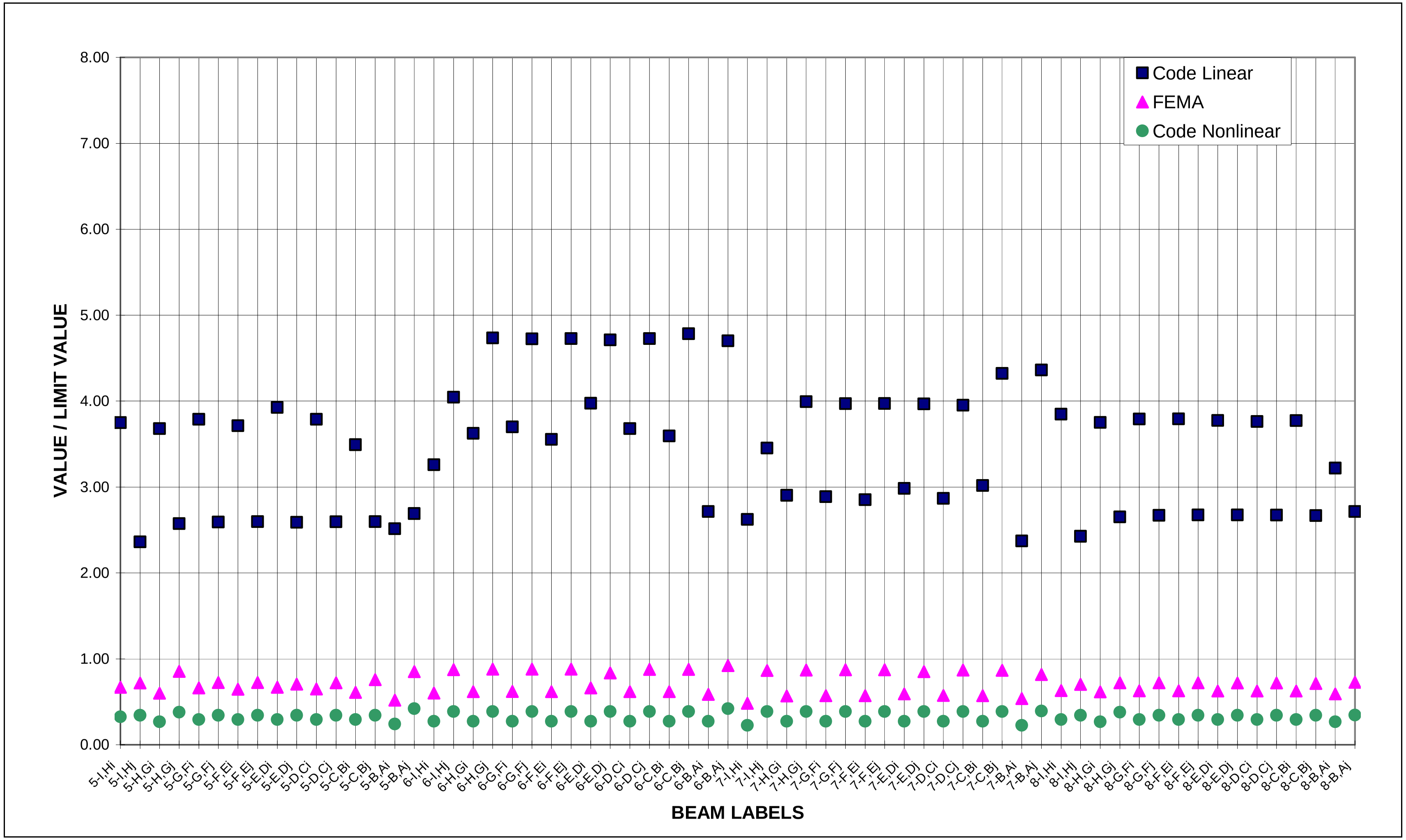


Figure B.13 Existing System Life Safety Performance Level 1st Storey y – Direction Beams Performance Levels

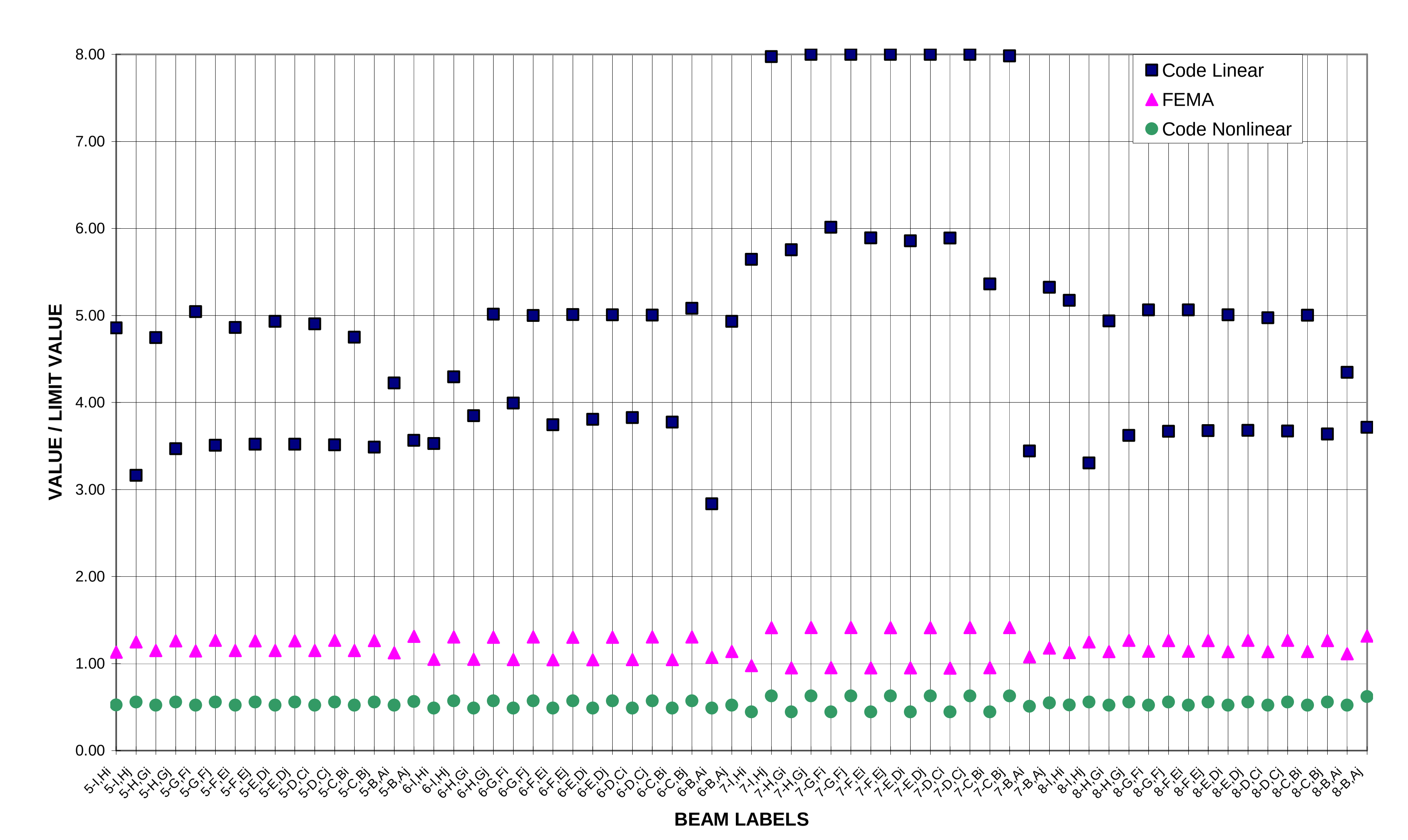


Figure B.14 Existing System Life Safety Performance Level 2nd Storey y – Direction Beams Performance Levels

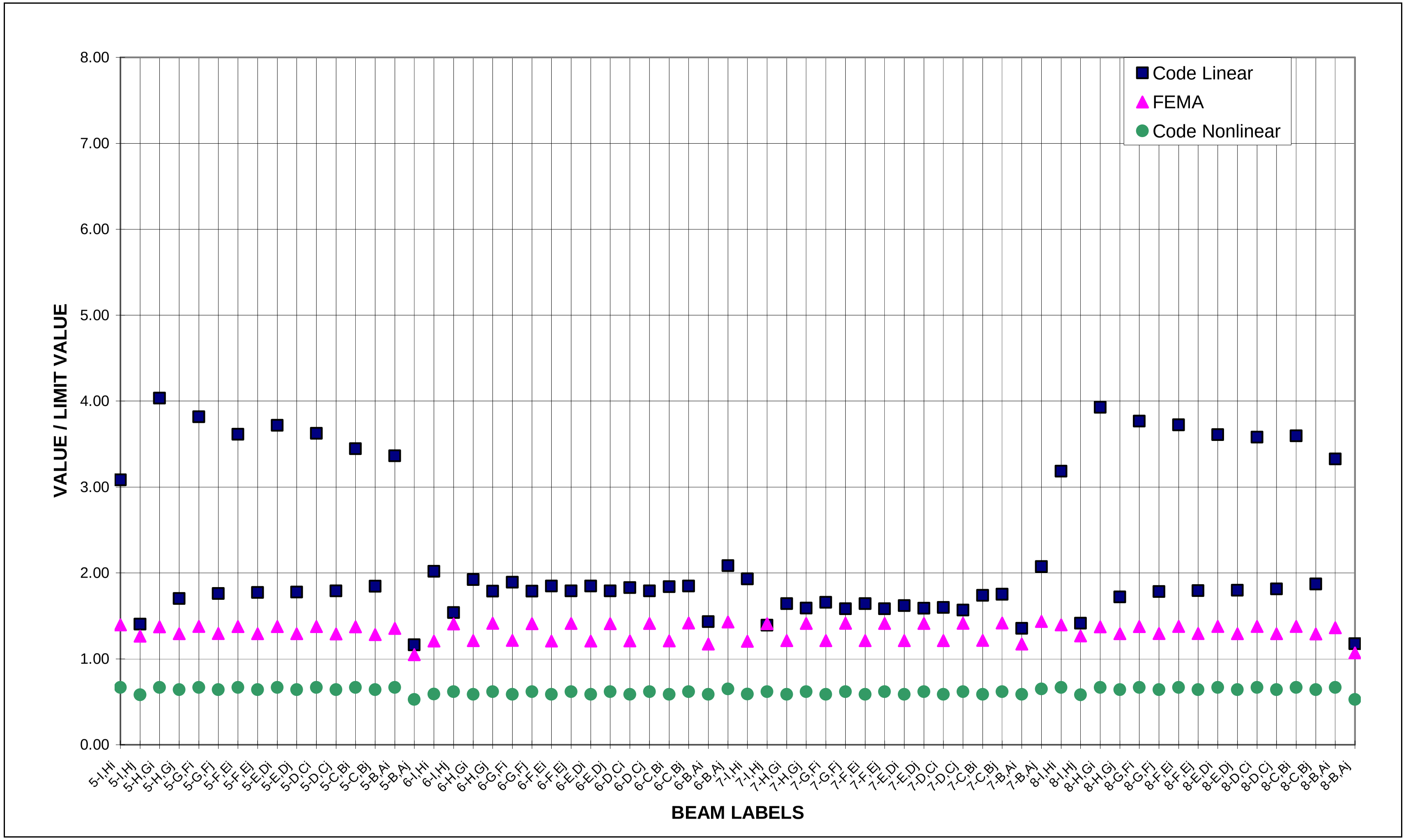


Figure B.15 Existing System Life Safety Performance Level 3rd Storey y – Direction Beams Performance Levels

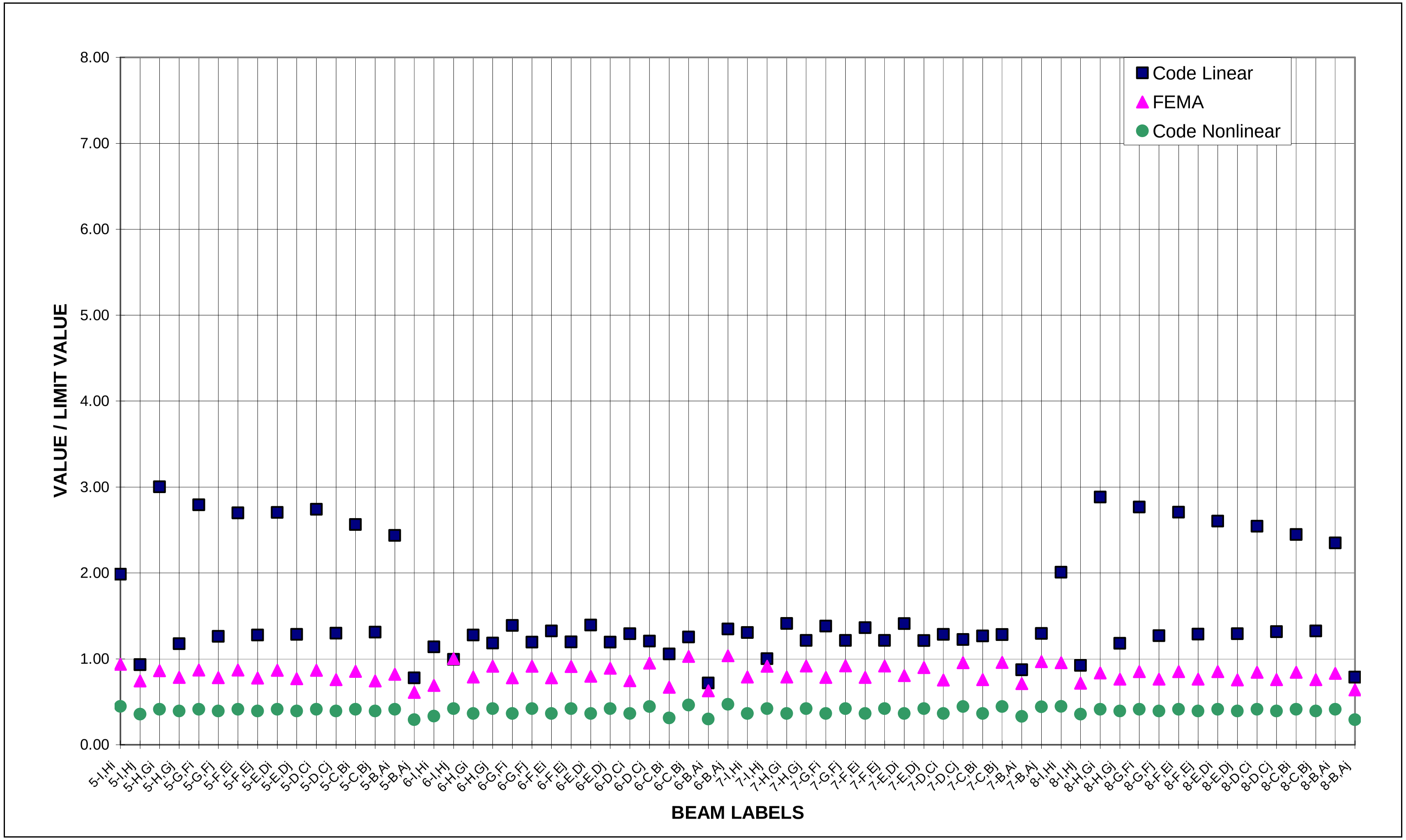


Figure B.16 Existing System Life Safety Performance Level 4th Storey y – Direction Beams Performance Levels

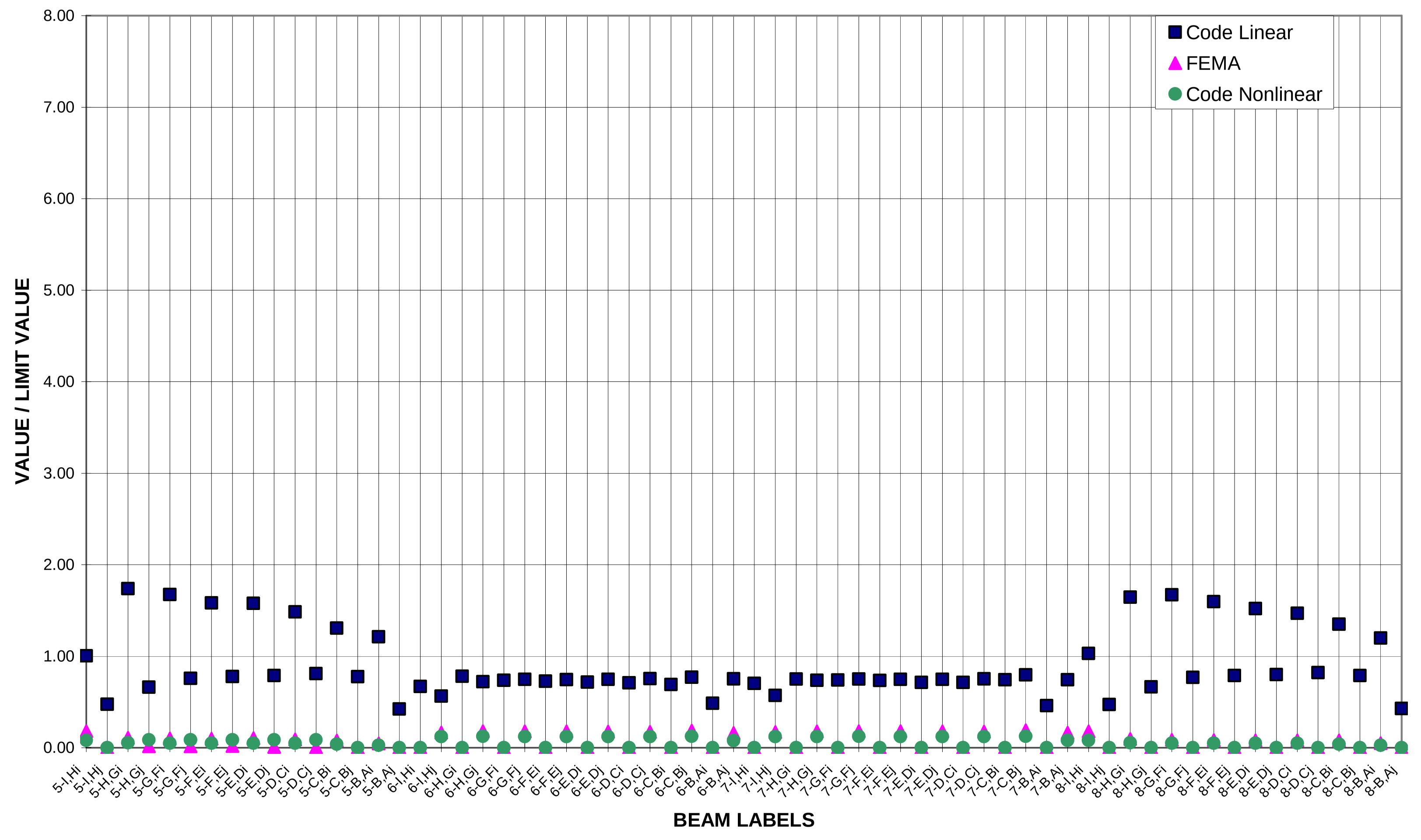


Figure B.17 Existing System Life Safety Performance Level 5th Storey y – Direction Beams Performance Levels

