Seismic Performance Assessment and Strengthening of Gazimağusa Namık Kemal Lisesi

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ABSTRACT

Many destructive earthquakes occurred in Cyprus. However, the potential seismic risk of the buildings in Cyprus is not known well since vulnerability is unknown. Especially in the Northern part of the Island building inventory has variation regarding seismic performance. On the other hand, in Northern Cyprus there are more than 150 school buildings with different ages. Most of these buildings have been constructed before the use of modern seismic codes. In other words, only gravity loads have been considered in the design of these school buildings. It is, therefore of paramount importance for us to identify seismic risk of buildings and especially school buildings.

Other important point is the corrosion of steel reinforcement in concrete, based on structure age and environmental causes, would easily decrease the service life of buildings. This could cause a disaster in case of an earthquake. The natural disasters could cause loss of human lives and increase the cost of repair of damaged buildings.

In this thesis, Gazimağusa Namık Kemal Lisesi (GMNKL) was evaluated as a reference school building and possible remedial measures for the building were discussed. GMNKL building is reinforced concrete and it has been constructed in 1956. This research was performed in three parts. In the first part, visual inspection of GMNKL was performed and deteriorations observed and other factors influencing earthquake resistance were determined. In the second part, material strengths (concrete and steel) were determined by using destructive and non-destructive tests. Also, reinforcement details were determined. Following the data collection, linear elastic and nonlinear static analysis were performed to identify possible weaknesses of buildings. Finally, in the third part of the research possible rehabilitation techniques were recommended.

In order to collect information several tests were performed on concrete and reinforcement of the building. Among these tests one of the test was the determination of corrosion potential and the results were used in the analysis. Analysis was based on FEMA 356, ATC-40 and Turkish Earthquake Code (2007). After analysis results of the existing building, the most suitable strengthening methods were selected to increase performance level of existing building. The strengthening methods used were column jacketing and shear wall construction. Applied strengthening methods were compared to each other based on the economic analysis and the seismic performance. The shear wall method was selected to be the best strengthening method based on criterias stated above.

It is expected that this study will be a reference for studying school buildings in other regions of Cyprus.

Keywords: Seismic performance, Destructive and non-destructive tests, Linear elastic analysis, Non-linear static analysis.

ÖZET

Kıbrıs`da son yıllarda birçok yıkıcı depremler meydana gelmiştir. Fakat, Kıbrıs`daki bu binaların sismik potansiyel riskleri iyi derecede bilinmemektedir. Özellikle adanın kuzey kesimindeki KKTC'de bina envanteri sismik performansta çeşitlilik göstermektedir. Diğer yandan, Kuzey Kıbrıs`ta 150`yi aşkın farklı yaşlarda okul binasının mevcut olduğu bilinmektedir. Bunların çoğu modern deprem yönetmeliklerinden önce inşaa edilmiştir. Diğer bir değişle, bu tür okul binalarının tasarlanırken yalnızca düşey ağırlık yükleri dikkate alınmıştır. Bu nedenle özelikle okul binalarının sismik risklerinin tanımlanması gerekmektedir.

Diğer önemli bir konu ise bina yaşına ve çevre etkilerine karşı beton içerisindeki betonarme donatısında meydana gelen korozyonun binanın kullanım yaşını kolayca azaltmasıdır. Bu olgu deprem anında binaya ciddi zararlar vermektedir.

Bu tezde, referans okul olarak seçilen Gazimağusa Namık Kemal Lisesi (GMNKL) değerlendirilmiş ve uygun güçlendirme yöntemleri tartışılmıştır. GMNKL binası betonarme olup ana bina 1956 yılında inşaa edilmiştir. Bu tez araştırması 3 kademede yapılmıştır. İlk kademede, görsel değerlendirme, mevcut olumsuzluklar ve diğer deprem dayanımına karşı etkiler belirlenmiştir. İkinci kademede, malzeme dayanımları (beton ve çelik) yapılan tahribatlı ve tahribatsiz methodlarla belirlenmiştir. Aynı zamanda, taşıyıcı elemanlardaki donatılar saptanmıştır. Veri toplama sonrasında, doğrusal elastic ve doğrusal olmayan statik analizler uygulanmış ve olası güçsüzlükler belirtilmiştir. Son kısımda, ayrıntılı olarak uygun güçlendirme teknikleri tartışılmış ve önerilerde bulunulmuştur.

Beton ve çelik özelliklerini belirlemek için birçok deney yapılmıştır. Bu testler arasında, çelik paslanma potensiyel deneyi de yapılmış ve elde edilen sonuçlar analizlerde kullanılmıştır. Analizler FEMA 356, ATC-40 and Türk Deprem Yönetmeliği`ne (2007) uygun olarak yapılmıştır. Analiz sonuçlarına göre mevcut binaya en uygun güçlendirme methodu seçilmiş ve binanın performans seviyesi yükseltilmiştir. Güçlendirmede kolon mantolama ve perde duvar yöntemleri kullanılmıştır. Uygulanan güçlendirme yöntemleri kendi aralarında ekonomik yönden ve performanslarına göre de karşılaştırılmışlardır. Uygulama sonucunda en iyi güçlendirme metodunun perde duvar yöntemi olduğu belirlenmiştir.

Bu çalışmanın Kıbrıs`ın diğer bölgelerindeki okul binalarına örnek bir çalışma olması beklenmektedir.

Anahtar Kelimeler: Sismik performans, Tahribatlı ve tahribatsız deneyler, Doğrusal elastik analiz, Doğrusal olmayan statik analiz.

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And for my family, words are worthless for their support. Thank you very much.

Dedicated

to

My parents

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CHAPTER 1

INTRODUCTION

1.1 Preface

For those countries which are located in the earthquake region, it is important that they should quickly determine the condition of the buildings and whenever needed decide on possible strengthening methods. Besides that, to determine the performance of a building and classify its safety level for a presumed earthquake will help to limit the damages during an earthquake. Like in many other countries, due to the experiences gathered from earthquakes, the earthquake regulations get improved. In the latest earthquake codes the safety levels of buildings are increased. Due to the fact that many of the buildings in our country were built before the present earthquake code and keeping in mind that they were mostly built for purposes other than they are used today. It seems inevitable that all buildings should be checked for their earthquake performance, beginning with the important public buildings in high risk areas.

The earthquake performance of a building can be defined as its capacity of bearing seismic demand loads. The non-linear static method developed to identify seismic performance, known as pushover analysis, goes back to the 1970's. However, from the frequency of publications, it can be seen that it has gained much importance within the last 10 – 15 years. In these works, the primary discussion started about implementation, advantages and disadvantages of the matter. The method is compared with presently used linear elastic methods and non-linear time history analysis methods.

The purpose of the pushover analysis is to evaluate the performance of a building by determining the resistance and deformation tolerance in relation to the earthquake system matching the relevant performance levels. This method also considers secondary impacts, the hyper-elastic behavior of materials and the redistribution of internal forces. Linear analyses methods try to meet the above stated requirement by using certain coefficients given by the regulations, but still are insufficient to explain the damages of a building after an earthquake. It is completely uncertain how a building could behave after earthquakes. Conclusively it could be stated that linear calculation methods are insufficient in the earthquake performance calculations of buildings.

There are various reasons why the resistance of building has to be improved. Buildings with mistakes or deficiencies in the project or its implementation tend to show weaknesses or damages in various elements; the change of the usage of some buildings in time can require some changes in the load bearing construction. Apart from that, the most important reason for the need to restore and strengthen a building is due to the effects of earthquakes. In order to understand the reasons and requirements for restoration and strengthening of building against earthquake, we have to understand the terminology of earthquake resistant construction design.

According to the currently applicable principles of earthquake resistant design, buildings are designed to minimize human losses in a heavy earthquake, which is assumed to happen at least once in their economic lifespan. The result of this approach could be stated as follows: Buildings with earthquake resistant design will suffer damages in their load bearing or non-structural elements as a result of the heaviest presumed earthquake. In a context where building designed to resist forces are presumed to be damaged, it is certain, that old or new buildings without sufficient precautions for earthquake resistance will suffer much more critical damages.

The need to catalogue, examine and repair existing buildings is derived from the fact that in TRNC, as in many other fields too (the corrupt construction practice of the recent years) has begun threatening human health and social and economic safety severely. Construction procedures which did not consider natural disasters could cause serious loss of lives and economic loss. The most critical natural disaster in this region is earthquake, which is mostly not seriously sufficiently considered during design.