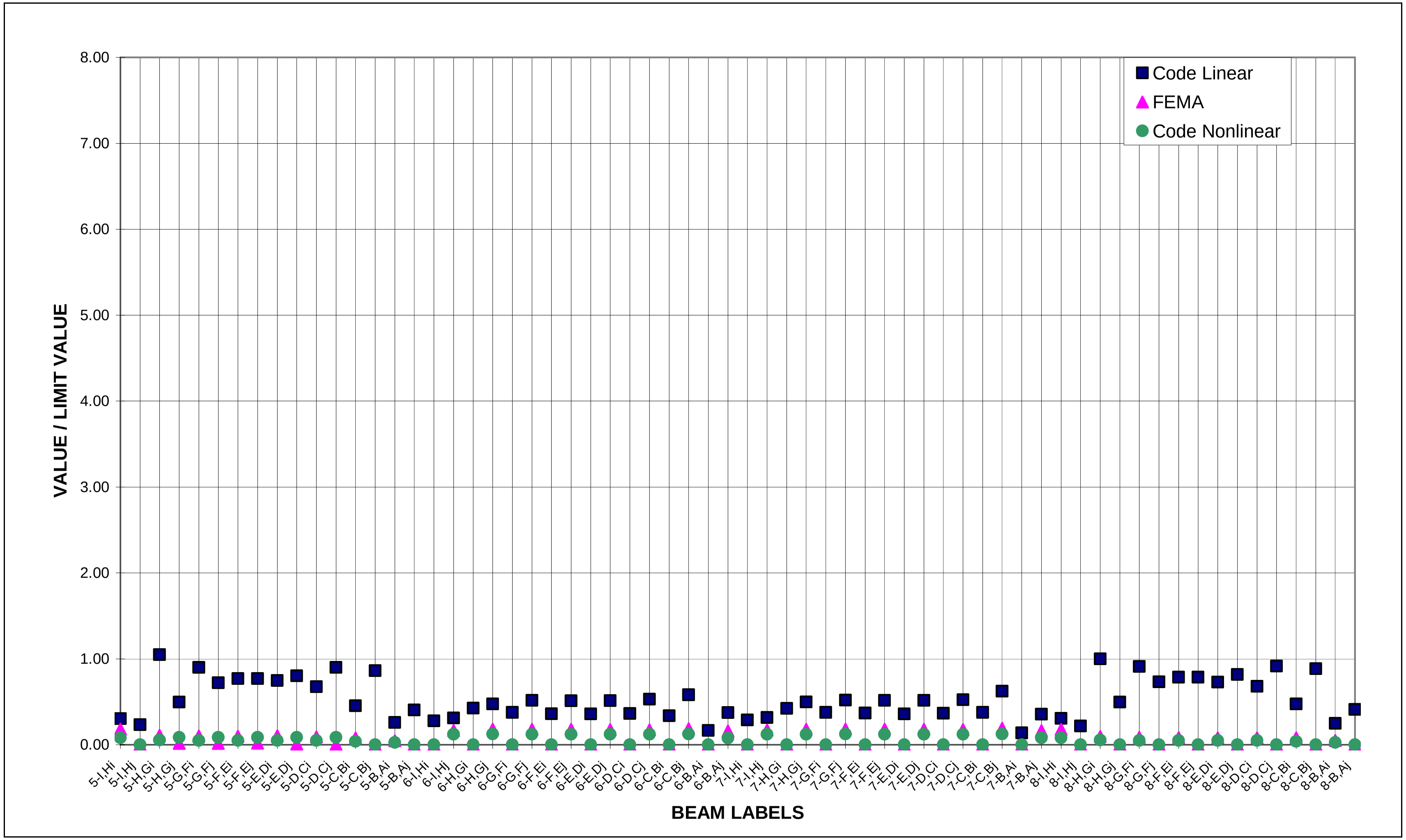


Figure B.18 Existing System Life Safety Performance Level 6th Storey y – Direction Beams Performance Levels

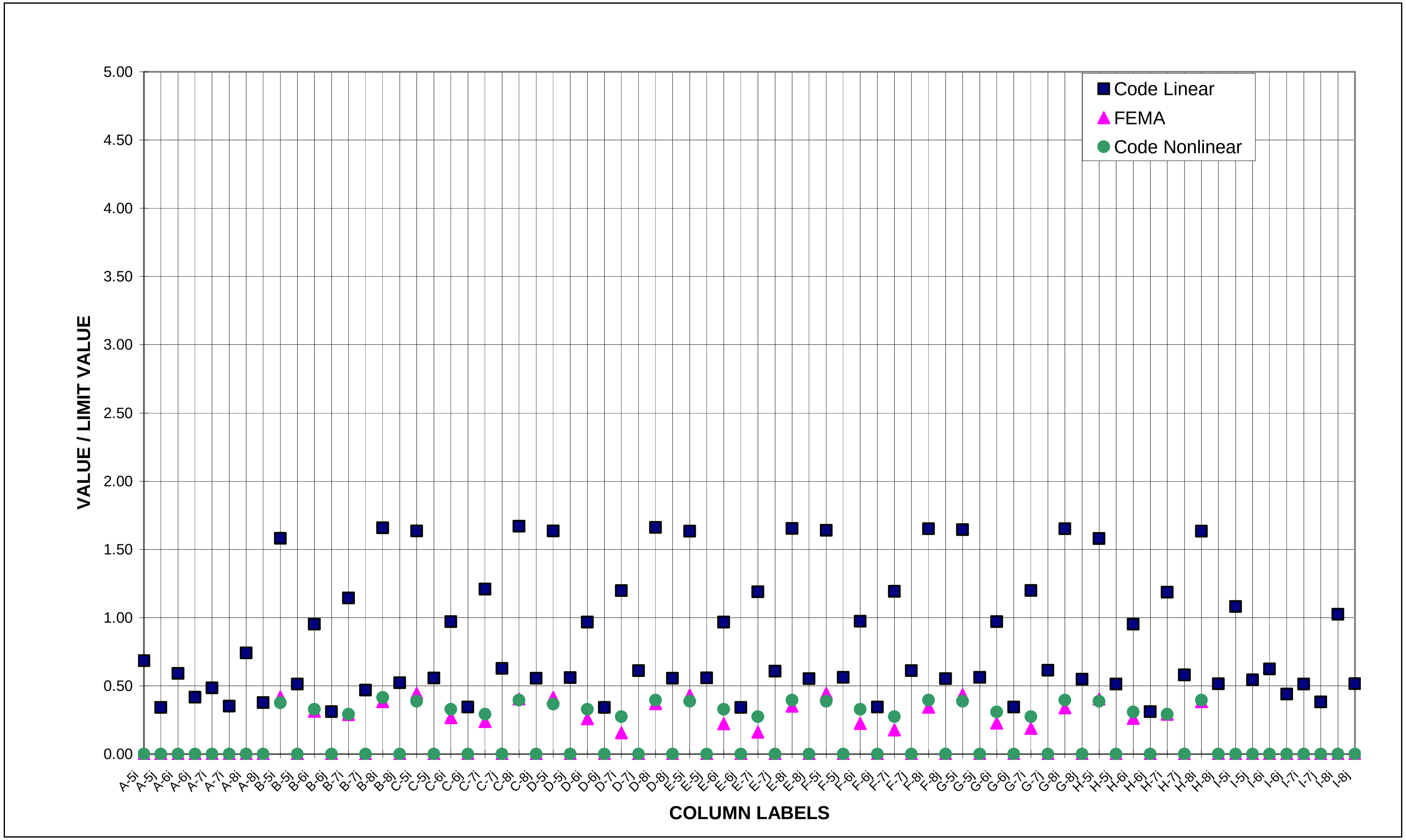


Figure B.19 Existing System Life Safety Performance Level 1st Storey y – Direction Columns Performance Levels

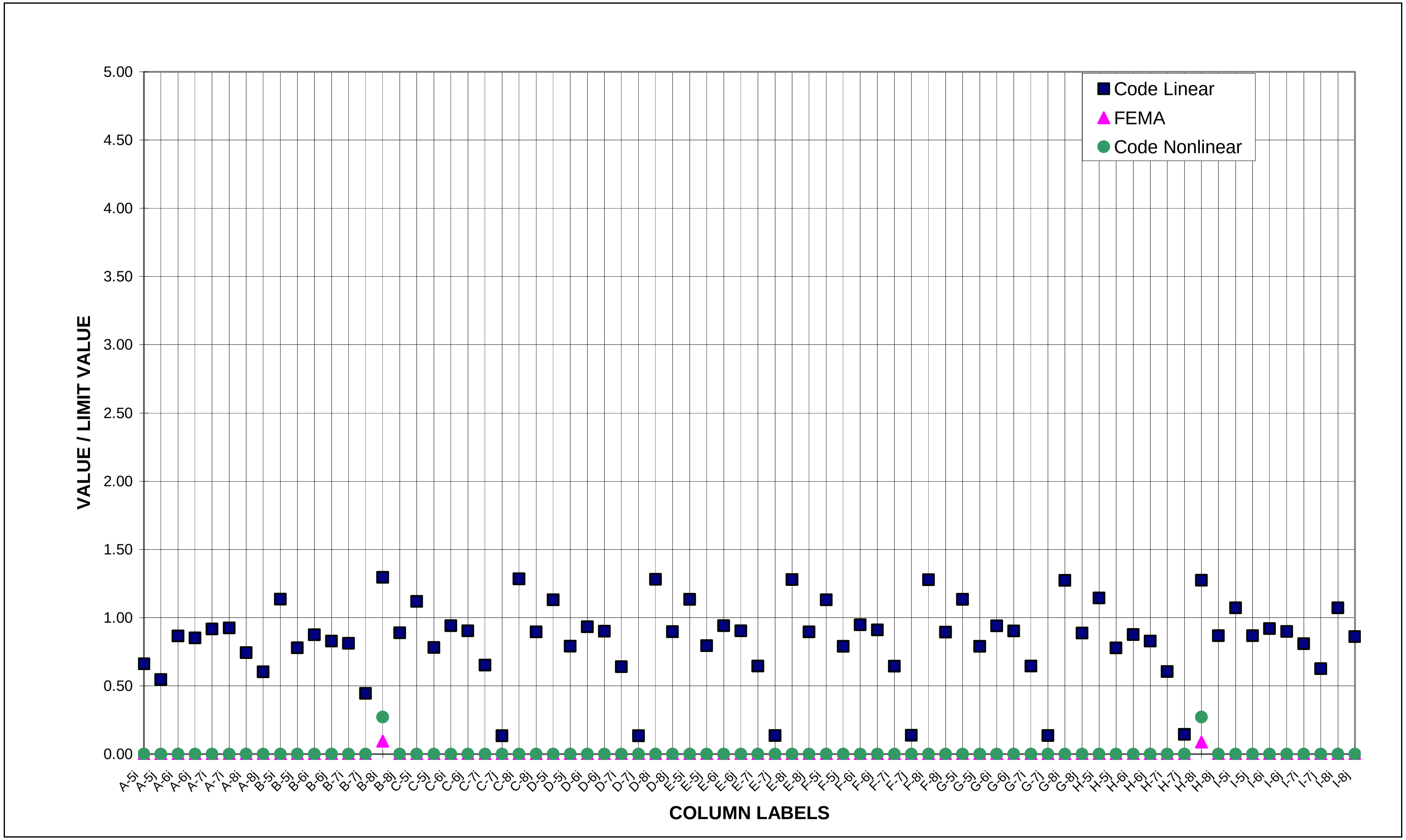


Figure B.20 Existing System Life Safety Performance Level 2nd Storey y – Direction Columns Performance Levels

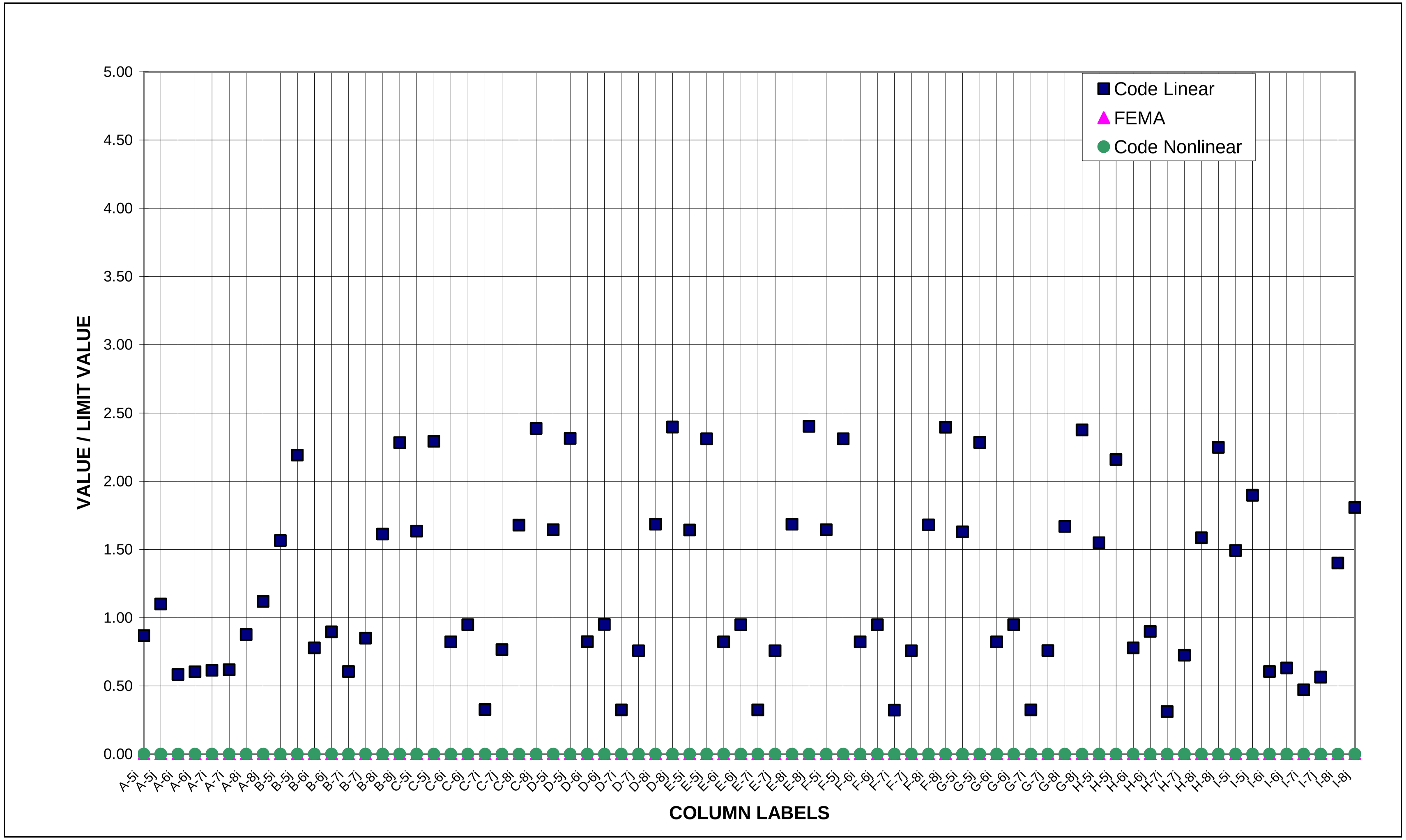


Figure B.21 Existing System Life Safety Performance Level 3rd Storey y – Direction Columns Performance Levels