The average age of schools in TRNC and the related problems (weakening of material, lack of construction compliance to regulations etc) are examined and evaluated in terms of earthquake risks and in that context the present status is shown together with the statement of the problems, proposed solutions and the proposed methods of strengthen the buildings. As being one of the oldest school buildings in the TRNC and being the first Turkish Cypriot school built, the Gazi Mağusa Namık Kemal Lisesi (GMNKL) has a distinguished importance and was hence chosen as a pilot school for this thesis.

School buildings and Hospital buildings generally have higher earthquake performance than the other buildings as they should remain intact after the anticipated earthquake. For this reason, buildings of that kind are directly subject to broad evaluation. Within the scope of that broadened evaluation, the examinations of the building materials are of a special point of interest. The objectives of the examination of these building materials are not for only quality control. It is also important to determine the carbonation of concrete, the oxidation of the reinforcement and if applicable, to detect related weaknesses and damages, to verify if concrete layers and reinforcement configurations are implemented according to the official requirements.

1.2 Objectives and Scopes

The main objective of this thesis is to determine possible weaknesses of one of the old school building in Famagusta, namely Namık Kemal Lisesi. Possible remedial measure in order to improve seismic performance of the building is also considered. In order to achieve above objective several stage from identifying the structural system material properties, modeling, analysis etc. have been performed and prevented in this thesis. It is expected that this study will be a reference for studying school building in other region of Cyprus.

After various implemented destructive and non-destructive tests from GMNKL, the building was modeled using ideCAD-5.511 software. This model was analyzed using the non-linear static (pushover analysis) and linear elastic analyses to determine the building that may collapse during or after an earthquake. In upcoming chapters we will see the detailed results of these analysis and economically reasonable methods of improving the stability of the building according to the test results.

During linear analysis, the elastic capacity of the building can be assessed; non-linear methods have to be considered for determination of hyper-elastic capacity. The non-linear method of calculation where horizontal (lateral) loads are increased while the vertical loads are kept constant is called static pushover analysis. As this method reflects the realistic behaviour of a building during seismic shocks then, it allows the calculations to be done more accurately. With the static pushover analysis the deformation behaviour of all structural elements of a building can be defined. Exceeding the boundaries of the elasticity of the material, this calculation method also utilizes its plastic capacity. The performance oriented analysis can easily answer the below questions:

a) Which non-structural elements will suffer damage?
b) How is the distribution of damages within the load-bearing system?
c) What is the extent of those damages?

Conclusively, the static pushover analysis examines the status of the building at the point where the demand of seismic forces meets the response of the building. The characteristics of a building at that performance spot are determined in advance according to the occupation purpose and our expectancies of that building. The main objective at that point is to ensure a minimum level of vital safety regardless of the economic situation.

Today ATC-40 and FEMA-356 have two approaches for the performance oriented design, which are actually similar to each other and, in chapter 7 of newly set up Turkish seismic code 2007 of (TEC,2007) performance evaluation methods are also mentioned. The performance point defined by the codes will be achieved for the strengthening techniques. According to the anticipation of these regulations the most broadly used non-linear methods of analysis are the Capacity Spectrum Method (ATC-40) and the displacement Coefficient Method (FEMA-356). According to the

TEC 2007 the usage level of a building has to be determined by assessing the condition through Linear Elastic Analysis.

If a requirement for strengthening is decided, the strengthening process should consider economic convenience and applicability. Nowadays there is a rapid development in the techniques for strengthening the damaged buildings or buildings suffering reduction of their capacity. The most common methods are CFRP, Exterior Sheathing and adding shear walls. As can be seen from several tests, CFRP has a shear-column effect if implemented on brick surfaces (Özcebe and et all, 2003) and the convenience of application makes it to be used commonly. As the process of exterior sheathing is a little more complicated and depending on the usage purpose difficulty can occur for a certain time inside of the building what can make the application difficult. As of the lack in the sectional dimensions of the columns, they hardly achieve the limit values of DL, LL and seismic loads they have to bear according to the regulations. For this reason the dimensions for necessary elements have to be increased. According to the article written by Kaltakci et all (2007), the assessment and strengthening of the school building which was built before 1998 were made with the methods stated above. In the conference draft of Ersoy (2007) strengthening methods and implementation requirements are explained in detail. In 2008 Kanit R. and the others have analyzed the performance of Aksehir Girls' Vocational School built in 1972 and determined the characteristics of the used materials with destructive and non-destructive methods, and strengthened weak elements by Exterior Sheathing Applications and shear walls.

1.3 Organization and Outline

In Chapter 1, introduction to the subject, background information about the assessment procedures, objectives and scope of this study are mentioned. In Chapter 2, general information about the factors affecting seismic performance of buildings are given. Information about assessment methods are described in Chapter 3. Chapter 4 explains seismic performance assessment analysis procedures and ideas. In Chapter 5, repair and strengthening methodologies are explained. Chapter 6 gives the properties of bearing elements and results of tests. In Chapter 7, linear and non-linear analyses are performed to determine the performance assessment of GMNKL. Summary, conclusions and recommendations are given in Chapter 8.

CHAPTER 2

FACTORS AFFECTECTING SEISMIC PERFORMANCE OF A BUILDING

2.1 Introduction

The structural system of a building has to resist seismic loads as a whole as well as each structural element of the system should be provided with sufficient stiffness, stability and strength to ensure an uninterrupted and safe transfer of seismic loads down to the foundation and soil. In this respect, it is essential that floor systems possess sufficient stiffness and strength to ensure the safe transfer of lateral seismic loads between the elements of the structural system [TEC 2007].

Over the past years in Turkey, the majority of structural damages in reinforced concrete buildings after an earthquake are determined to be due to the following:

- Insufficient or inaccurate reinforcement details.
- Non-earthquake conform, architectural design and bearing systems which are either not earthquake-resistant or involving weaknesses,
- Construction errors.

The issues which are stated above in three groups have played the major role in the earthquake damages on buildings in our country. These issues will be discussed individually.

2.2 Irregular Bearing Systems of Structures

2.2.1 Projections In Plan

The earthquake resistance of a building starts at the architectural planning phase. For example, buildings without symmetry in their project or projections of upper floors will be negatively affected in their earthquake resistance. On the other hand, the supporting structural system of the building will be negatively affected, in terms of safely resisting the inertia forces developed by the earthquake. The lack of symmetry in the structural system of a building causes the formation of torque. Torque cause increase in shear forces especially in the column and shear walls near the perimeter of the building. The disruption of symmetry could as well be causes by the supporting structure, as by infill walls, which were not considered in the calculation. Although infill walls are generally omitted in the calculations, they improve the lateral rigidity of a building significantly [Canbay et all, 2003].

Buildings with big extensions in the layout will cause damages by accumulating tension at the corners. Three examples of extensions are as in shown in Figure are 2.1. The cases where projections beyond the re-entrant corners in both of the two principal directions in plan exceed the total plan dimensions of the building in the respective directions by more than 20%. At that kind of buildings damages can be prevented by two kinds of precautions. One of those precautions is to separate wings or extensions with expansion joints. The second precaution is to build bearing walls in critical areas.

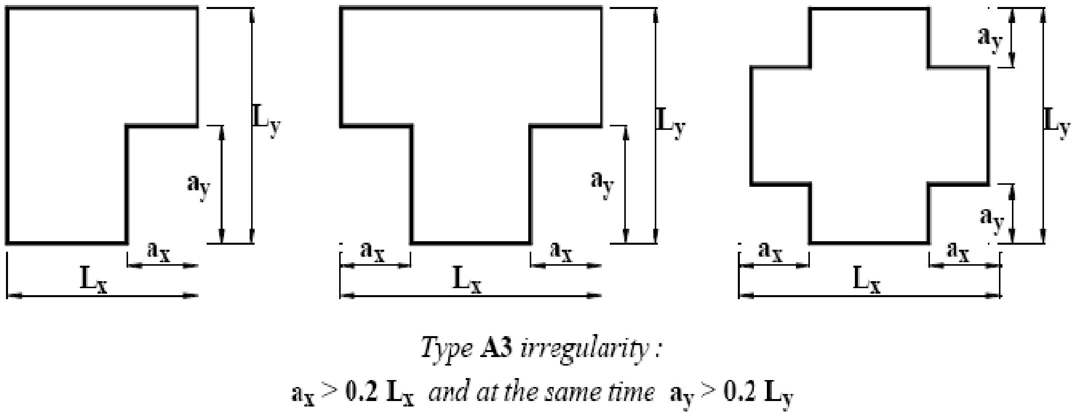


Figure 2-1 Irregularity in the plan of a project [TEC2007]

2.2.2 Soft or Weak Storey

One of the most common type of damage results from ``soft`` or ``weak`` storey as shown in Figure 2.2 and Figure 2.3. During the earthquake, the presence of a soft story increases deformation demands very significantly, and put the burden of energy dissipation on the first-story columns. Many failures and collapses can be attributed to the increased deformation demands caused by soft stories, coupled with lack of deformability of poorly designed columns. According to the TEC 2007 a storey is considered weak if the relation of the effective shearing area of any storey to the upper next one is less than 0.8. A soft storey on the other hand, results from a storey that (especially ground floor) is less rigid than the others. The occurrence of a soft storey could result from the bearing structure, or infill walls. As an infill wall enhances the rigidity of the frames, stories without or few infill walls will possess less lateral rigidity than others.