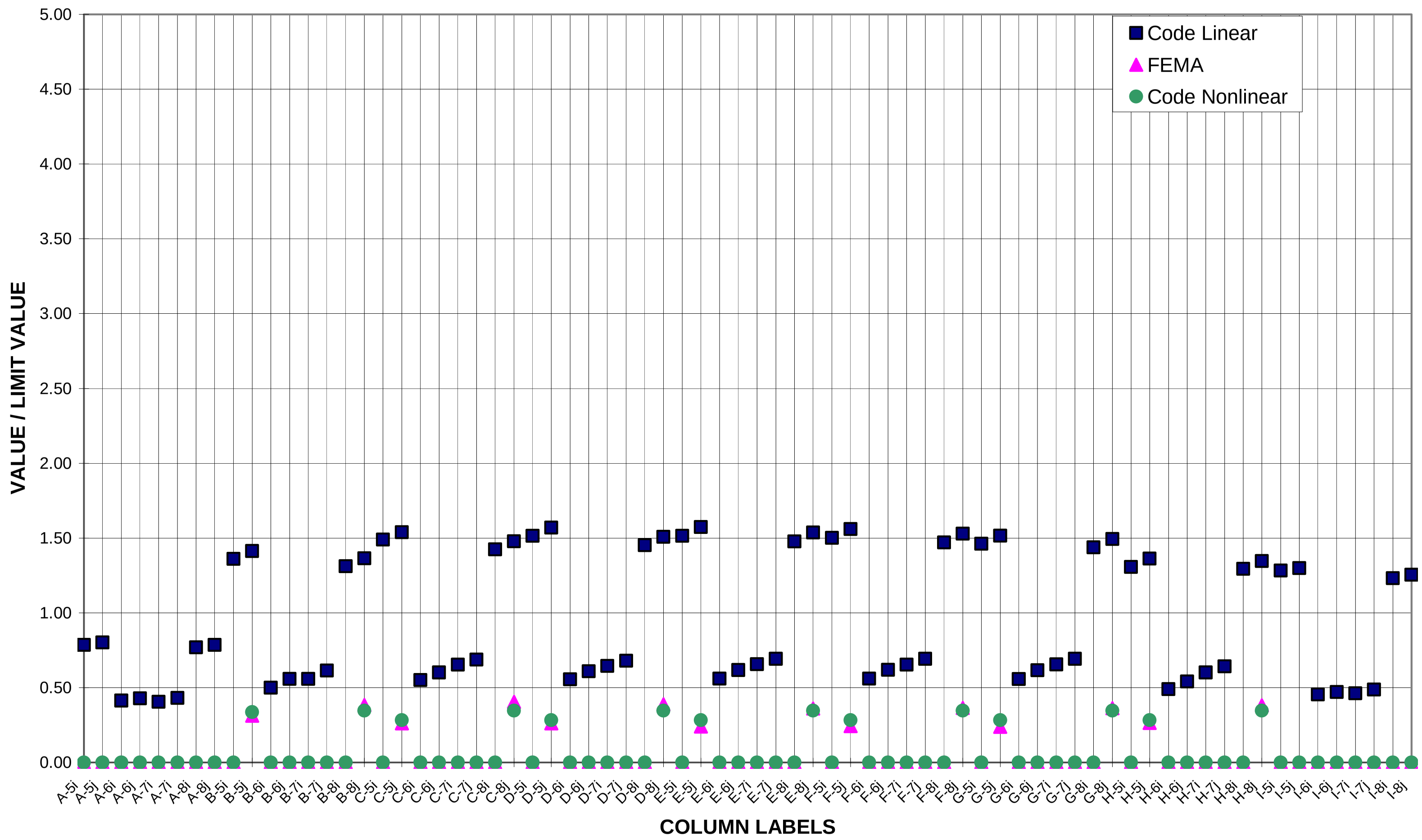


Figure B.22 Existing System Life Safety Performance Level 4th Storey y – Direction Columns Performance Levels

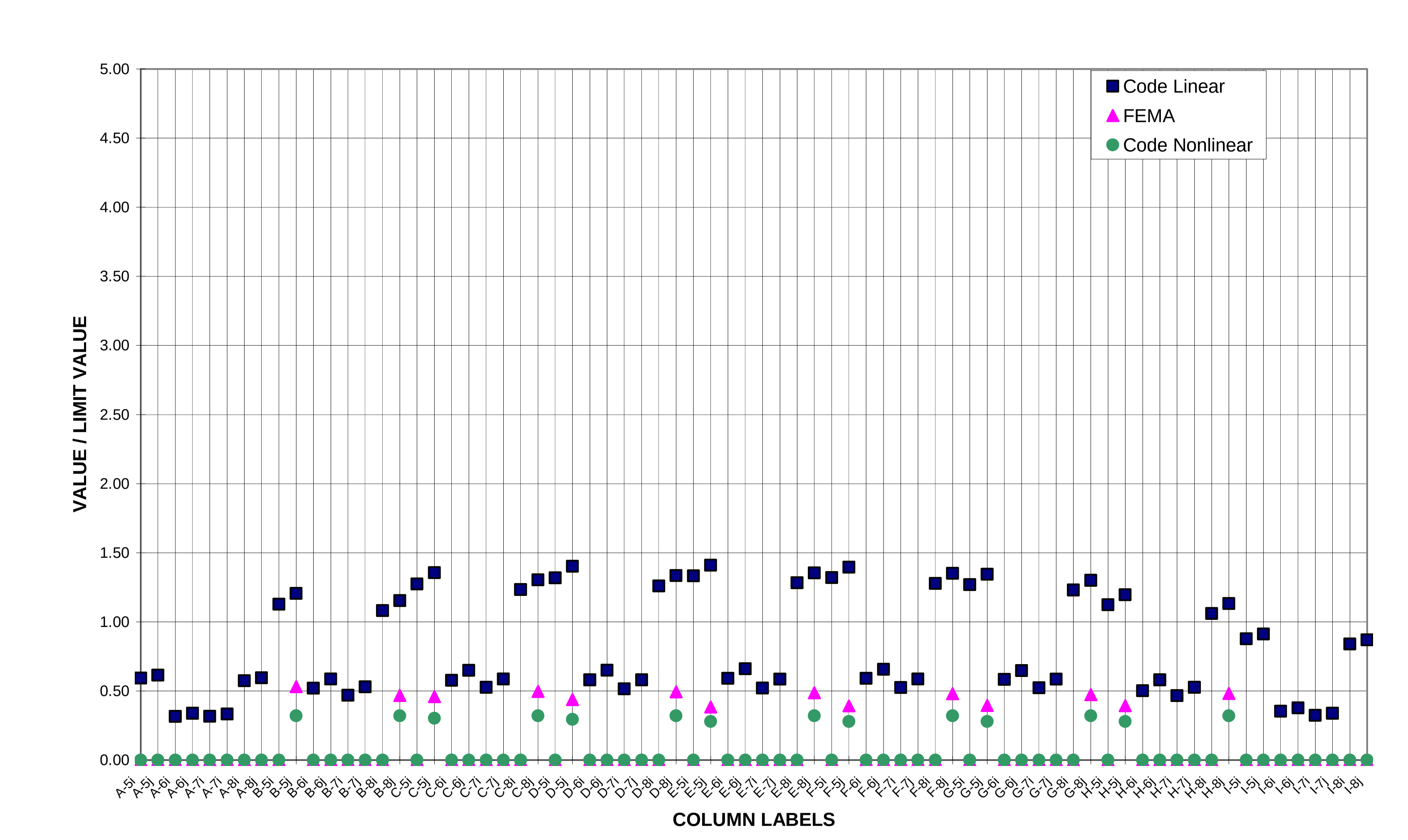


Figure B.23 Existing System Life Safety Performance Level 5th Storey y – Direction Columns Performance Levels

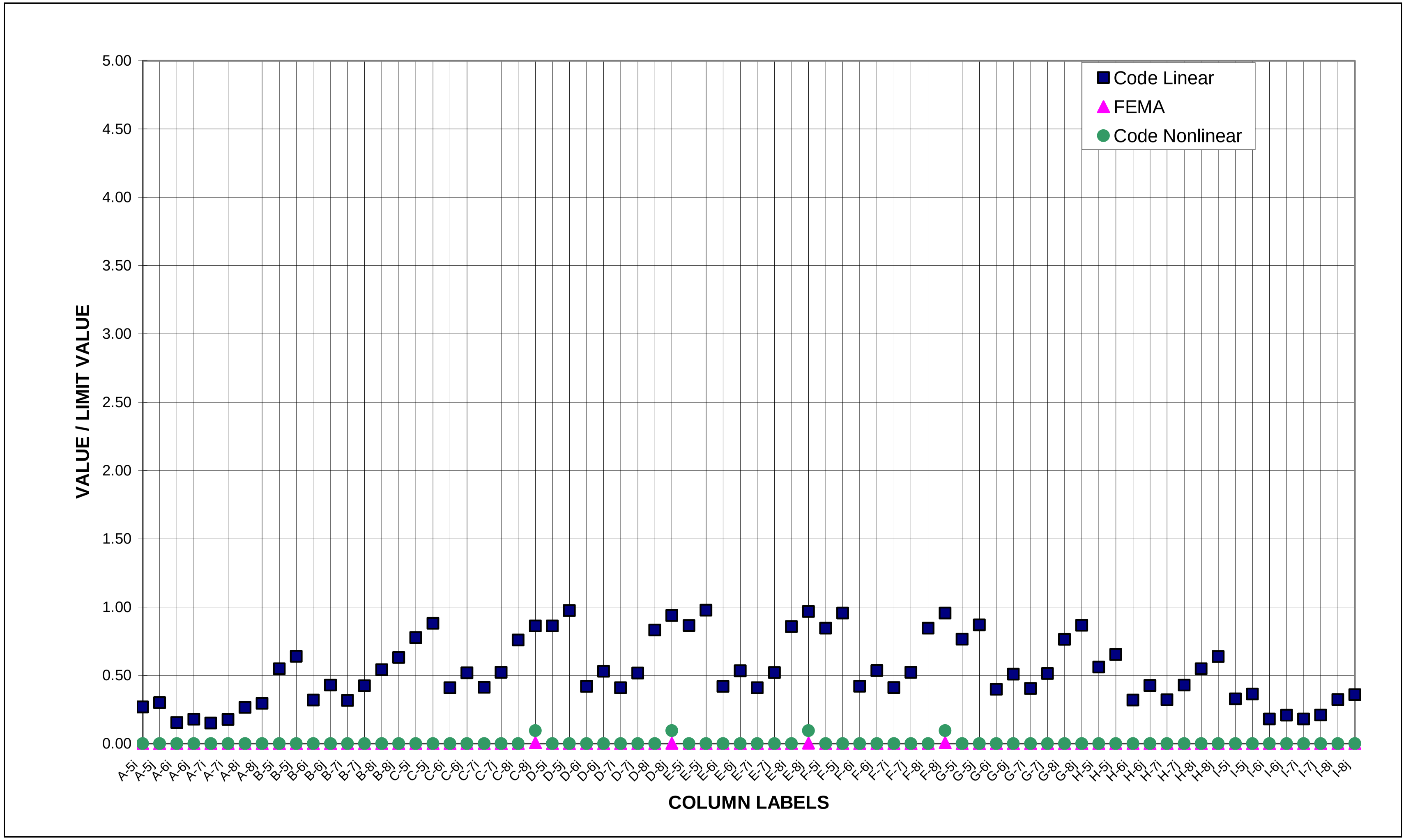


Figure B.24 Existing System Life Safety Performance Level 6th Storey y – Direction Columns Performance Levels

APPENDIX C

RETROFITTED SYSTEM, LIFE SAFETY PERFORMANCE LEVEL, GRAPHICAL REPRESENTATION OF THE COMPERATIVE ASSESSMENT RESULTS

In this appendix, retrofitted system comparative assessment results of all frame elements for life safety performance level are presented in graphical forms. Comparative results summary for both immediate occupancy and life safety performance levels are also given in Chapter VI.

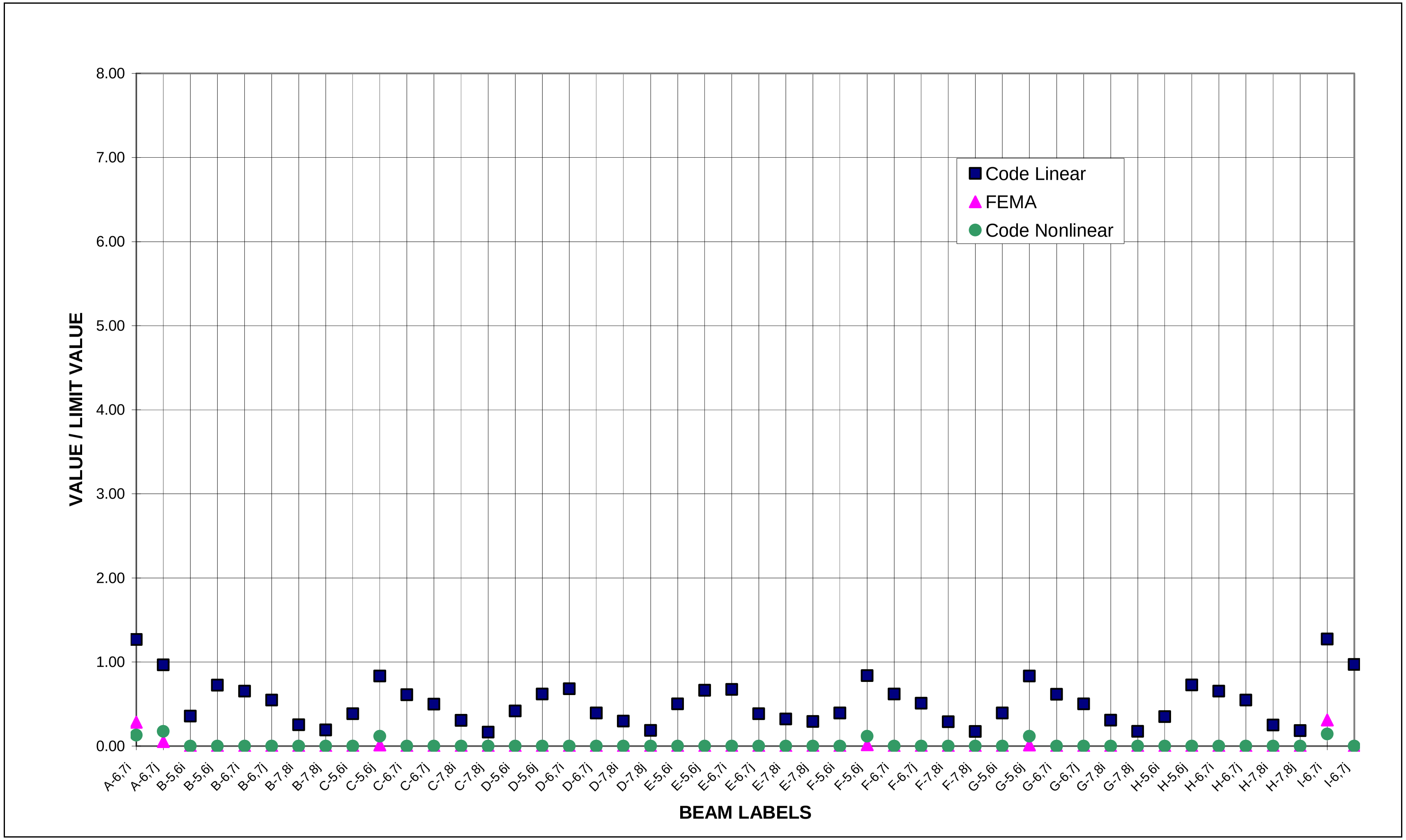


Figure C.1 Retrofitted System Life Safety Performance Level 1st Storey x – Direction Beams Performance Levels