In all seismic zones, columns at any storey of the building shall in no case be permitted to rest on top or at the tip on of cantilever beams or haunches provided in the columns underneath. Structural walls shall in no case be permitted in their own plane to rest on the beam span at any storey of the building [TEC, 2007].

As explained above, in reinforced concrete buildings, the case where in each of the orthogonal earthquake direction, stiffness irregularity factor, η_{ci}, which is defined as the ratio of the effective shear area of any storey to the effective shear area of the storey immediately above, is less than 0.80. This relation is shown below by Equation 2.1.

$$\eta_{ci} = (\sum A_e)_i / (\sum A_e)_{i+1} < 0.80 \qquad (2.1)$$

On the other hand, soft storey is the case where in each of the two orthogonal earthquake directions, stiffness Irregularity Factor, η_{ki}, which is defined as the ratio of the average storey drift at any storey to the average storey drift at the storey immediately above, is greater than 2. This relation is shown below by Equation 2.2.

$$\eta_{ki} = (\Delta_i/h_i)_{ave} / (\Delta_{i-1}/h_{i-1})_{ave} > 2.0 \qquad (2.2)$$

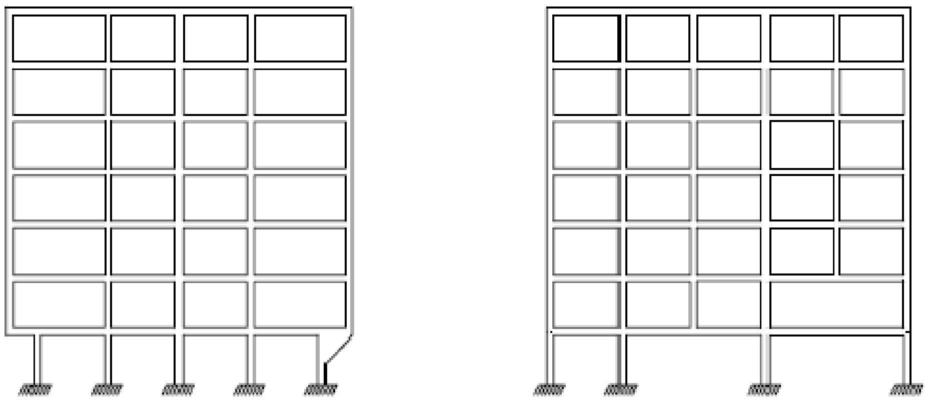


Figure 2-2 Soft storey

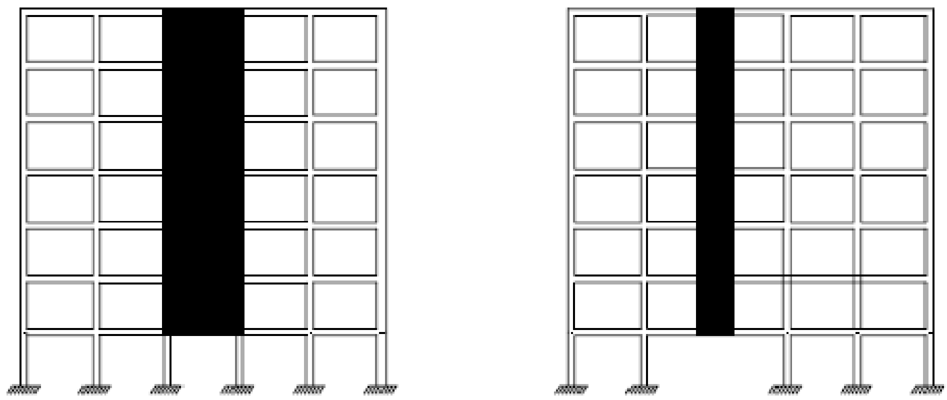


Figure 2-3 Weak storey

2.2.3 Short Columns

Another structural weakness is due to the short columns. Figure 2.3 shows how short columns can occur as a result of the load carrying elements as well as from windows which over span the whole wall. This second type can easily be overseen as infill walls are generally omitted in the calculations. Under the impact of an earthquake, any column can bear loads on both ends. In that case, the shearing force on the column ($\boldsymbol{V_e}$) will be defined as follows, where the moment capacity of the column at both ends are stated as $\boldsymbol{M_a}$ and $\boldsymbol{M}_{ü}$.

$$V_e = \frac{M_a + M_{ü}}{\ell_n} \tag{2.3}$$

$$V_e \leq V_r \tag{2.4}$$

$$V_e \leq 0.22 A_w f_{cd} \tag{2.5}$$

Where $\boldsymbol{V_e}$ is shear force taken into account for the calculation of transverse reinforcement of column or beam, $\boldsymbol{V_r}$ is shear strength of a cross section of column, beam or wall, $\boldsymbol{A_w}$ is effective web area of column cross section, $\boldsymbol{f_{cd}}$ is design compressive strength of concrete, $\boldsymbol{\ell_n}$ is clear height of column between beams, clear span of beam between column or wall faces, $\boldsymbol{M_a}$ is moment at the bottom of column clear height which is used for the calculation of column shear force, $\boldsymbol{M_{ü}}$ is moment at the top of column clear height which is used for the calculation of column shear force.

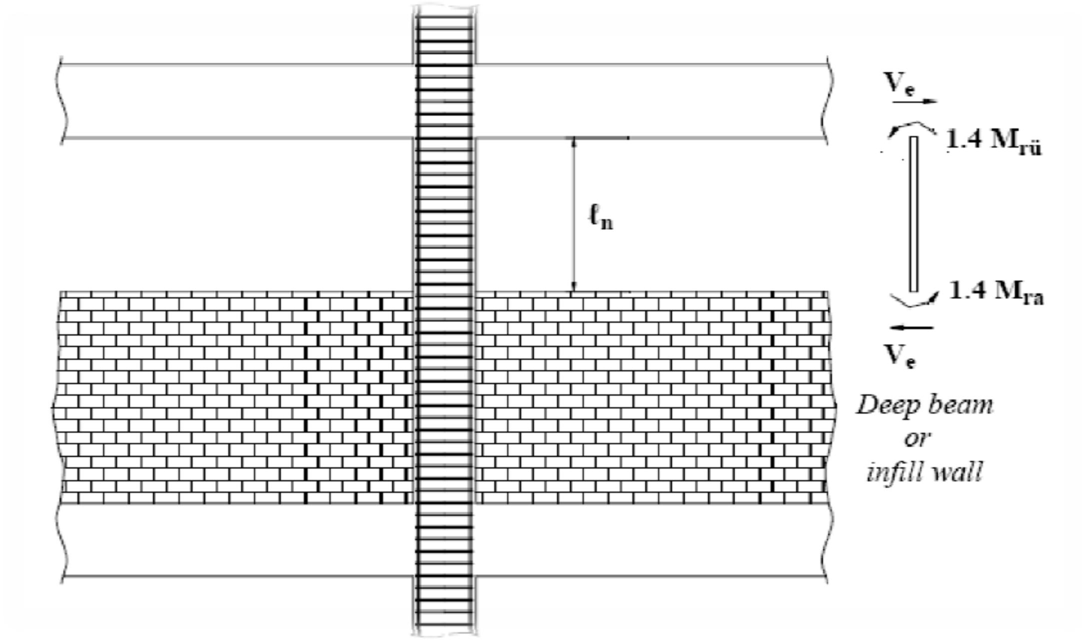


Figure 2-4 Example of short columns [TEC 2007]

In cases where short columns cannot be avoided, shear force for transverse reinforcement shall be calculated by equation 2.3. The moments in equation 2.3 shall be calculated at bottom and top ends of the short column as

1.4 and 1.4

Where is ultimate moment resistance calculated at the bottom of column or wall clear height, is ultimate moment resistance calculated at the top of column or wall clear height.