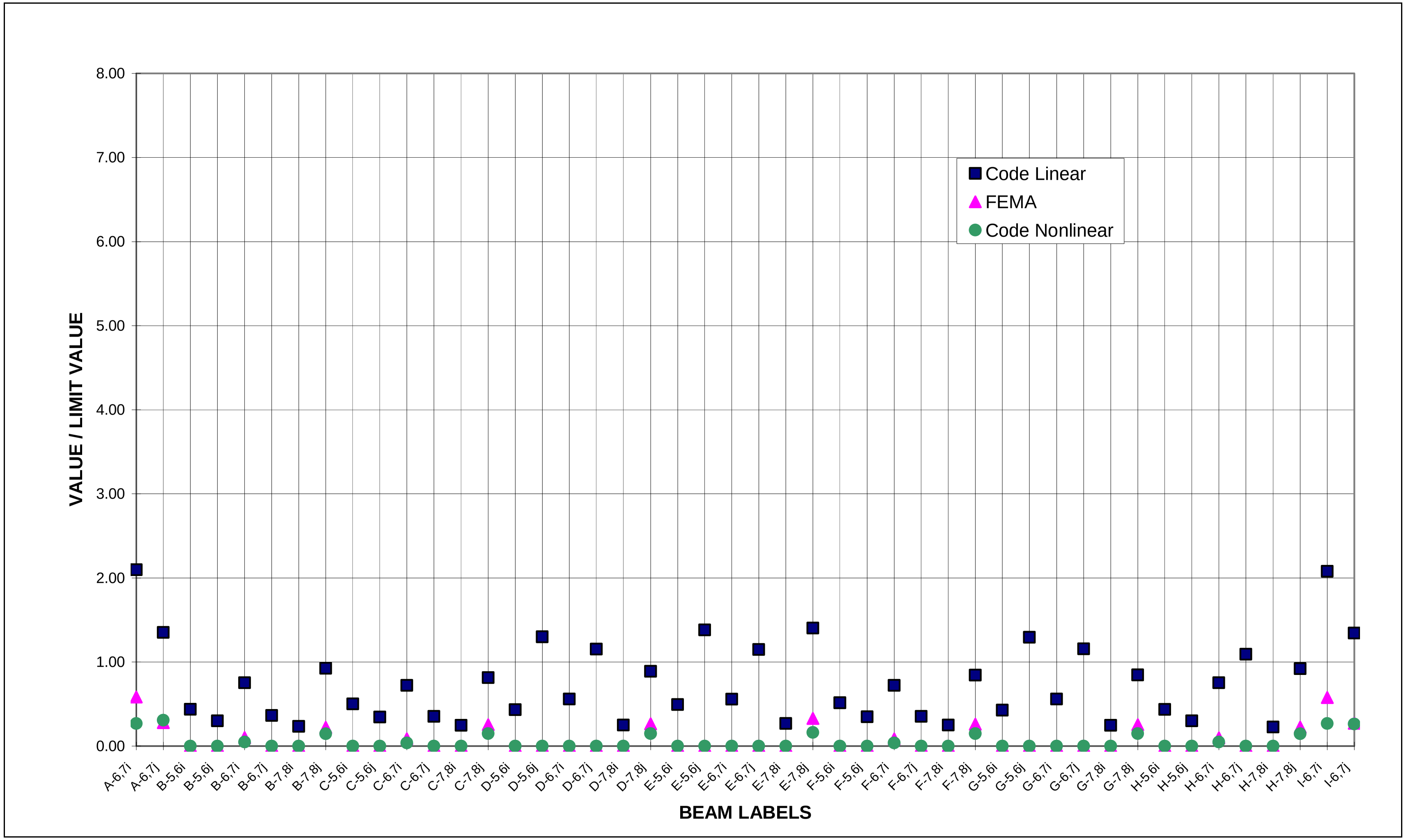


Figure C.2 Retrofitted System Life Safety Performance Level 2nd Storey x – Direction Beams Performance Levels

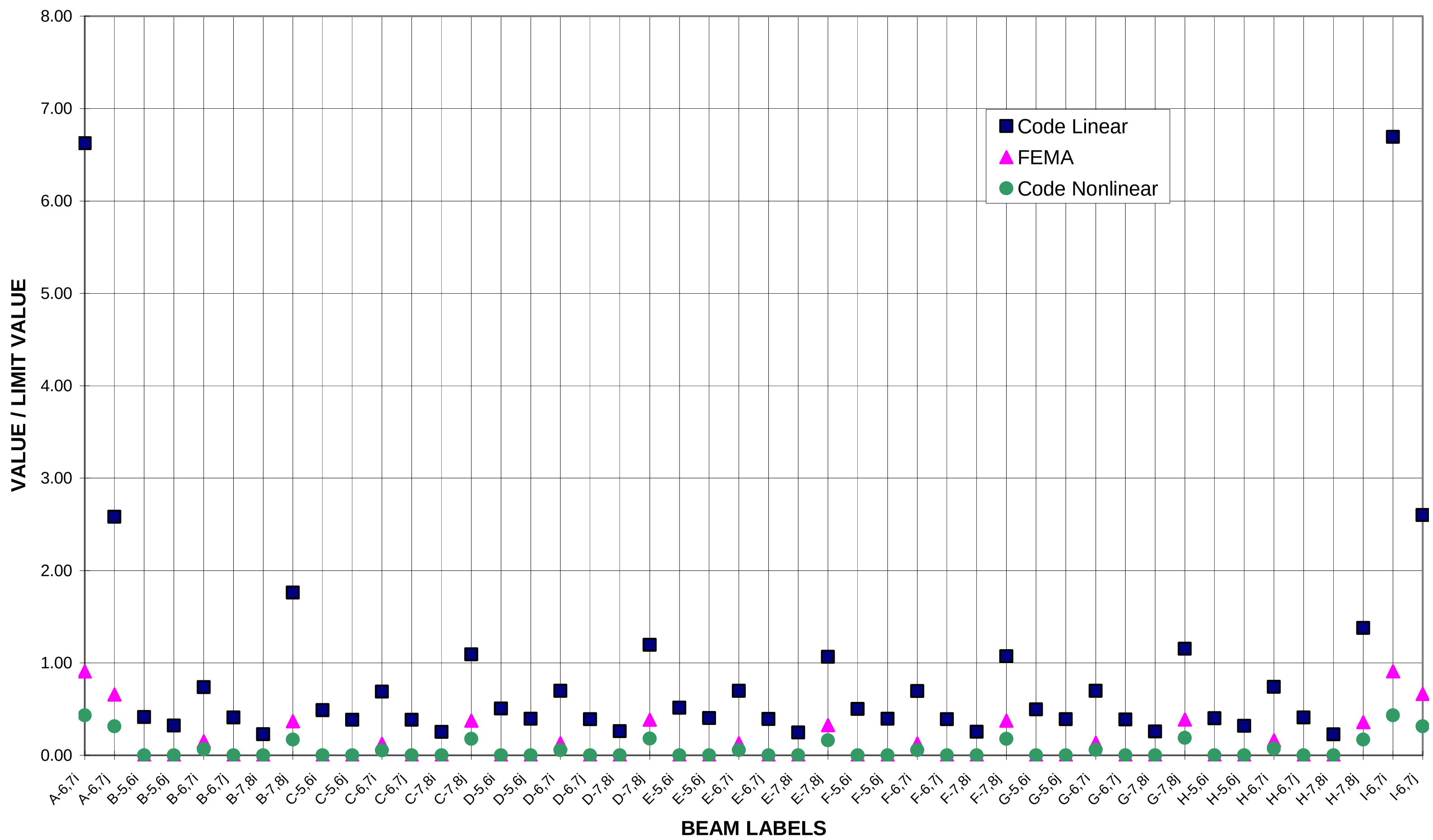


Figure C.3 Retrofitted System Life Safety Performance Level 3rd Storey x – Direction Beams Performance Levels

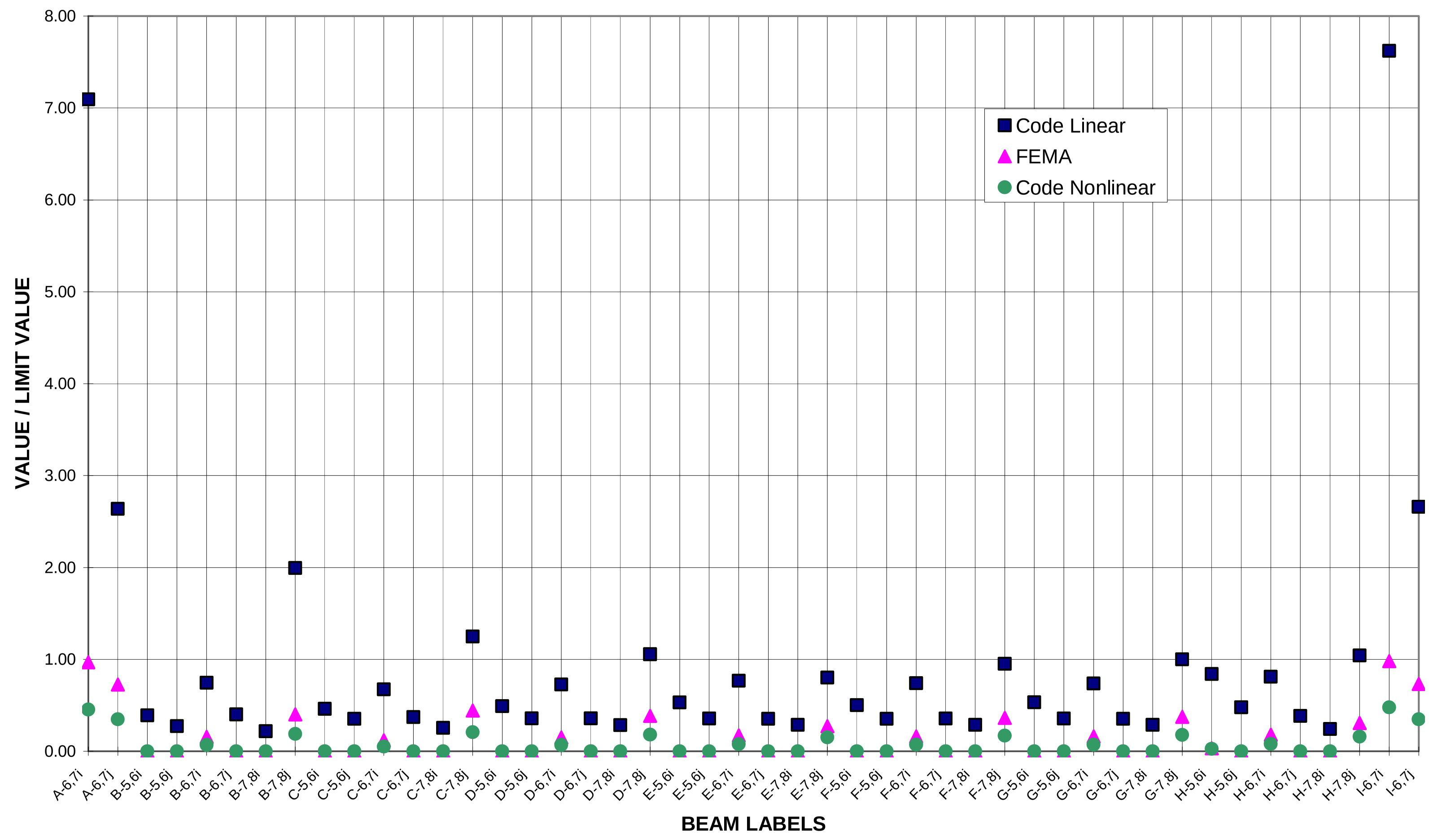


Figure C.4 Retrofitted System Life Safety Performance Level 4rd Storey x – Direction Beams Performance Levels

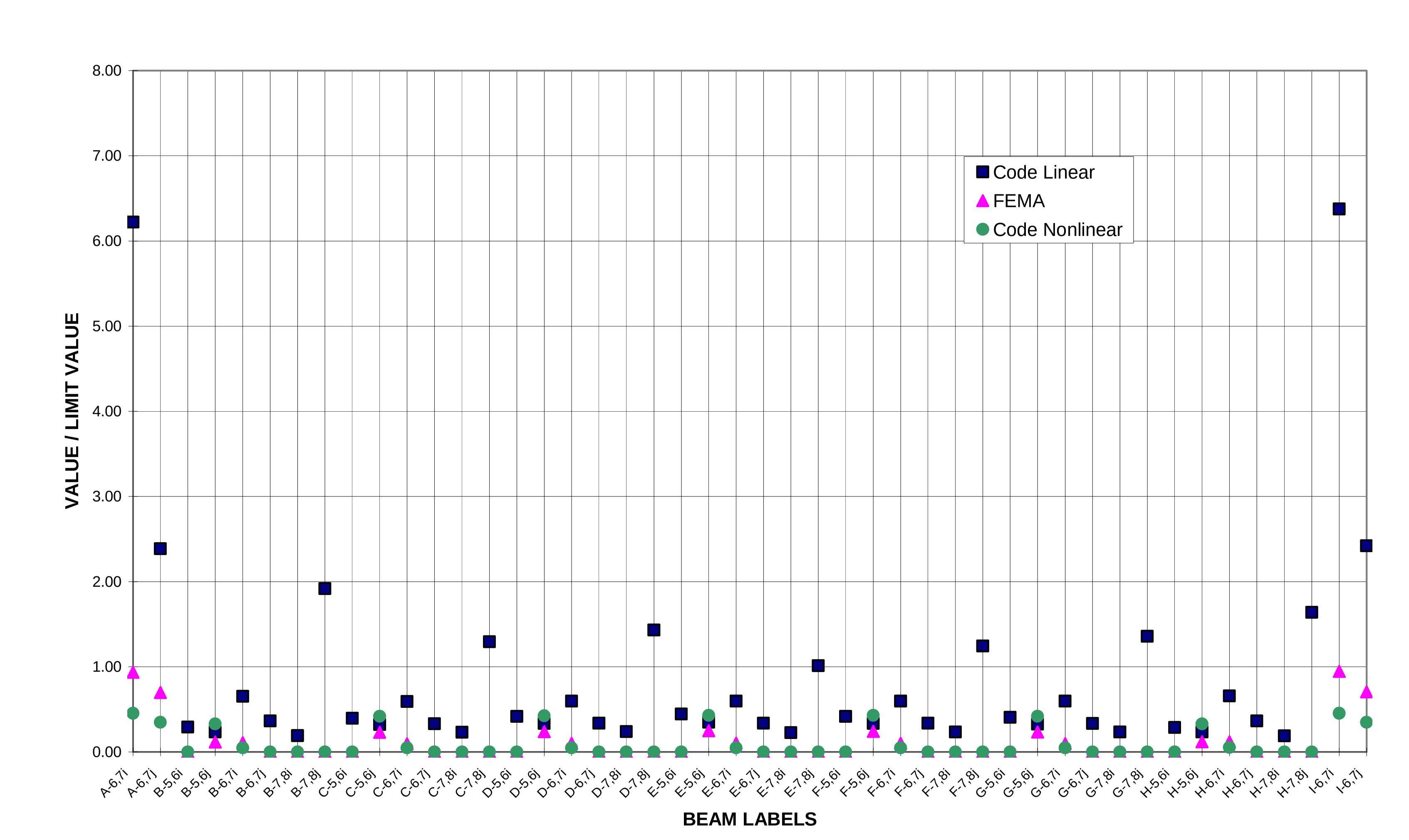


Figure C.5 Retrofitted System Life Safety Performance Level 5rd Storey x – Direction Beams Performance Levels

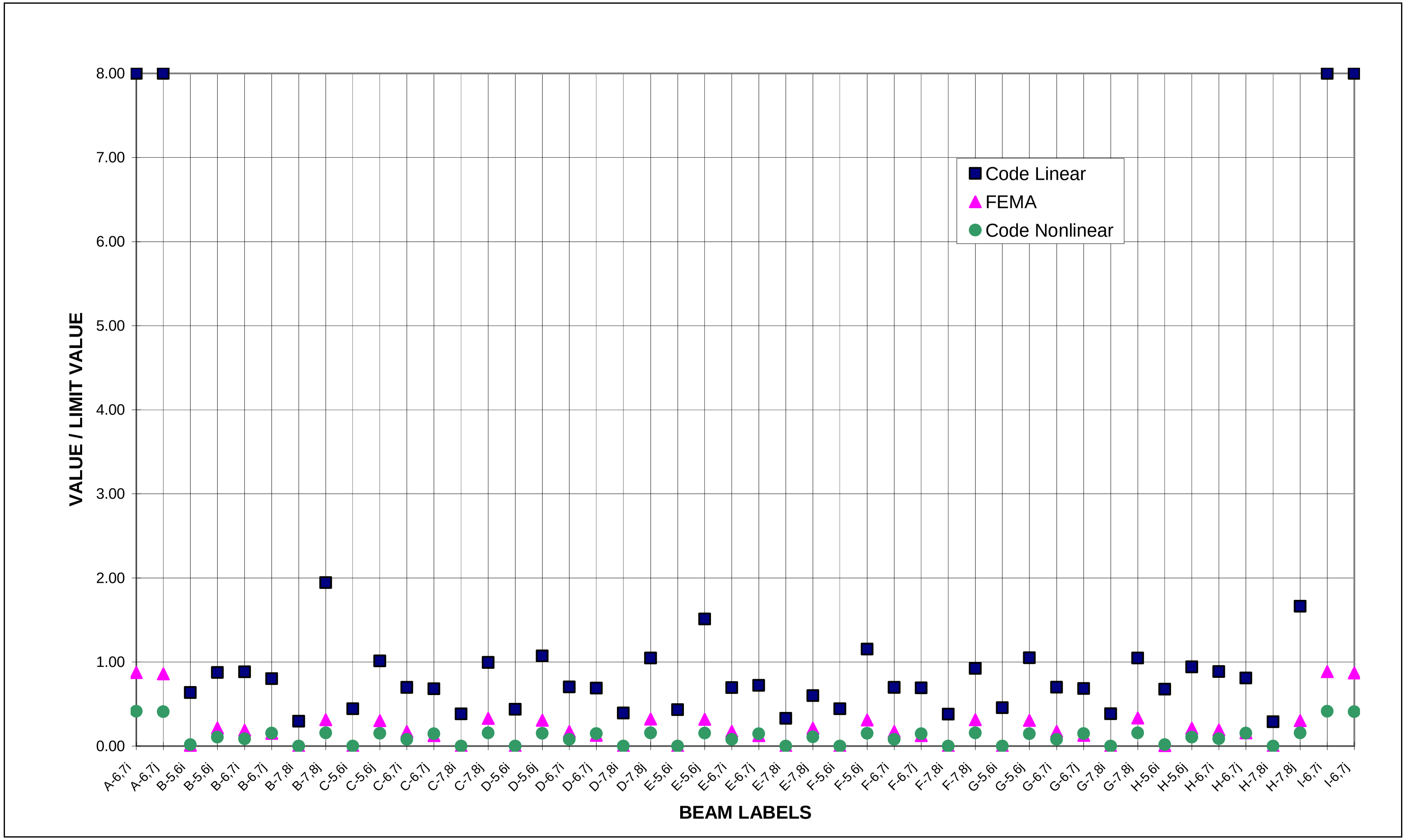


Figure C.6 Retrofitted System Life Safety Performance Level 6rd Storey x – Direction Beams Performance Levels

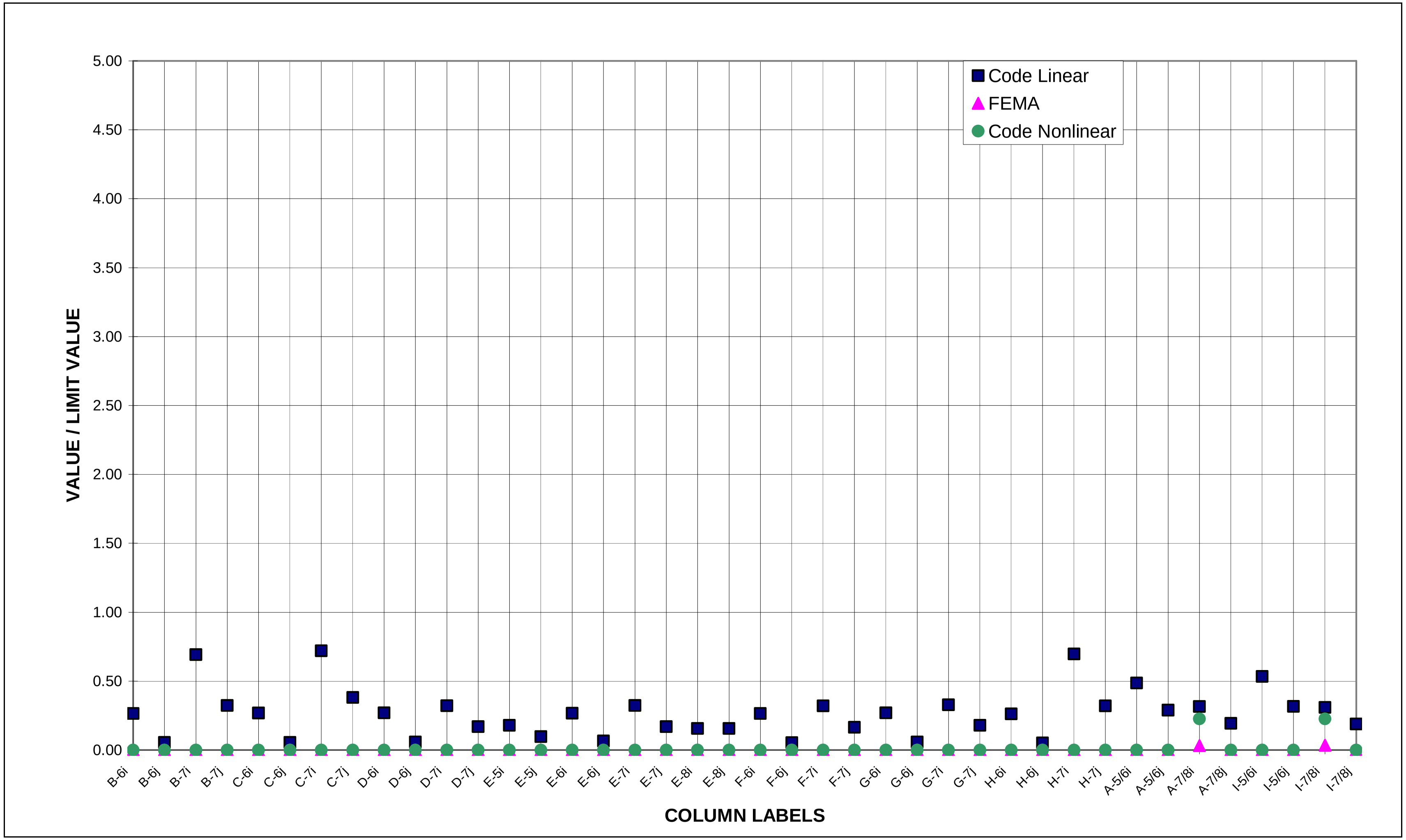


Figure C.7 Retrofitted System Life Safety Performance Level 1st Storey x – Direction Columns Performance Levels

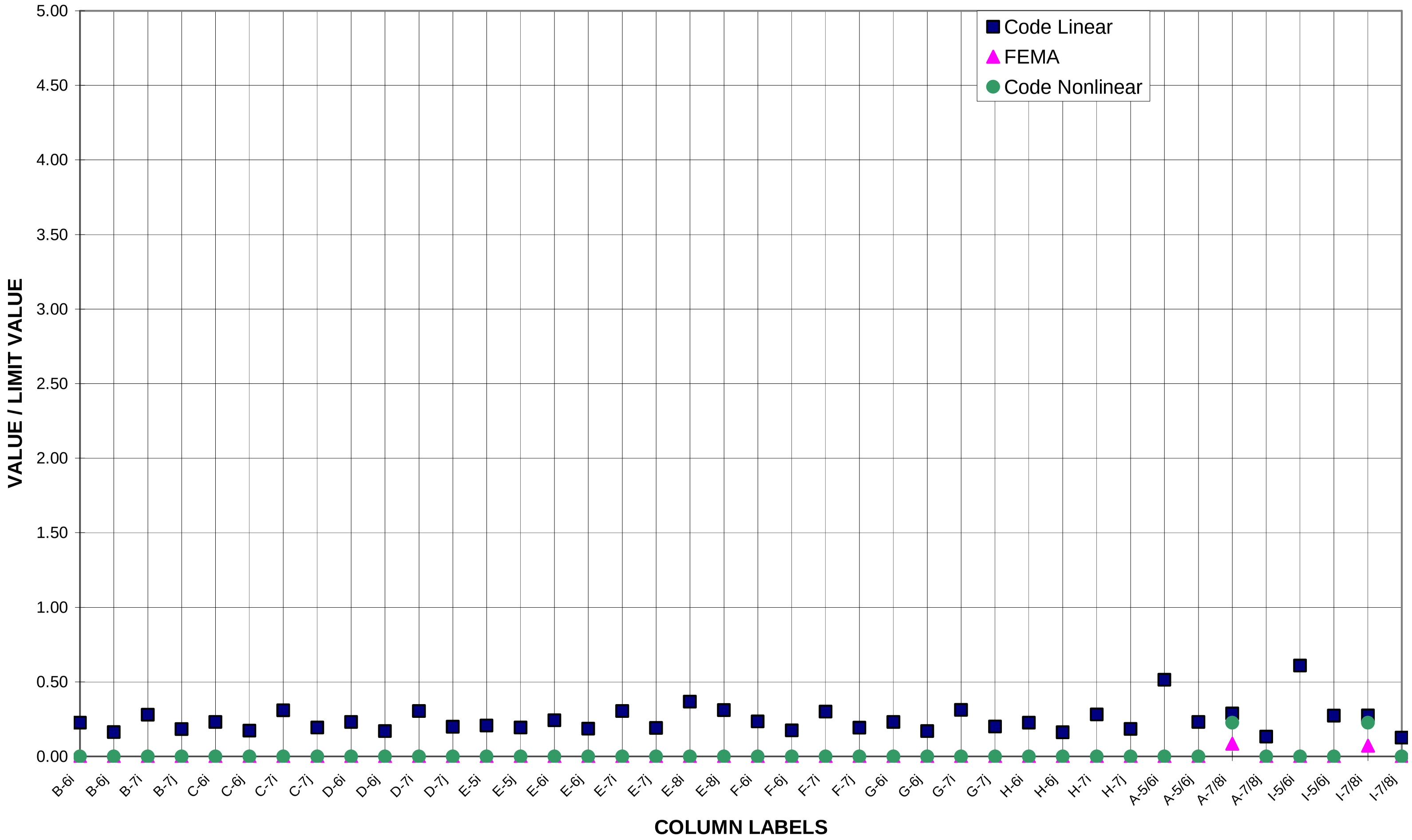


Figure C.8 Retrofitted System Life Safety Performance Level 2nd Storey x – Direction Columns Performance Levels

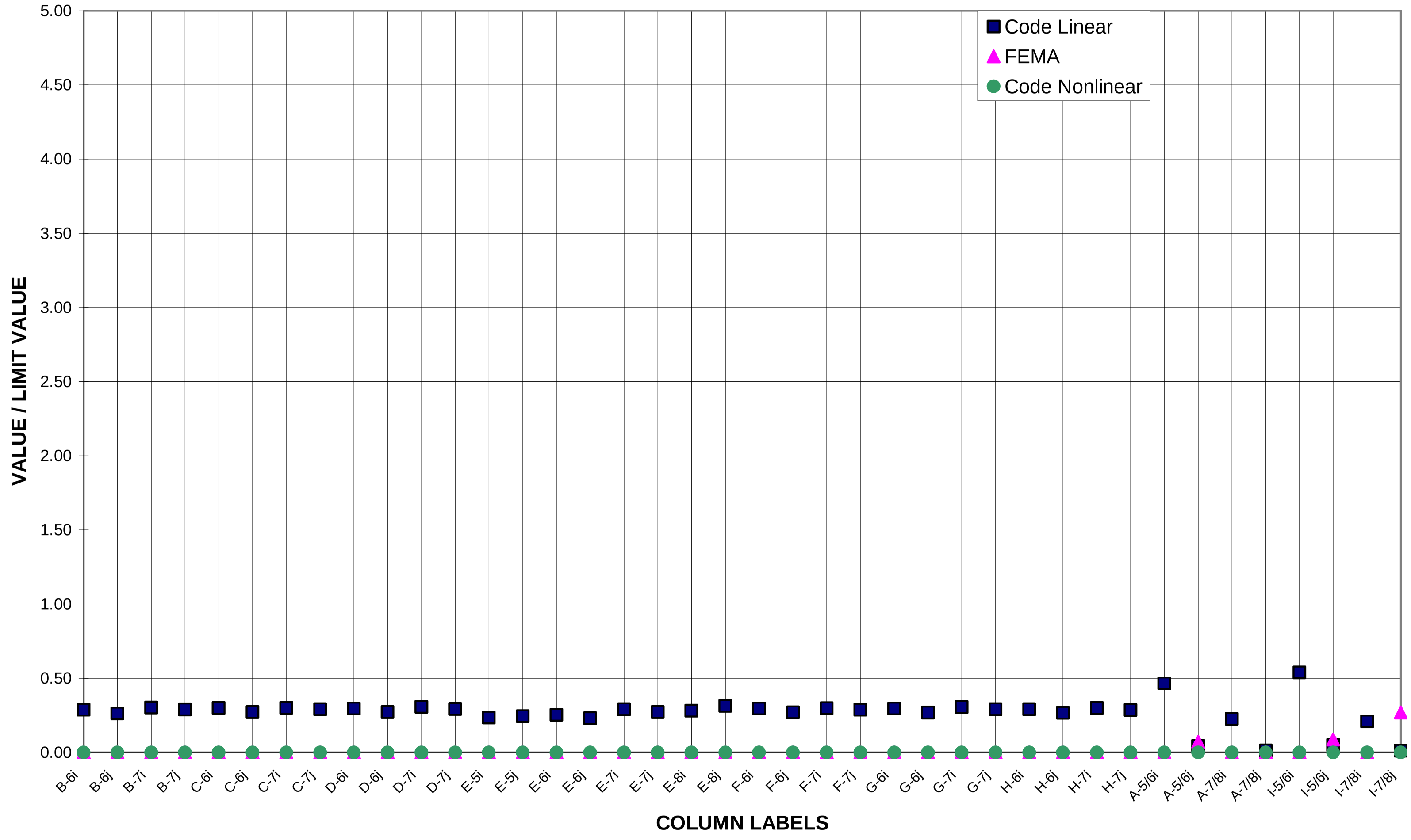


Figure C.9 Retrofitted System Life Safety Performance Level 3rd Storey x – Direction Columns Performance Levels

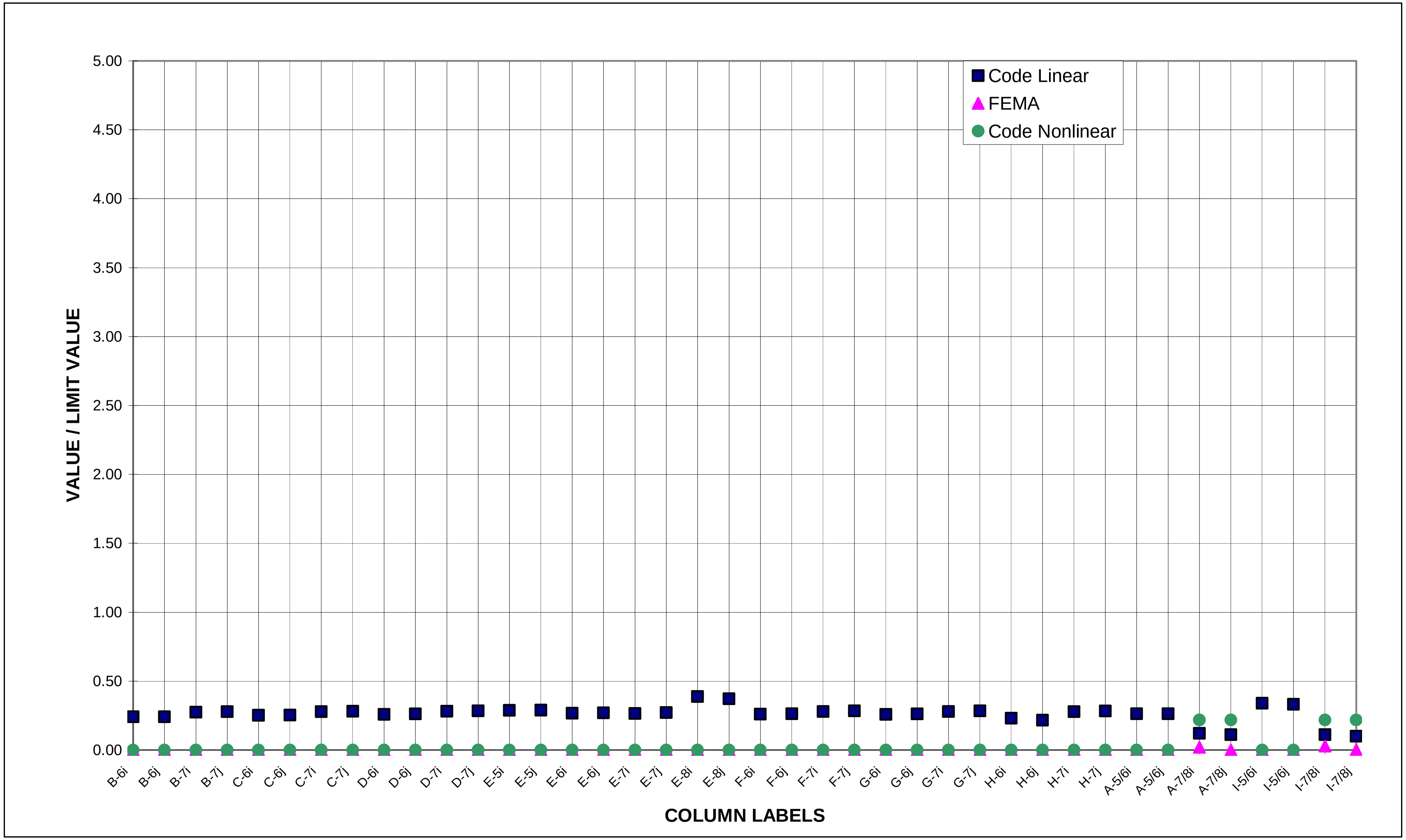


Figure C.10 Retrofitted System Life Safety Performance Level 4th Storey x – Direction Columns Performance Levels