In addition, calculated shear force shall satisfy the conditions given by Equation 2.4. The minimum transverse reinforcement requirements and conditions of arrangement defined in TEC 2007 for column confinement zones shall be applied along the length of the short column. Transverse reinforcement shall be extended along the full storey length of columns which are transformed into short columns in between infill walls (Figure 2.3) [TEC 2007].

One cause for damages on buildings in Turkey is due to having stronger beams than columns. In that case plastic hinges occur at the columns, which are less ductile than beams. The column ends do not have generally sufficient reinforcing hooks, Therefore, plastic hinge causes an immediate and brittle break which mostly results in collapse of the building.

2.3 Reinforcement Details

Deficiencies caused by reinforcement details which resulted in severe damages or collapses will be briefly mentioned in this part. Generally the anchorage length of the reinforcement of the beam is too short (i.e. less than the required development length). This is due to design engineer who omits this detail. This reinforcement might have to carry tensile stresses, too. Also, it is known that the fresh concrete is not compacted properly at the bottom of column and connection point with beams (Figure 2.4). As the areas where possible plastic hinges occur are not wrapped, those sections which do not have enough ductility, will absorb energy, which makes them fail collapse [Çizmecioğlu, 2007].

Figure 2-5 Fracture of column, and insufficient reinforcement details in column-beam connection region [Çizmecioğlu, 2007]

The overlapping joint of longitudinal reinforcing bars should preferably be made in the mid height of the storey. Joints made at the storey level would suffer the highest torque. In TRNC the length of overlapping mostly is shorter than demanded by the legislation; in addition to that, those critical zones do not have sufficient reinforcing hooks. This is extremely wrong and dangerous. Overlapping joints of insufficient length which are implemented at storey levels are the biggest causes of damages [Doğangül, 2007].

If overlapping joints are to be made at storey level, each floor should not be more than 50% and adequate overlapping lengths are provided and over the length of overlapping sufficient reinforcement hooks are implemented. Buildings have to be able to absorb enough energy in order to withstand severe earthquakes. The absorption of energy depends on ductility, which itself depends on the reinforcing hooks at the edges of the elements.

- Special Seismic Hooks and Crossties

Hooks and crossties used in columns, beam-column joints, wall end zones and beam confinement zones of all reinforced concrete systems of high ductility level or nominal ductility level. All seismic zones shall have special seismic hooks and special seismic crossties for which requirements are shown in Figure 2.5.

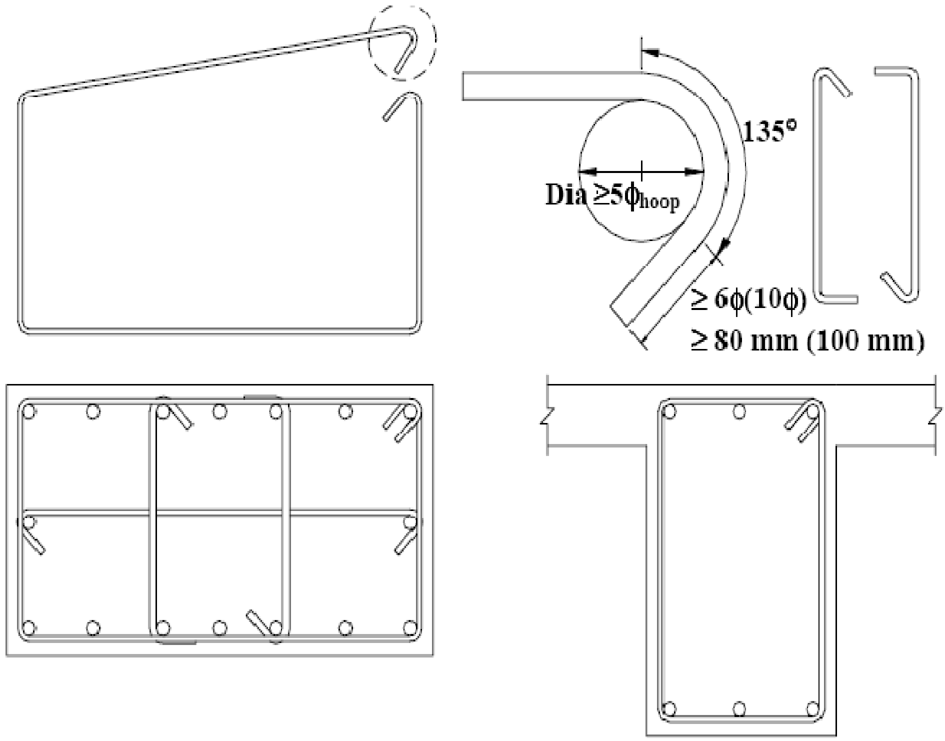


Figure 2-6 Special seismic hooks and special seismic crossties [TEC 2007]

CHAPTER 3

DETERMINATION OF MATERIAL CHARACTERISTICS AND BUILDING GEOMETRY

3.1 Introduction

In general, information gathered before the earthquake generates the required basic information for the assessment of the building's performance under the effect of an earthquake which is expected to take place in future. Information can be gathered for a building which has encountered the earthquake. However, the purpose here is not the determination of the damage. Occurrences of damages during the earthquake certainly make it easier to get to know the building, to identify the behavior of the building and at the same time provide significant contributions to the information compiled from the building. Besides, these damages reveal the weaknesses of the building and lead the way for the purpose of and design and strengthening. Further to that, damage determination is a process which is done with different purposes and different methodologies.

Information to be collected from the building is decisive for the configuration of analytical structure model to be prepared for the performance assessment of the building and for the assessments of the performance calculation results. As stated in Chapter 7 of 2007 TEC 2007, the scope of the information to be collected should at least be sufficient to be able to prepare the structural model. For that purpose, it is required to determine the structural system of the building, to measure material characteristics and to ascertain reinforced concrete details.

The information to be obtained from a building having its information gathering done must be sufficient to be able to reach the below listed results within the framework of Chapter 7 of TEC 2007.

1. Determination of information level (limited, moderate or comprehensive).
2. Determination of the properties of concrete and reinforcing bars.
3. Calculation of elements' critical cross-section strengths (bending, shear).
4. Determination of size, location and number of reinforcing bars in sections.
5. Determination of failure types of reinforced concrete elements.
6. Determination of bending and wrapping reinforcement amounts and their details which are required for the determination of elements' damage limits.

3.2 Building Geometry

Building geometry, with its meaning used here, contains dimension and function information belonging to load-bearing system and architectural characteristics of the building. Building geometry determination covers the entire system and element dimension determinations to be used for the analytical modeling of the building either for the existing state or in the strengthened state. Architectural and static building survey is composed by the system planning of each floor's architectural and load-bearing system and critical cross-sections. Processing the architectural and static building surveys on the floor plans which integrate each other makes it easier to comprehend the building and to determine the options of the strengthening. For that purpose, the frame axes are defined on both of the building survey plans and the clearances of the axes are indicated. Dimension of partition walls, parapets, window and door spaces are shown on the architectural building survey by giving their dimensions. Wetted area, area functions and floor pavement materials are entered in the plan.

On the other hand, entire characteristics of the building load-bearing system are indicated in the static building survey. Places and dimensions of the columns, load-bearing walls and beams, floor coverings, thickness of the floor coverings and floor covering holes are defined by using a coding system. Floor plans obtained with the building survey basically contain the entire information and details taking place in the floor plans which are drawn in the project of a new building. However, it is not required to have information on architectural details in the floor plan which do not cause load on the building and do not have effect on the load-bearing system. Building survey is implemented at two stages as taking the measurements on the field and converting the plan and cross-sections into modeling and main drawings. Optical or mechanical length measuring tools are used in the field in order to take measurements.

If the building already has its architectural and/or static projects, this situation provides convenience for the building survey work and at the same time it increases the sensitivity of the works to be done. In the event of having the project ready, building status is determined whether it is in its as-built designed status or it is in a different state. If the projects are not available, building survey work will be more detailed [Sucuoğlu, 2008].