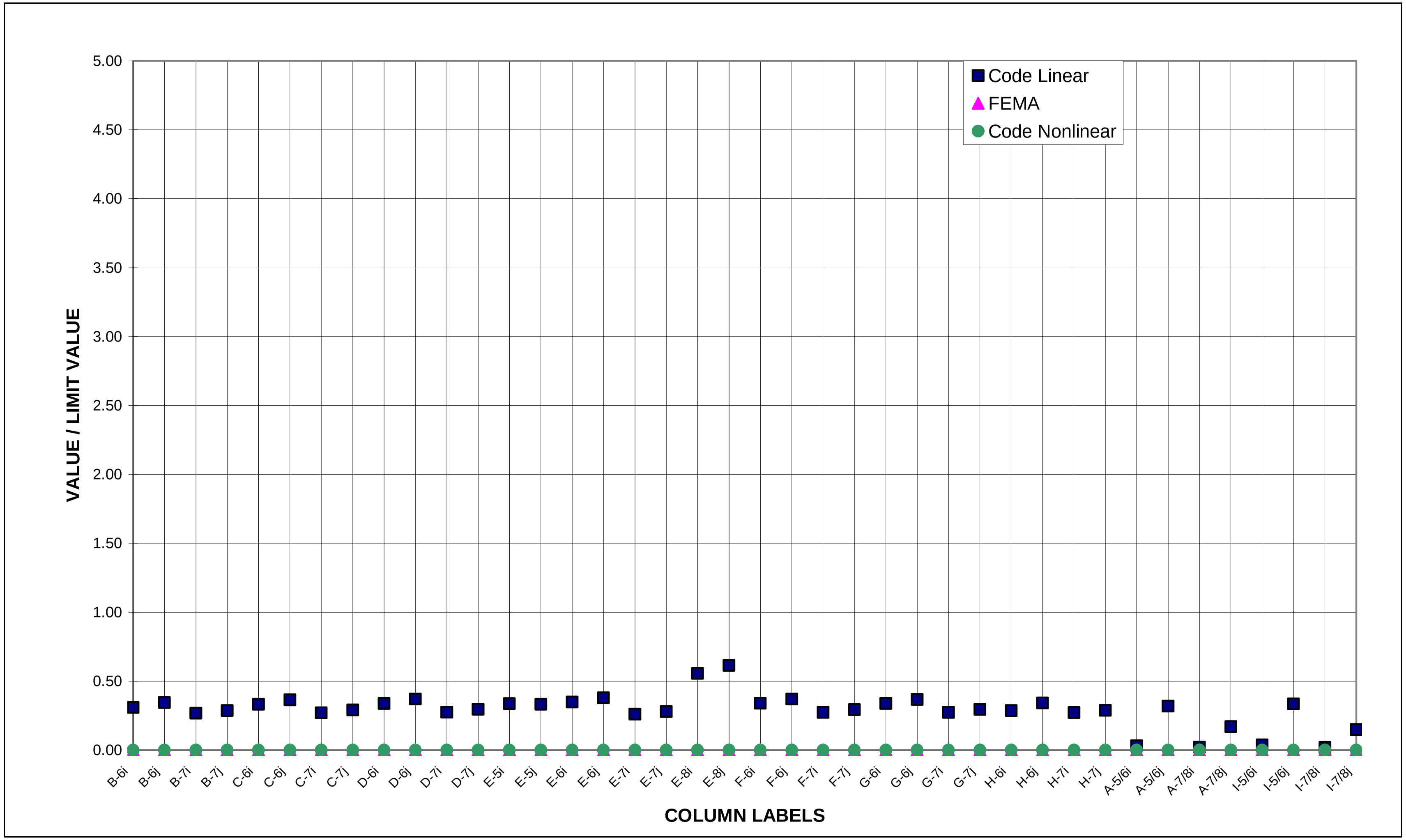


Figure C.11 Retrofitted System Life Safety Performance Level 5th Storey x – Direction Columns Performance Levels

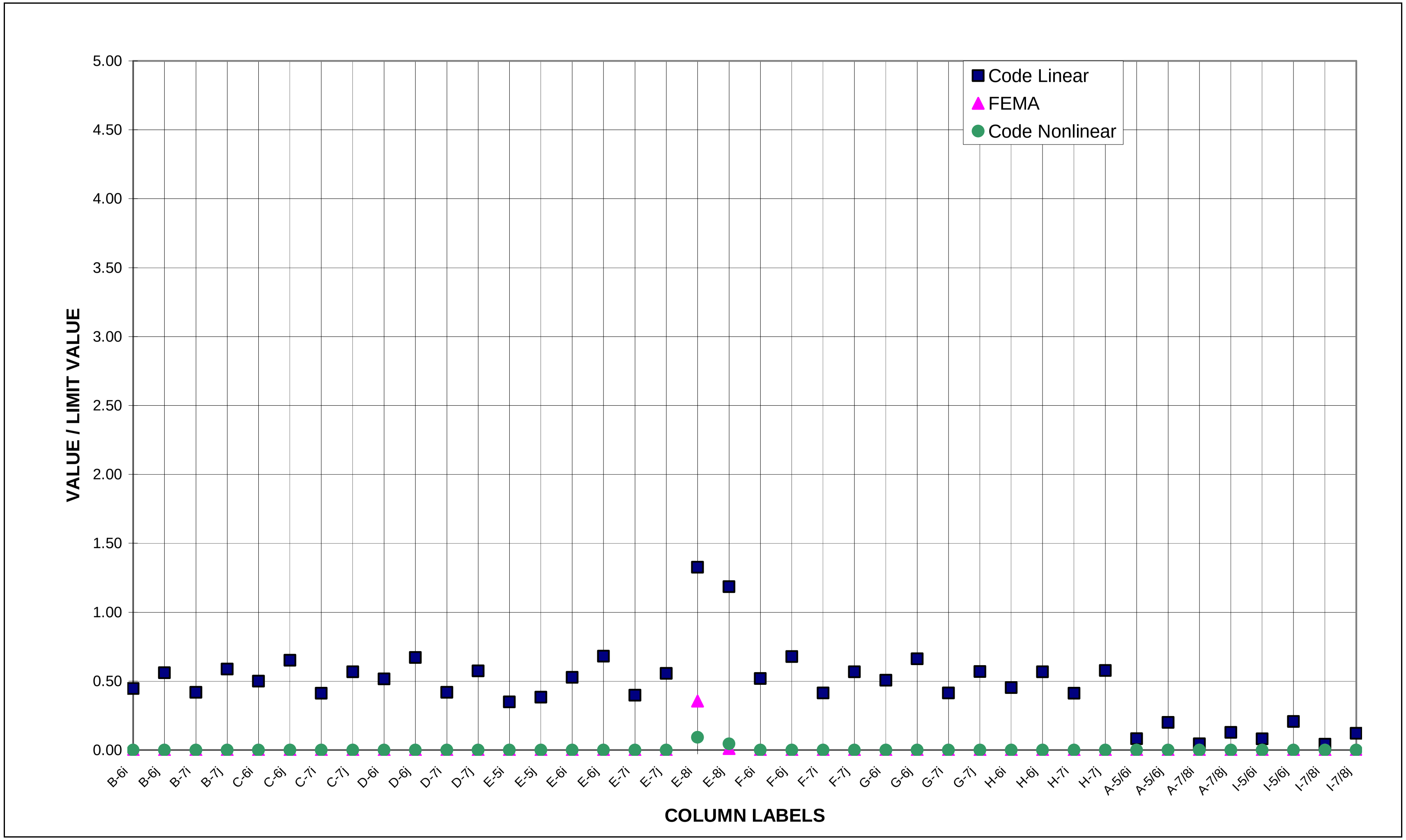


Figure C.12 Retrofitted System Life Safety Performance Level 6th Storey x – Direction Columns Performance Levels

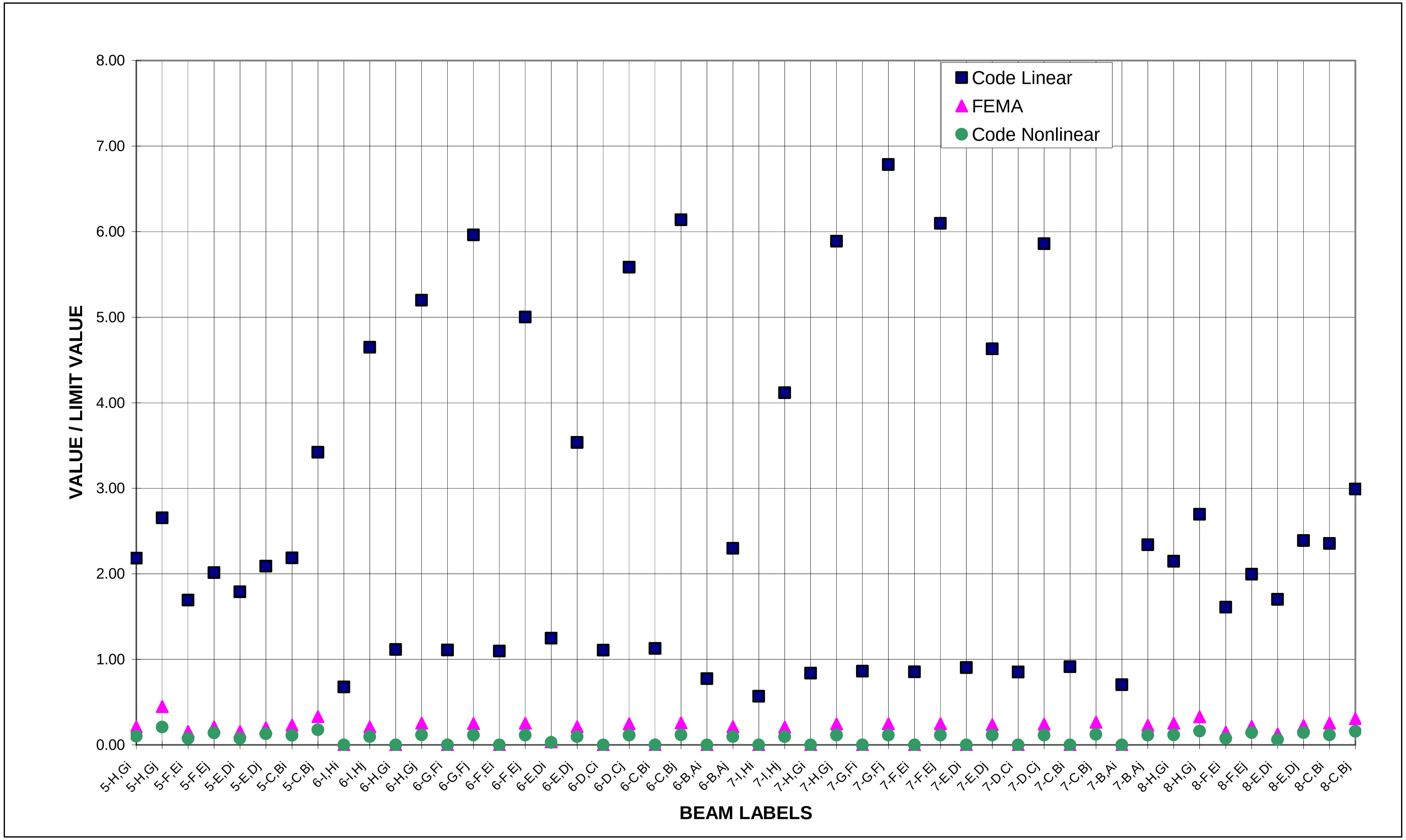


Figure C.13 Retrofitted System Life Safety Performance Level 1st Storey y – Direction Beams Performance Levels

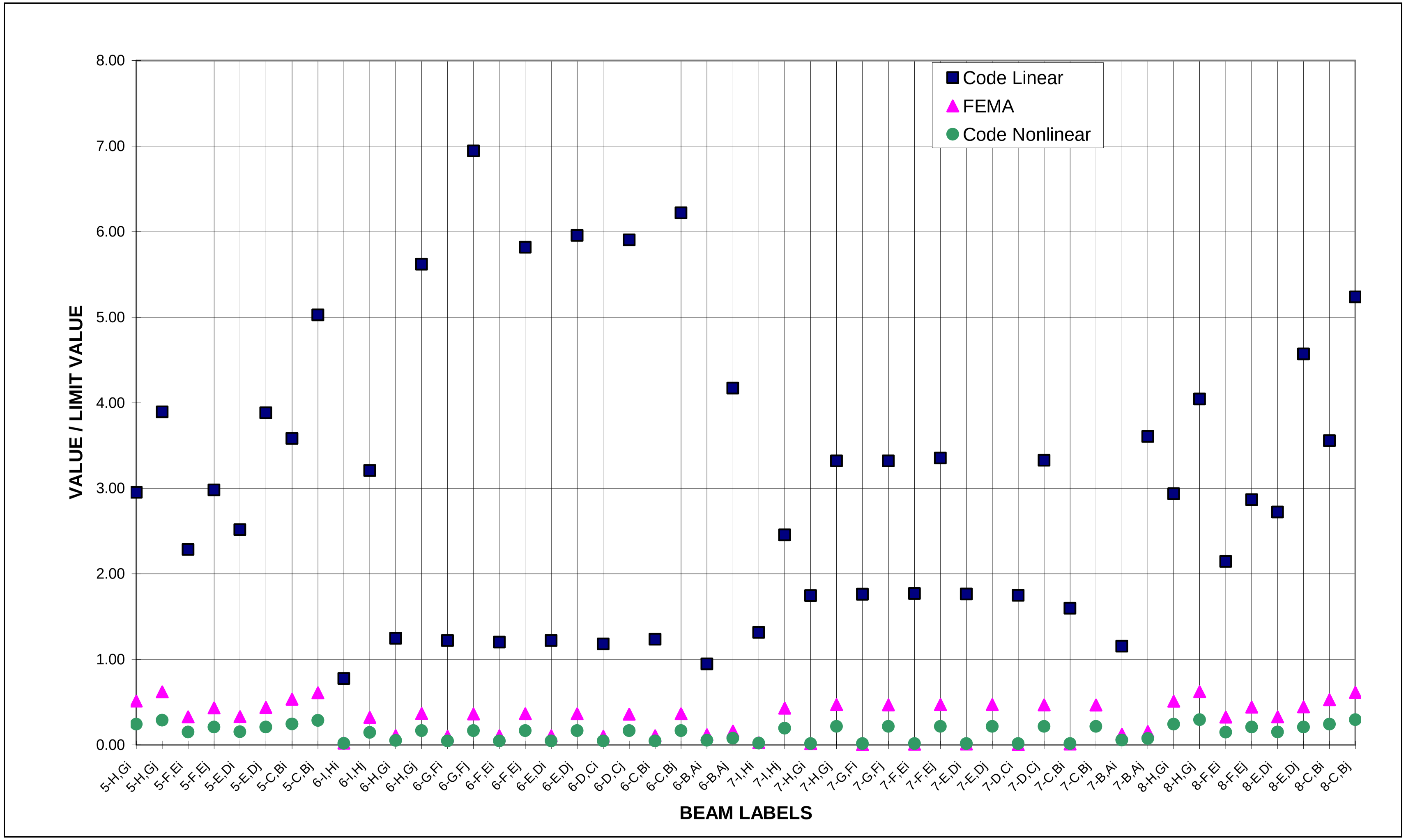


Figure C.14 Retrofitted System Life Safety Performance Level 2nd Storey y – Direction Beams Performance Levels

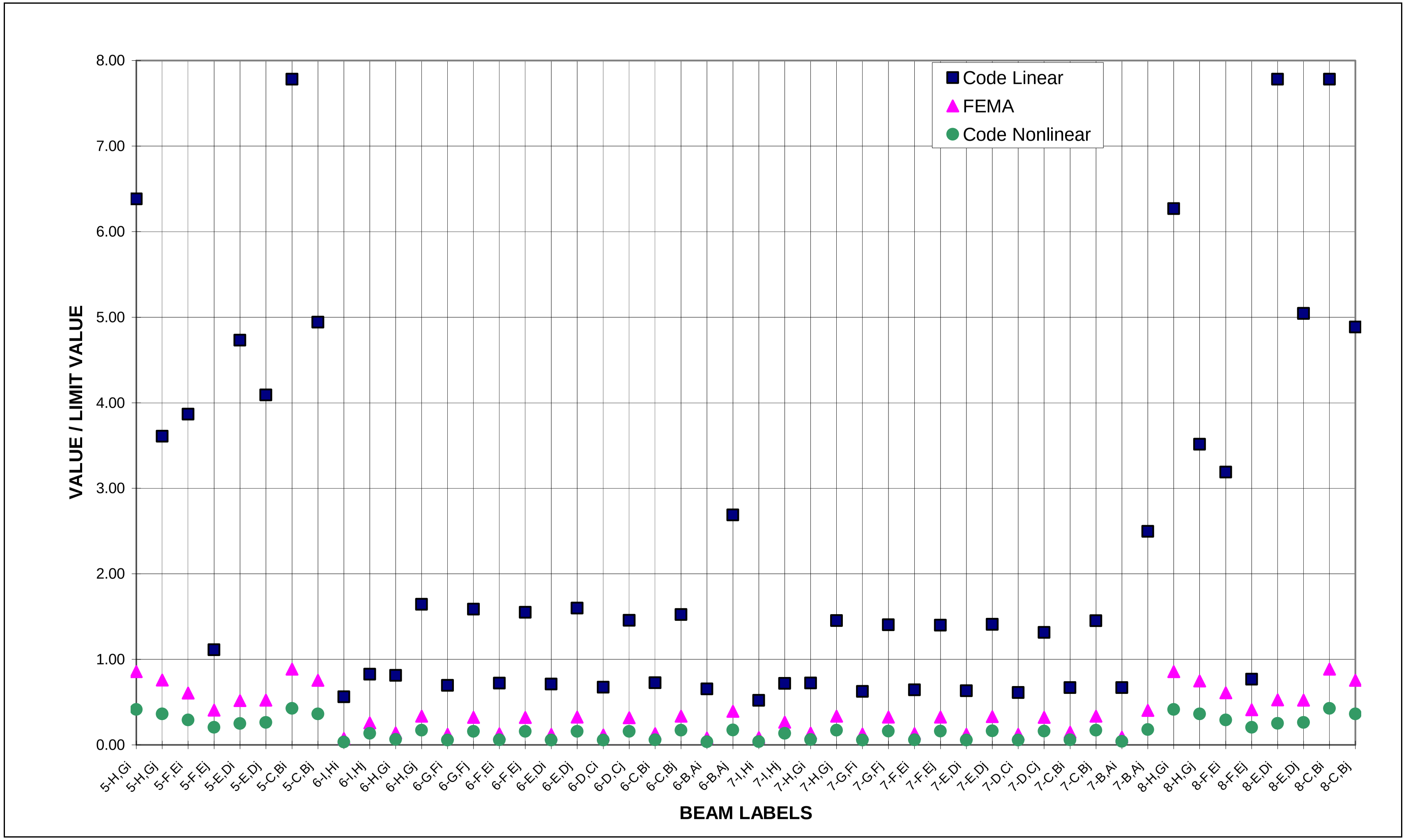


Figure C.15 Retrofitted System Life Safety Performance Level 3rd Storey y – Direction Beams Performance Levels