3.3 Material Characteristics

Essential structural materials in a reinforced concrete building are concrete and reinforcement steel. Strengths and the distribution of these strengths within the building must be known for both of these materials. For this purpose, there are destructive and non-destructive examination methodologies.

3.3.1 Determination of Concrete Characteristics

Since the concrete of the existing buildings is generally produced in the worksites and their placements and curing are not implemented perfectly, their strengths can be different than the ones foreseen in the relative projects. Ascertaining the strength of the concrete can be done by applying destructive and non-destructive methodologies. There are many kinds of experiments in non-destructive methodologies. The most commonly used ones are: rebound hammer (Schmidt Hammer) and ultrasonic pulse velocity. Destructive type test is generally the concrete cores taken from building.

Rebound hammer is a tool which is very much preferred because of its low cost and convenience of using. When this hammer is applied for reading purposes, the plaster taking place on it's application surface should be lifted and the surface of the concrete must be brought to smooth state by grinding (Figure 3.1) [Neville, 2003]. The reading obtained with the rebound hammer shows the hardness of the concrete's surface. The readings are then converted to equivalent cylinder compressive strength by using a calibration curve depending on the characteristics of the hammer.

Figure 3-1 Reading the surface hardness value with Schmidt Rebound Hammer

On the other hand, in the experiments done by using ultrasonic devices, determination of the sound waves passing from the inside of the concrete is done. It has been proved by many researches done in the past that this measurement has direct relationship with the modulus of elasticity of the concrete [Sucuoğlu, 2008]. By utilizing the empirical relationships between the modulus of elasticity of the concrete and compressive strength, the strength of the concrete is obtained from the measured speed of the sound. The costs of the required devices to implement these kinds of experiments are comparatively higher than the cost of the rebound hammer. Besides, the applications of the experiments are considerably slow and troublesome.

Core sample taking and core samples test are the most commonly used ones among the destructive methodologies to obtain the strength of the concrete [Figure 3.2]. During the selection of the places to take the cores, care must be taken that the core should represent the strength of the concrete in a good way and it should not cause any decrease in the strengths of the constructional elements. Cores should not be taken from the zones of the reinforced concrete elements which encounter high-pressure tensions. Efforts must be given not to cut the reinforcement as much as possible when the core is taken and the natural humidity of the samples must be maintained until the experiment day [Neville, 2003].

Core samples are examined in the laboratory one by one and existing cracks, voids and reinforcement diameter, if any, and their locations must be noted and all of these should be taken into consideration during the evaluation of the results. Cores should be tested under the axial pressure after making the capping and their strengths and crushing forms should be entered in the records. It is required that the void of the place from which the core is taken, must be filled with high-strength repair mortar. The most important point to be emphasized here is the fact that the concrete characteristics determined with this methodology have considerably high reliabilities.

Figure 3-2 Obtaining cylinder concrete core sample from column

Having very low concrete quality in the existing buildings and having too much of alterations within the structure and even within the element can create too much of differences between the concrete strengths determined by destructive and non-destructive methods. For this reason, in TEC 2007, it has become compulsory to determine the existing concrete strength by only applying core sampling methodology. Minimum numbers of core samples to be taken are nine by having at least three samples from each floor. In order to determine the variability of concrete strength in the building, application of rebound hammer methods adapted with core experiments are proposed in the Regulation. Rebound hammer readings should be done for the elements from which the cores are taken and at least 10-12 hammer readings should be done for the elements from which the cores are taken and the average values of these must be used in the adaptation process [TEC 2007].

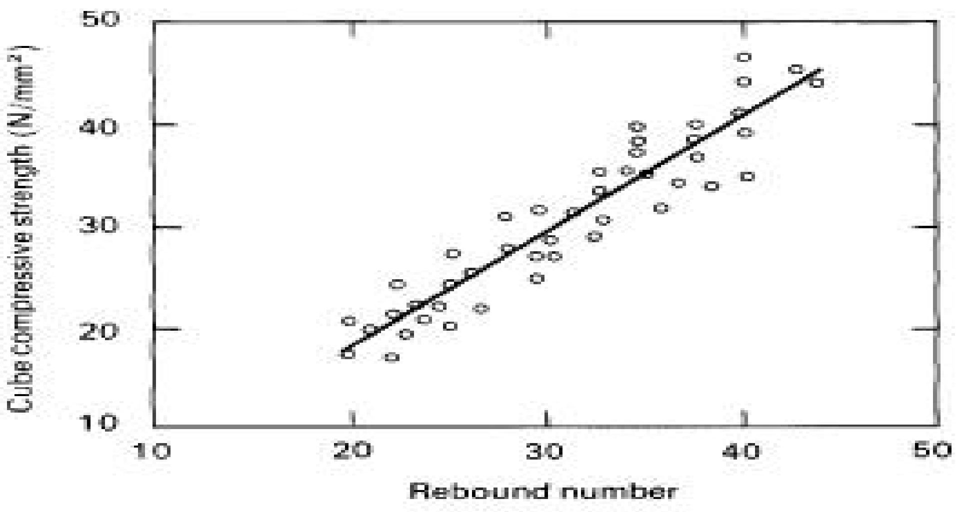


Figure 3-3 Adaptation of Rebound Hammer Readings with Core Experiment Results [Bungey and Millard, 1996]

In general, it is more secure to have the target (median-standard deviation) value instead of the median of the adaptation function obtained with longitudinal regression analysis Figure 3.3 [Bungey and Millard, 1996].

3.3.2 Determination of Reinforcement Characteristics

Cross-section and reinforcement details of the reinforced concrete elements contain important information and details which are required for the determinations of elements' bending and shear strengths and deformation characteristics. Among the main items of this issue, longitudinal reinforcement number and diameter, coupling length in longitudinal reinforcement or hook detail, the place of the lapping zone and lapping length, lateral reinforcement amount, hook characteristic of lateral reinforcement or earthquake stirrup, the thickness of the concrete covering and the effects of the corrosion in the reinforcements can be listed.

It is possible to use destructive methodology as well as non-destructive methodology in the determination of reinforcement details. Non-destructive methods are able to present new possibilities to the engineers being parallel to the new technological developments taking place in this area [Neville, 2003]. Ferroscan device determines the place, diameter of the reinforcement bar which is buried in the concrete and also determines the thickness of the concrete covering. In the event of having the reinforcement being buried more than 10 centimeters; it should be taken into consideration that the sensitivity of the device will be less. In despite of the fact that this device provides reliable information regarding the place and the number of the reinforcement, their reliabilities in terms of diameter determination are less. It is very easy to use these devices and it is possible to collect rather detailed information about the reinforcement.

Ferroscan or reinforcement scanning devices specially provide reliable information for the determinations of the reinforced concrete curtain or reinforcing meshes having the location of being parallel to wide column surfaces. Ferroscan device consists of two components as one scanner and one recorder. Scanner sends the information of the reinforcement grid taking place underneath of the scanned area to the recorder. Recorder processes this information and then calculates the place, diameter and depth of concrete cover of horizontal or vertical reinforcement on every point of the defined coordinate taking place on the reinforcing mesh (Figure 3.4).

Figure 3-4 Scanning and recording components of a ferroscan

Reliability level has not been achieved yet in the topic of determining the details of the reinforcement by using non-destructive methods. For this reason, in order to determine the details of the elements in TEC 2007, compulsory conditions has been brought to examine buildings by using destructive methodologies on lesser amount of elements and on the contrary, it was requested to obtain supporting data by applying non-destructive methodologies to be practiced on more number of elements [Sucuoğlu, 2008]. Removing the depth of concrete cover is enough to determine the diameters of longitudinal and latitudinal reinforcement diameters in columns and beams, their gaps and lapping length and the details of the hook. In the zones having the concrete cover removed, the estimations were done with naked eye that would give most reliable results. A beam and column having their concrete cover removed for the purpose of examining are shown in Figure 3.5.

In the event of extreme corrosion leading to flaking in the diameter of the reinforcement in the examination carried out, the reduction occurring in the diameter of the reinforcement due to corrosion should definitely be determined. In these kinds of situations, it should not be forgotten that the adherence between the reinforcement and concrete will be decreased to a considerable extent. If hooked coupling has been done in the rusty reinforcement, it will be enough to consider the reduction in the diameter of the reinforcement to be caused by the rust. If straight coupling has been done in the rusted reinforcement, it will be appropriate to make assumptions in the calculation of the element's capacity to remain on the secure direction.

Figure 3-5 Determination of reinforcement details in columns and beams by removing the concrete cover

Figure 3-6 Determination of reinforcement bars in column and beam section by removed concrete cover

Steel reinforcement is a material produced considerably in a standard for the variability in its material characteristics is less. As can be seen in Figure 3.6, examination done with naked eye by removed element's concrete cover, reinforcement class can easily be determined (S220, S420, straight reinforcement, ribbed reinforcement). In cross-section calculations, it is suitable to use characteristic yield strength belonging to the relevant steel class as the safe value for the strength of the steel (220 or 420 MPa). However, in the event of having a comprehensive examination in the building, this is not sufficient. Preferably, the sample of reinforcement must be taken from a zone such as the basement curtain in which the reinforcement is not forced and the hammer test should be carried out in the laboratory. Steel yield and shear strengths obtained from the pull-test and deformation characteristics will provide the determination of the steel class [Sucuoğlu, 2008].

CHAPTER 4

SEISMIC PERFORMANCE ASSESSMENT

4.1 Introduction

With the existing scope, basic purpose of structure assessment methodologies is to have the estimation of the performance for the buildings to be strengthened under a foreseen earthquake with various analysis methodologies. It is required that foreseen strengthening strategy should have the qualification to meet the target performance level. Consequently, the assessments based on the performance are done by comparing of forces and deformations with some limit values. In this chapter, analysis of strengthened structures under the effect of a foreseen earthquake and the general approach taking place in the methodologies proposed by the literature for the assessment take place. In general, in spite of concentration on reinforced concrete type buildings, the methodologies taking place here are general and they can also be applied to the other structural systems.

Earthquake hazard calculation provides usage of determination of the ground motion to which a building is located in a geographical region. Determination of the effects created by this motion on the building and probable damage amount to be caused requires the implementation of a separate assessment [Yakut, 2008].

In standards of many countries, linear static and non-linear static (displacement-based) methods and performance assessment methods are mentioned. In this thesis, ATC-40 (Applied Technology Council 1996), FEMA-356 (Federal Emergency Management Agency 2000) and TEC-2007 (Turkish Earthquake Regulation) are explained briefly. By performing the analysis in order to determine structural systems design, assessment and capacities, internal forces and deformations are calculated. Sizing or capacity control is done by depending on these determined forces and deformations. Broadly the methods of analysis are as follow:

- Linear Analysis
  - Static Analysis (Linear Performance Analysis)
  - Dynamic Analysis
- Non-Linear Analysis
  - Static Analysis (Pushover Analysis)
  - Dynamic Analysis (Time-History Analysis)

It is possible to divide calculation methodologies into two groups as linear and non-linear as listed above. Both of these groups are divided into two sub classes as static and dynamic, since the most commonly used ones among these methodologies are linear and non-linear static methodologies. Linear methodologies are the ones taking place in most of the regulations and they are the most commonly used equivalent static load methodology, mode superposition using response spectrum and time history analysis. On the other hand, non-linear calculation methodologies are pushover analysis and non-linear time history analysis.

Some of the ready programs which can perform ``Pushover analysis`` are able to calculate plastic hinge characteristics by itself. For this reason, it is required that the user would define cross sectional dimension and reinforcement details. IDE-CAD 5.511 program is able to calculate load-deformation relations by using defined cross section and material characteristics for the desired plastic hinge type. These relations are defined in IDE-CAD 5.511 as they are proposed in FEMA 356.

4.2 Non-Linear Analysis

A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic non linear force displacement relationship can be determined [Kadid and Boumrkik, 2008].

According to this methodology, a sample relation of systems base shearing force - top point displacement is as shown in Figure 4.1. This curve represents the building’s behavior of the structure under the increasing base shearing force. In other words, vertical axis of the curve reflects different earthquake effects while the horizontal axis shows the deformations corresponding to these effects. Consequently, the curve remains straight under low earthquake effects and represents structure’s elastic behavior.

On the other hand, under the increasing earthquake effects, load deformation relation changes because same elements will reach their elastic capacity and yield, thus the structure will go beyond the elastic limit. The last point of the curve corresponds to the status of the structure just before the collapse. This curve is actually a curve representing the capacity of the structure rather than the effect of a certain earthquake.

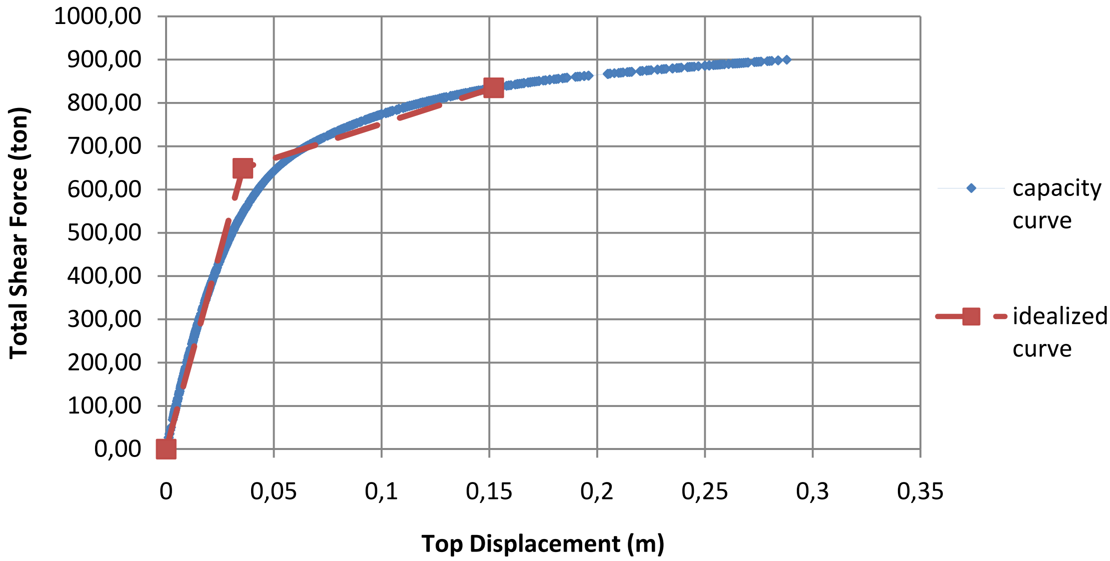


Figure 4-1 Capacity curve and idealized curve

Pushover analysis can be described as the structure's incremental analysis under changing statically equivalent earthquake loads. For each load increase, analysis is continued by comparing the internal forces as linearly calculated on the level of the element with the capacities of the element by changing the rigidity and strengths of the elements which exceed the capacity.

- Pushover analysis consists of the following steps:

Firstly computer model of the structure is configured, element loads are defined. Then vertical loads are applied on the structure by being in compliance with earthquake effect, linear analysis is done by applying incremental horizontal loads on the structure. Calculated base shear force, top point displacement, element's internal forces and nodal point displacements are recorded.

In Non-Linear analyses, element characteristics are represented with the plastic hinges which are defined on the element. Plastic hinge characteristics defined on the end points of the bar elements, are defined by being dependent on the force and deformations exposed during the analysis. Bending for the beams and axial load-bending interaction plastic hinges must be defined for the column and shear walls. However, in the case of having the dominancy of the shear force, plastic hinges having the characteristics of shear force-deformation must be defined [Ersoy, 2007].

Force deformation relations of the structural elements are very important in terms of structural mechanics and design. Deformations of the elements exposed under the forces are the most important indications to be used for the determination of the damage. Force-deformation relations changes according to type of the material and load effect. The experimental results indicate that cycle force-deformation behavior of a structure depends on the structural material and the structural system [Chopra, 2001].

These relations can be taken for the assessment of reinforced concrete structures as proposed by FEMA 356. Element load-deformation relations are determined with the assistance of linear lines formed by four parts as shown in Figure 4.2. Linear behavior are valid between point A and point B the part taking place between points B and C representing the rigidity which shows the element behavior beyond the linear limit. Curve CD shows instant drop in the strength and curve DE represents the limited capacity left behind and reflects reinforced concrete elements' non-linear behavior. The slope of the curve taking place between points B and C can be defined as 10% of the beginning rigidity (AB slope). Vertical coordinate of point C represents element's strength capacity and its horizontal coordinates represent the deformation where a decrease starts to take place. Point B corresponds to yield value of the element. Here, *a* and *b* values show the rotation of the beam. For reinforced concrete beam elements, these values are given as: *a*= 0.02, *b*= 0.035 and *c*= 0.2. In other words, when plastic rotation taking place at the end point of the beam reaches the value of 0.02, element will reach moment strength and when the rotation is 0.035, it will lose the bearing capacity completely [FEMA 356].