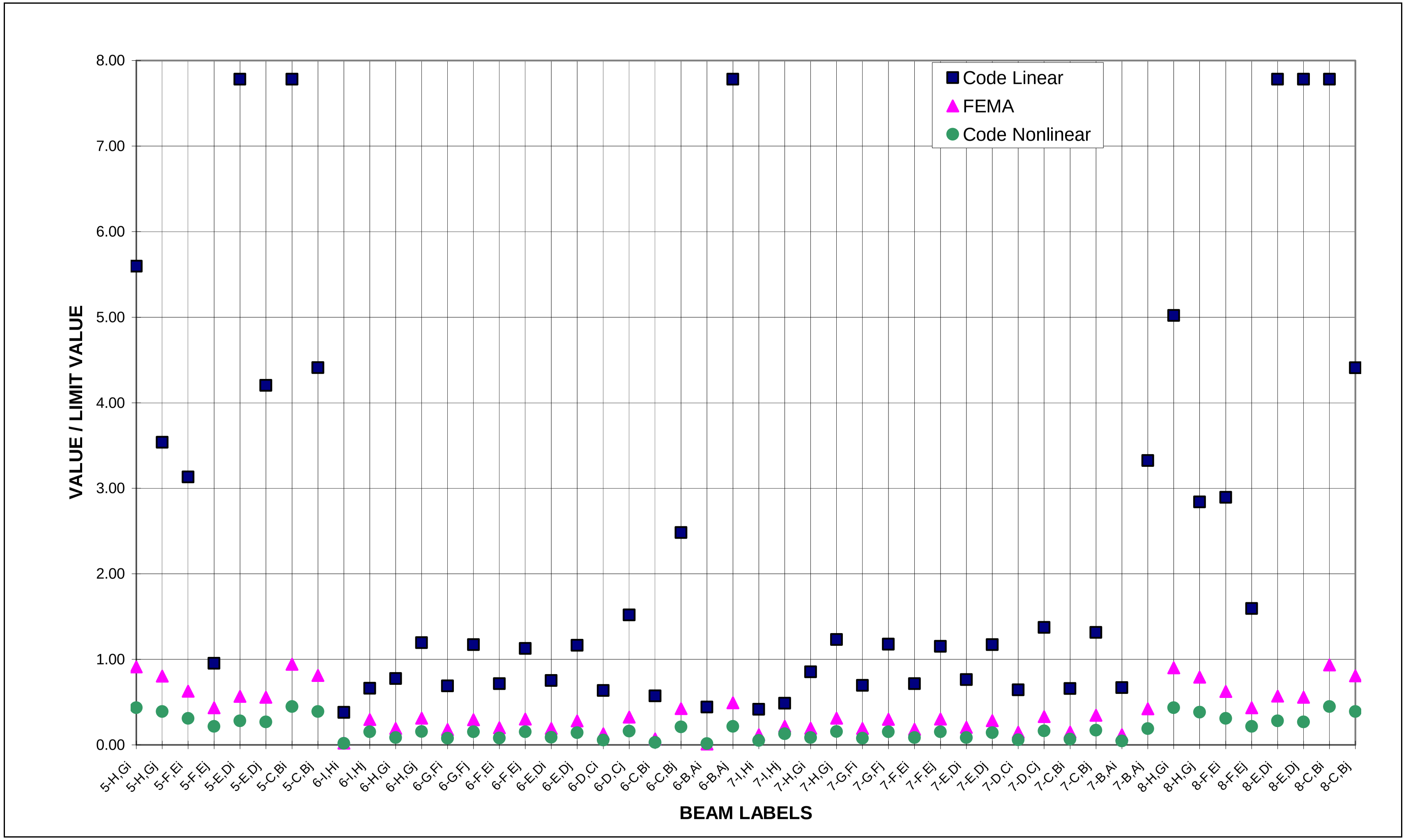


Figure C.16 Retrofitted System Life Safety Performance Level 4th Storey y – Direction Beams Performance Levels

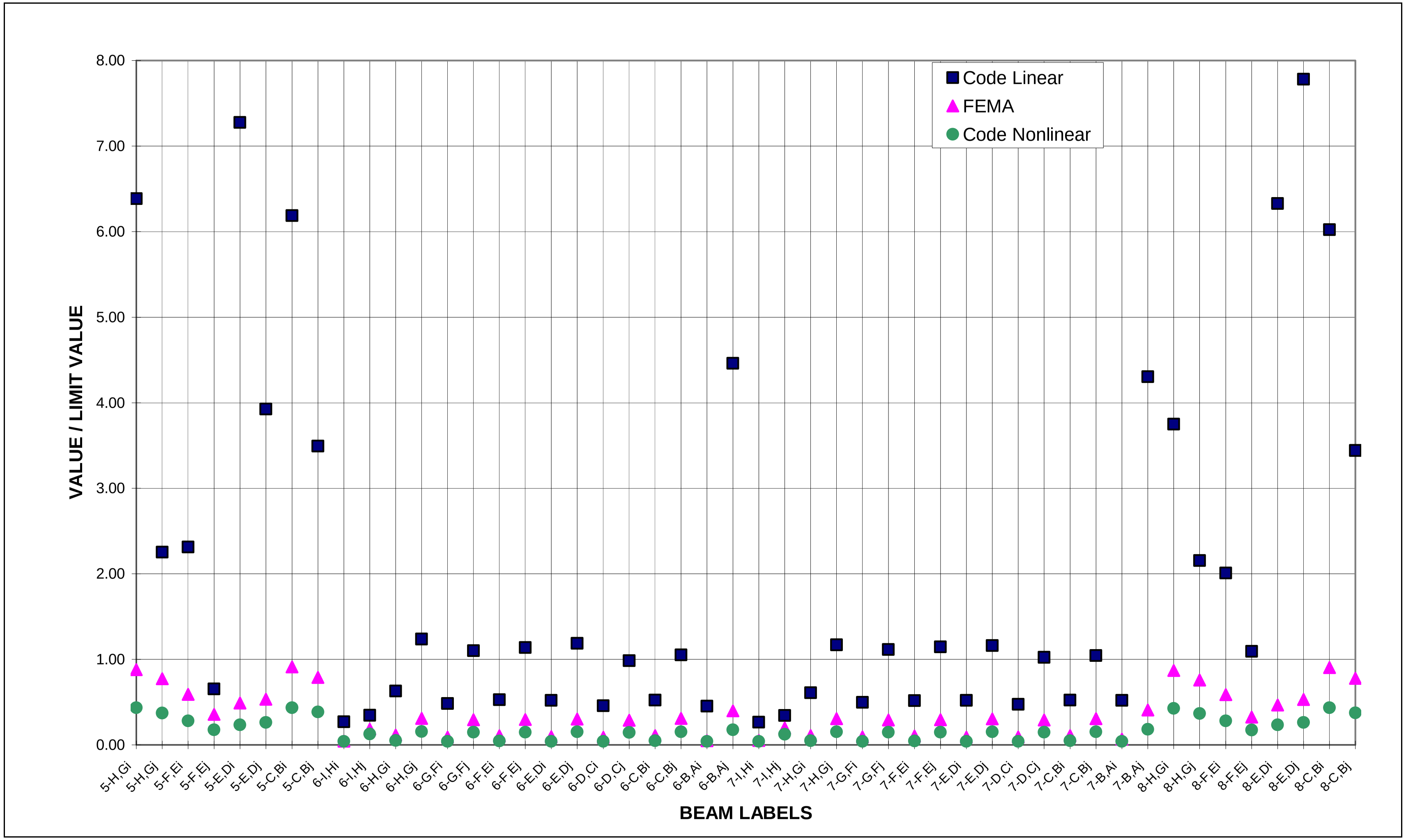


Figure C.17 Retrofitted System Life Safety Performance Level 5th Storey y – Direction Beams Performance Levels

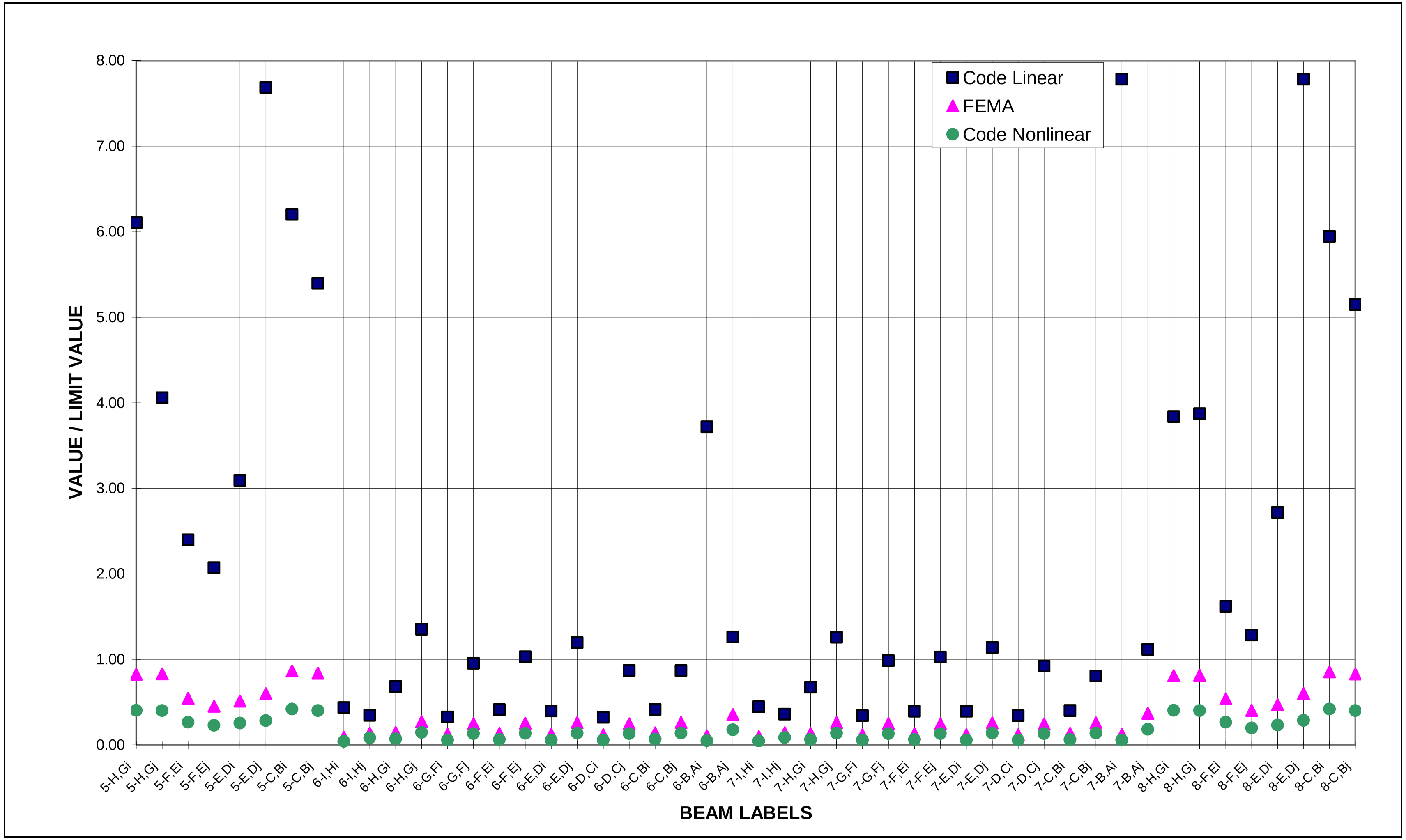


Figure C.18 Retrofitted System Life Safety Performance Level 6th Storey y – Direction Beams Performance Levels

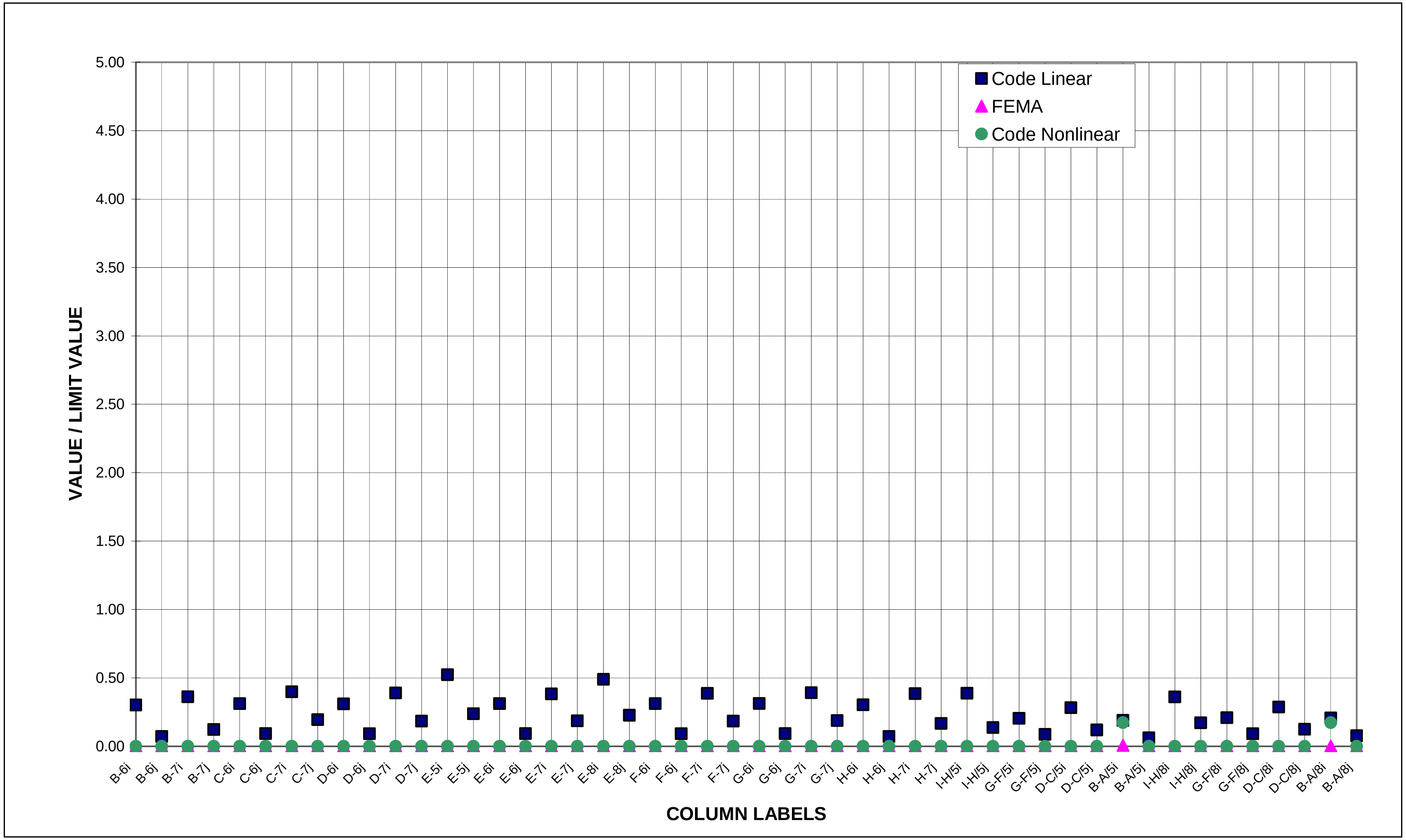


Figure C.19 Retrofitted System Life Safety Performance Level 1st Storey y – Direction Columns Performance Levels

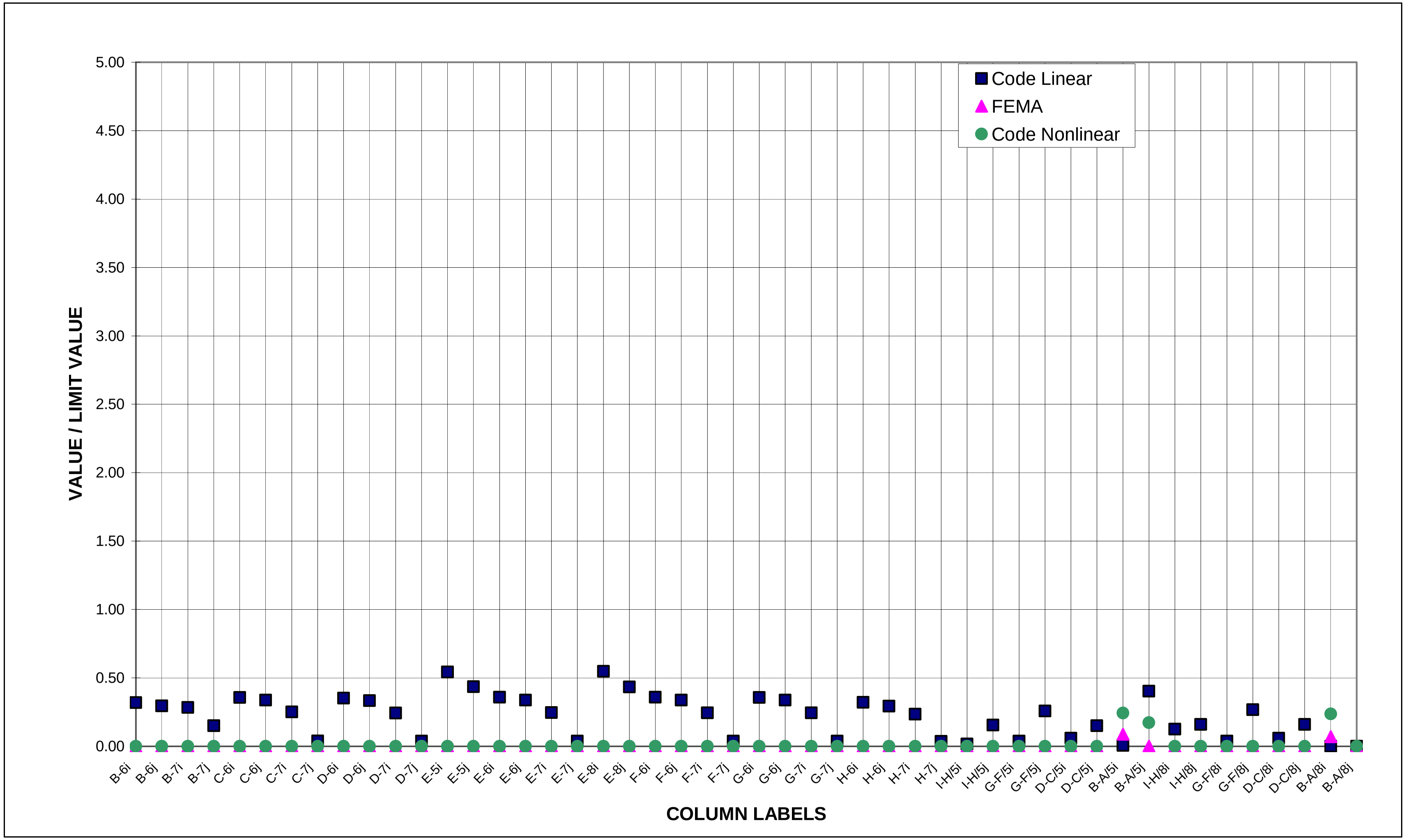


Figure C.20 Retrofitted System Life Safety Performance Level 2nd Storey y – Direction Columns Performance Levels

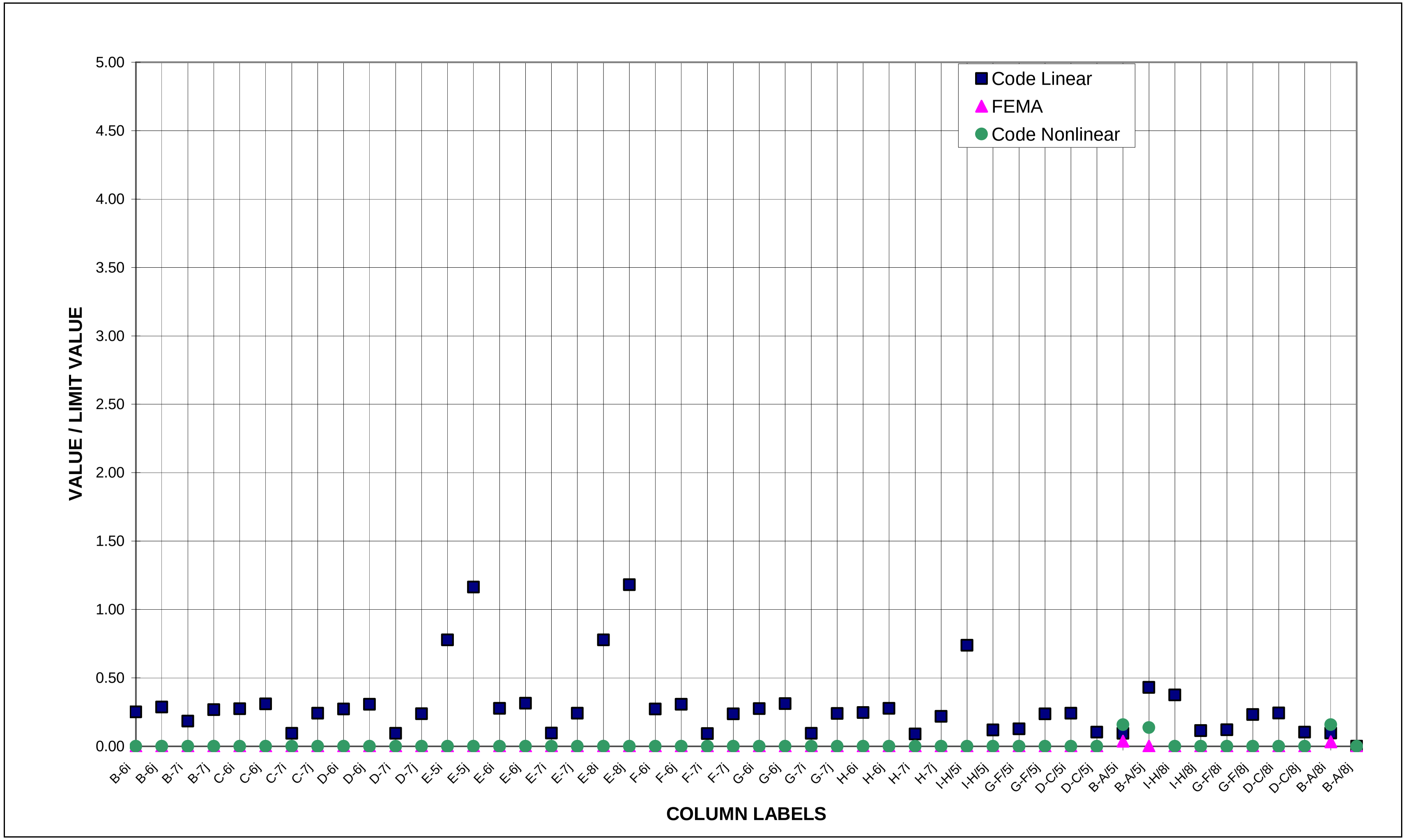


Figure C.21 Retrofitted System Life Safety Performance Level 3rd Storey y – Direction Columns Performance Levels

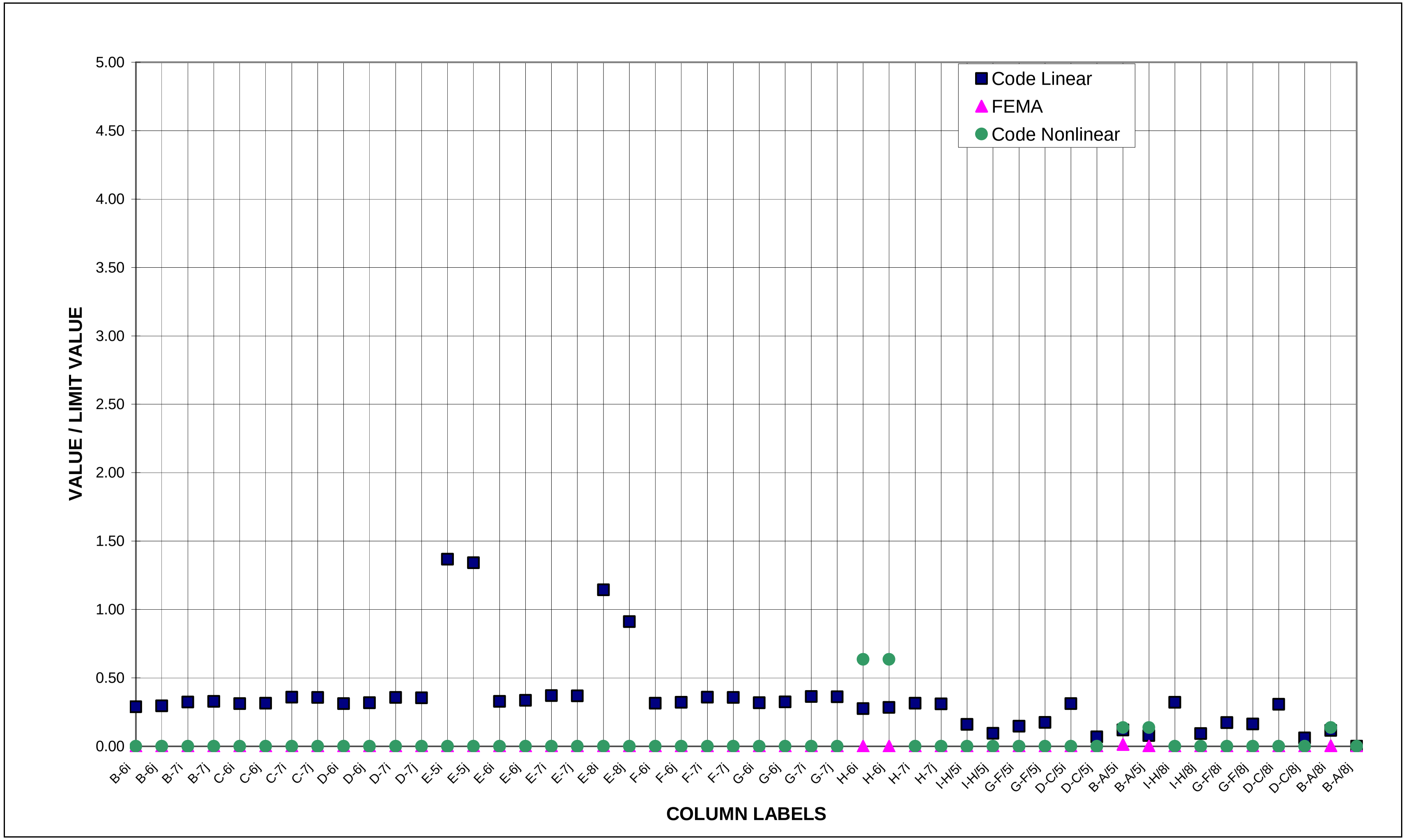


Figure C.22 Retrofitted System Life Safety Performance Level 4th Storey y – Direction Columns Performance Levels

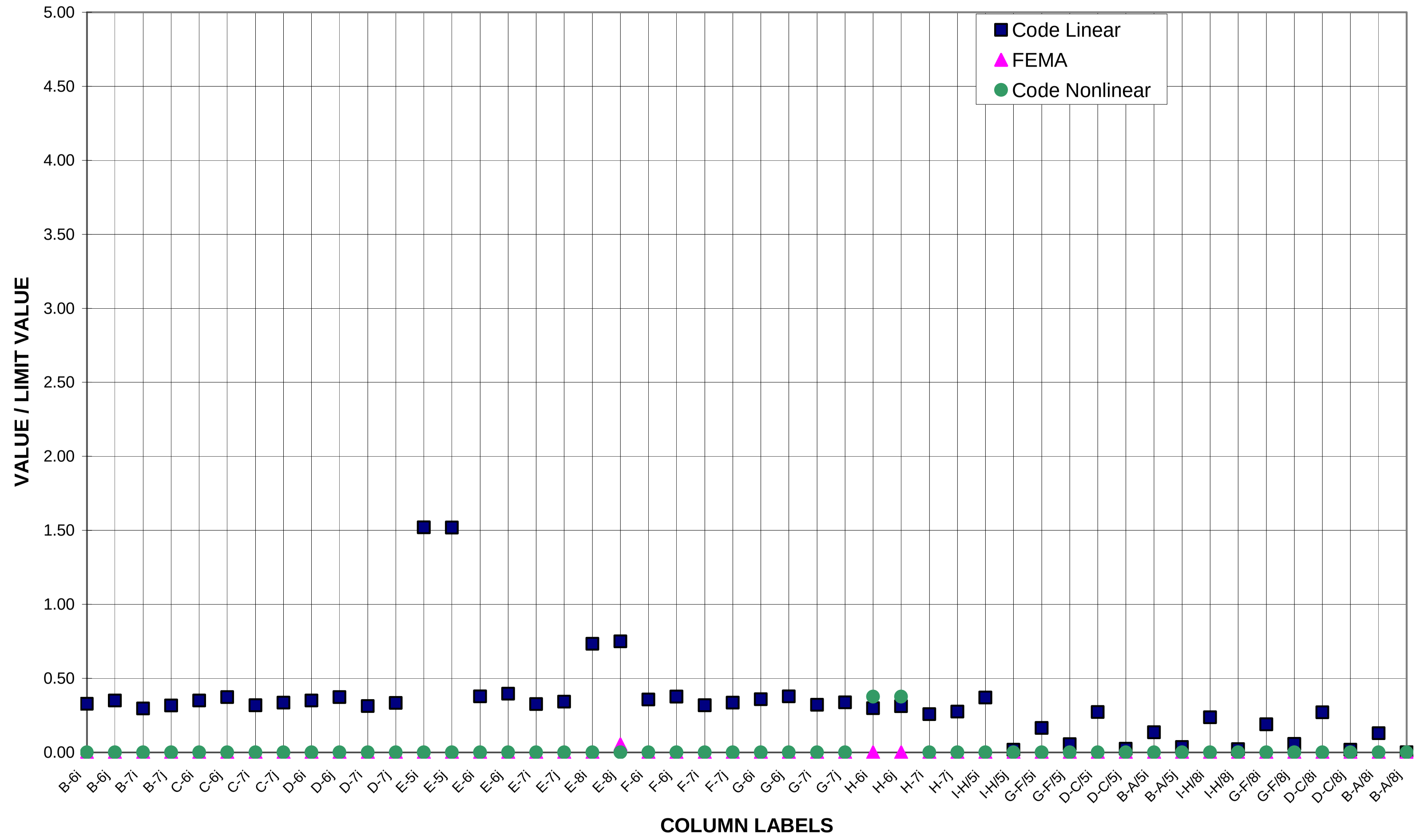


Figure C.23 Retrofitted System Life Safety Performance Level 5th Storey y – Direction Columns Performance Levels

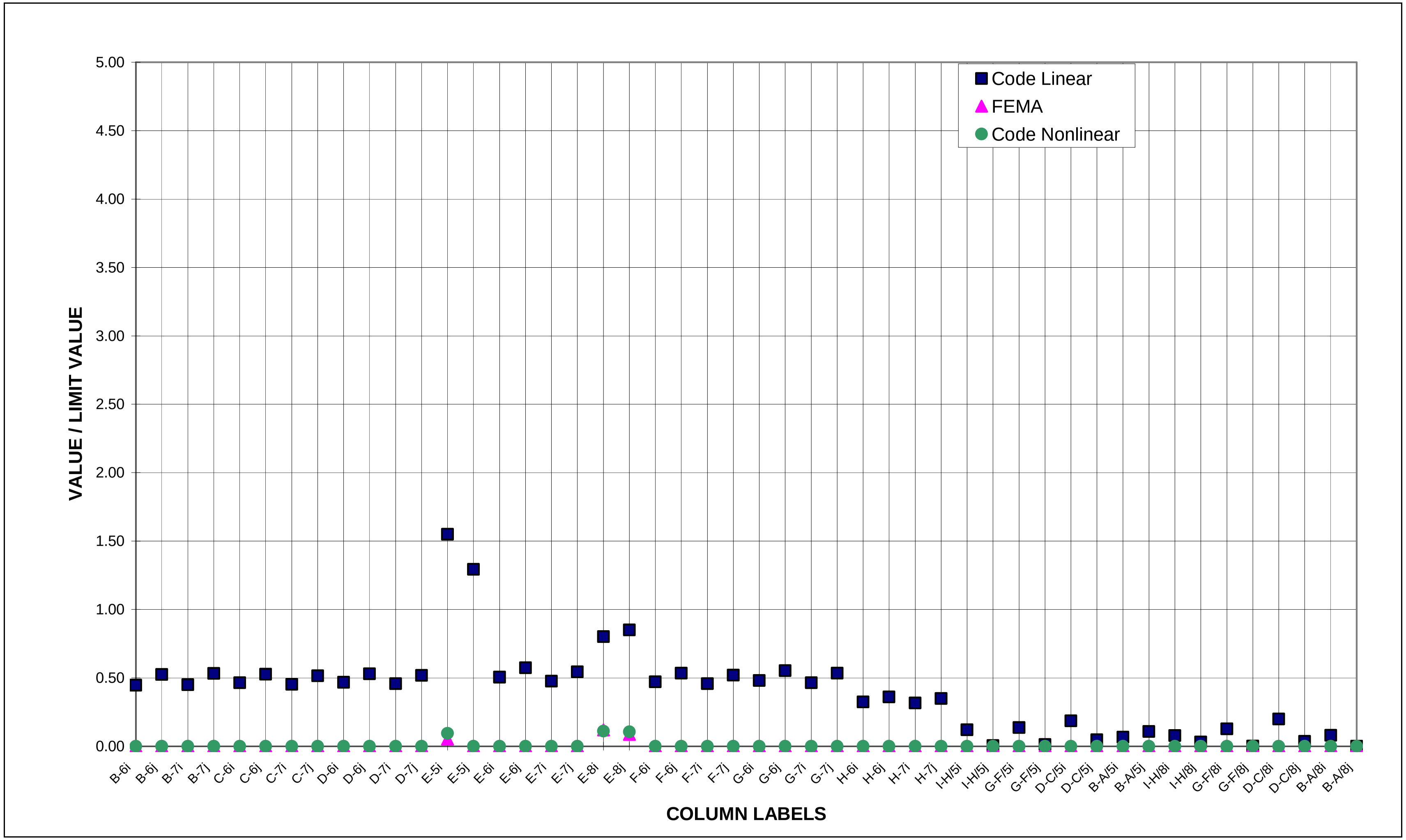


Figure C.24 Retrofitted System Life Safety Performance Level 6th Storey y – Direction Columns Performance Levels