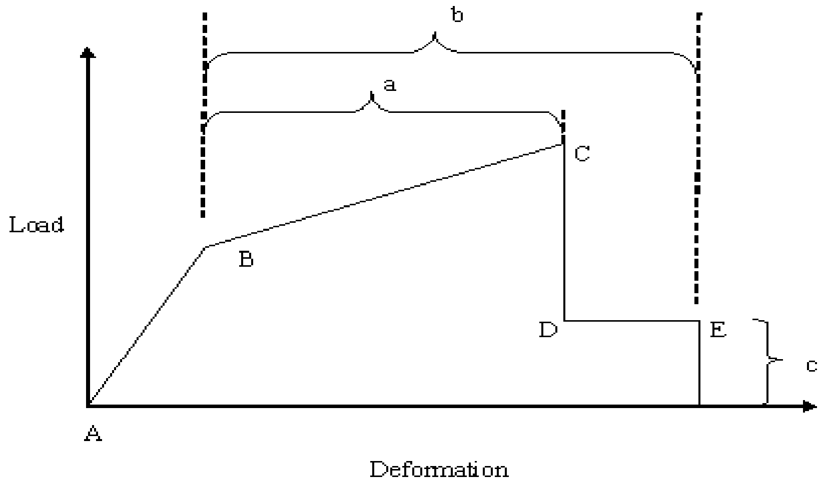


Figure 4-2 Generalized load-deformation relation [FEMA 356]

Load-deformation relation is obtained from the result of cross section calculations to be done for the elements and this is the healthiest way of doing this. Moment-rotation relations are calculated from the conversion of moment-curvature relations obtained from the analyses of cross section. When these conversions are done, moment-curvature relation is defined with two lines as elasto-plastic. In other words, non-linear moment-curvature curve is represented with a line up to the yield point and represented with another line between the yield point and collapse point

(Figure 4.3). In non-hardening moment-curvature relations, yield moment (***M_y***) is equal to cross section's moment capacity (***M_u***) [Ersoy and Ozcebe, 2004].

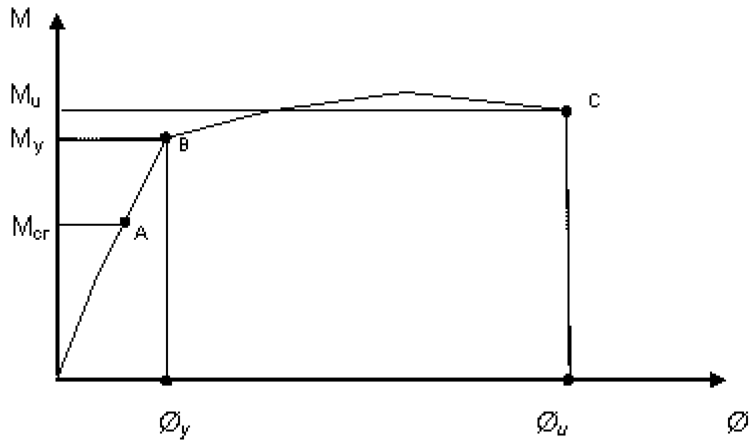


Figure 4-3 Moment-curvature relation [Celep,2008]

For converting moment-curvature diagram to moment-rotation relation, it requires to have reference to some basic information. In the relation shown in Figure 4.3, A shows concrete cracks, B shows the curvature of tension reinforcement's corresponding to the yield point. On the other hand, ***EI*** represents the rigidity of the cross section, which is calculated from the slope of the curve between the first yield point and the origin (equation 4.1). Curvature values are calculated from the cross section unit deformation distribution taking place in Figure 4.4 and are calculated from the equations given by 4.2 and 4.4 [Celep, 2008].

$$EI = M/\emptyset \tag{4.1}$$

$$\emptyset_y = \frac{\varepsilon_y}{d - c_y} \tag{4.2}$$

$$\emptyset_{ult} = \frac{\varepsilon_{cu}}{c_u} \qquad (4.3)$$

Where ***d*** is effective depth of section, ***C*** depth of neutral axis, ***EI*** is bending rigidity.

Since the behavior up to the yield limit is linear, when the rotation values are obtained from cross section curvature values, curvature values also change linearly throughout the element. Plastic hinge begins on the first place where the moment value on the element exceeds yield moment value. Since the progress of the plastic hinge on the element shall be up to a certain place, it is accepted that the plastic hinge on the limit state is expanded along the length of $\boldsymbol{l_p}$. This length is defined as plastic hinge length and it shows variations by being dependent on various parameters.

Öztemel (2003), by referencing to Paulay and Priestly (1992) outlined that, this variation for the reinforced concrete columns and beams usually takes place between the half of the dimension of cross section's curative direction and cross section's efficient depth (equation 4.4). Predominantly, for the bar elements being forced under the effect of bending moments, plastic hinge length, $\boldsymbol{l_p}$, is usually accepted being equal to half of the dimension of cross section's loading direction.

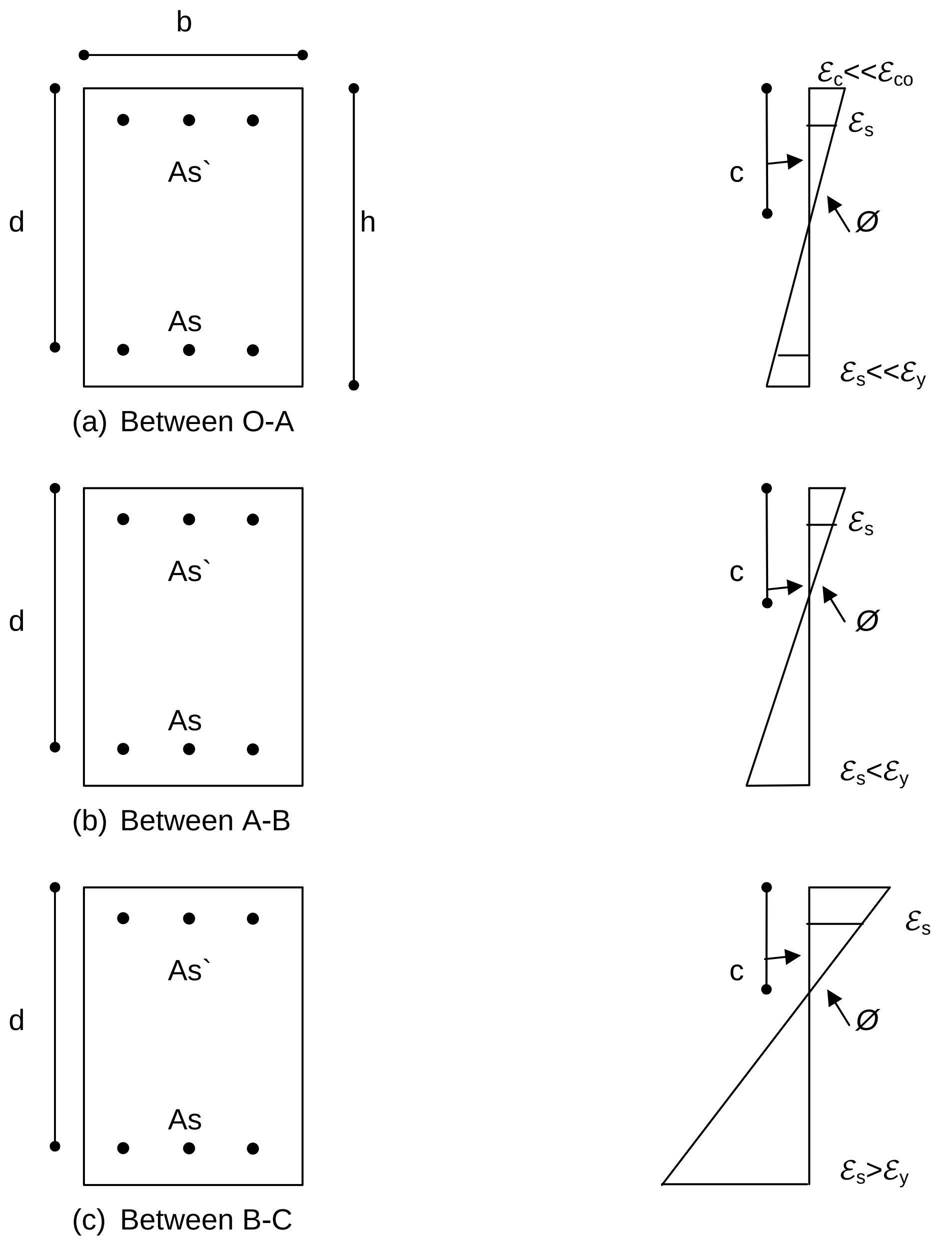


Figure 4-4 Cross Section Changing Distribution. a, b and c values were represented in Figure 4.3 [Celep,2008]

$$l_p = 0.5\ \text{h} \tag{4.4}$$

Where, $\boldsymbol{h}$ is the height of section.

The plastic rotation occurring along l_p is calculated from expressions stated below. It is assumed that calculated plastic rotation's value ($\emptyset_p$) is accumulated in the middle of l_p length. In reinforced concrete sections, plastic rotation capacity depends on some parameters as mentioned above. These are noted as follows;

i. The strain values, ε_{co}, ε_{cu}, ε_{sy} , and ε_{su} that identify the properties of $\sigma-\varepsilon$ relation.

   Where, ε_{cu} is ultimate strain of concrete, ε_{co} is strain at optimum stress level of concrete, ε_{cu} is yield strain of steel, ε_{su} is ultimate strain of steel.

ii. Amount and detailing of confinement.

iii. Amount and detailing of main (longitudinal) reinforcement.

iv. Dimensions of the section and the distance from the plastic section to the point of contra-flexure (l_c). These parameters are used to determine the length of the plastic section (l_p) [Öztemel, 2003].

$$\emptyset_p = (\emptyset_u - \emptyset_y) l_p \tag{4.5}$$

Model configured for non-linear analysis is done under the effect of the force distribution (Pushover Analysis). Most sensitive point of this analysis methodology is the distribution applied on the structure. General approach in this topic is to apply a horizontal load distribution which represents the structure's first mode. Since the load distribution corresponding to equivalent static load method is also based on this idea, this distribution can also be used. Structure is exposed to deformation until demolishing mechanism occurs to symbolize the collapse under the determined load distribution and by this way a capacity curve is obtained. Capacity curve is the one expressed by drawing structure's base shear force and top displacement reciprocally.

Capacity curve exhibits structure's behavior under different earthquake loadings. Consequently, there is the need for intermediary methodologies for the calculation of the effects representing certain earthquake loading. For this reason, some methodologies have been developed described in ATC-40 (In 1996, ATC published the ATC-40 report, Seismic Evaluation and Retrofit of Concrete Buildings) and FEMA 356 (Seismic Rehabilitation Pre-standard was published in 2000) and used commonly. FEMA 356 uses a procedure known as the Coefficient Method, and ATC-40 details the Capacity-Spectrum Method These methodologies are described below.

4.2.1 Capacity Spectrum Method (ATC-40)

Non-linear static procedures are employed in this document and the Capacity Spectrum Method is highlighted, since linear static procedures can not predict failure mechanism after first yield. Capacity-Spectrum Methodology aims the representation of a foreseen earthquake loading as a reaction spectrum and to calculate the effects which may occur on the structure approximately by comparing this spectrum with the capacity curve. In this approach, multi-degreed structure is degrade to a single degree system and then the earthquake effect in question is specified with it's equivalent of loading stage in the occurrence of capacity curve. The point taking place on the structure's capacity curve corresponding to this stage is defined as the performance point. Then, the results of the effect analysis corresponding to this point are scrutinized and the internal forces and deformations occurring on the elements are calculated.

Members are classified as brittle and ductile. If shear demand of any member exceed its shear capacity, member is defined as brittle, otherwise it is defined as ductile. For ductile members plastic rotation capacities are determined from moment – curvature relations. Plastic rotation capacities are calculated by multiplying the plastic curvature capacity with plastic hinge length. For ductile members, plastic rotation demands are calculated by the following steps. A stage of capacity-spectrum methodology is given below with its details:

1st Stage: Earthquake is shown with a spectrum calculated according to 5 percent damping (Elastic Response Spectrum). Spectrum is the indication of the demand to be created by the ground motion.

2nd Stage: Calculated spectrum is converted to ADRS (Acceleration Displacement Response Spectrum) (Figure 4.5). In other words, it is required that spectral acceleration-period graphic must be converted to spectral acceleration-spectral displacement. This process is done with equation given in equation 4.6;

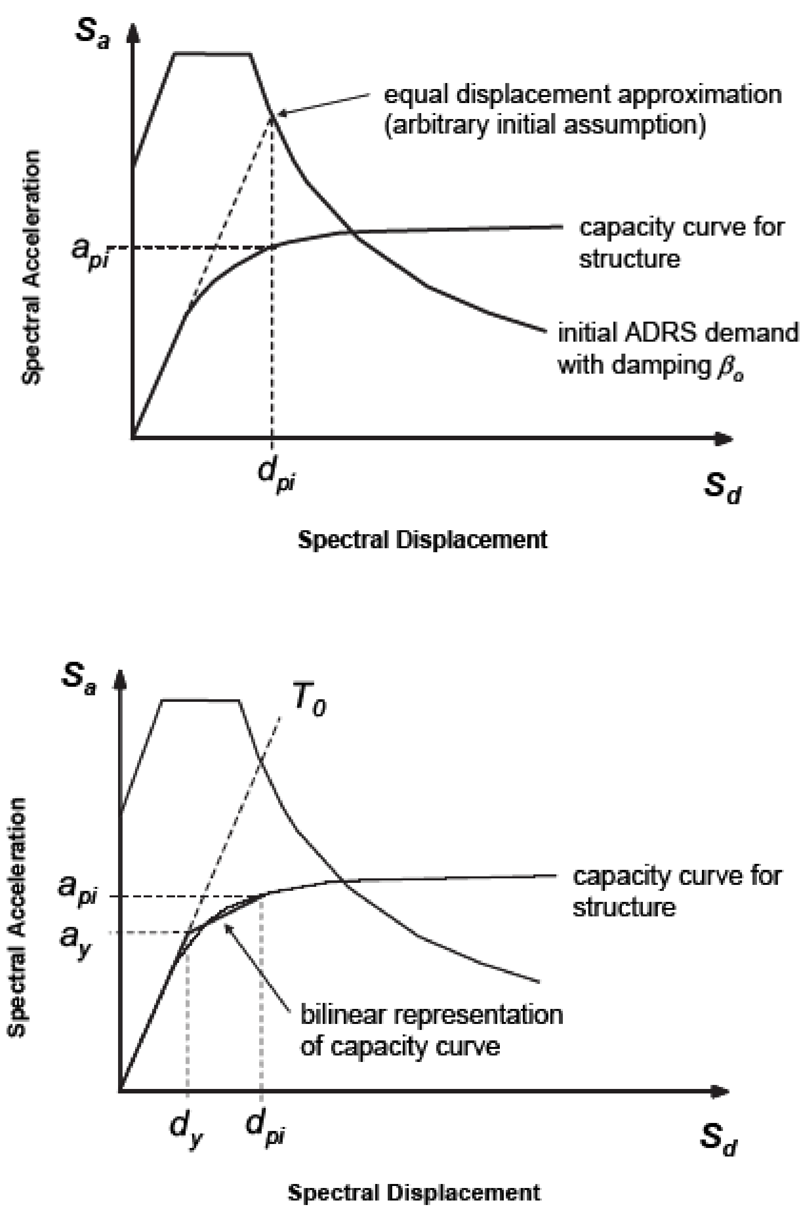


Figure 4-5 S_a and S_d relation in capacity spectrum methodology [FEMA 440]

$$S_d = \frac{T_i^2}{4\pi^2} S_a \qquad (4.6)$$

3rd Stage: Structure's capacity curve is obtained from Pushover Analysis. Below stated conversion is done in order to bring the capacity curve to a form being similar to the above stated spectrum.

$$S_a = \frac{V/W}{\propto_1} \tag{4.7}$$

$$\propto_1 = \frac{[\sum_{i=1}^{N} w_i \emptyset_i]^2}{w \sum_{i=1}^{N} w_i {\emptyset_i}^2} \tag{4.8}$$

$$S_d = \frac{\delta_{roof}}{\propto_2} \tag{4.9}$$

$$\propto_2 = \emptyset_{\mathrm{roof}} \frac{\sum_{\mathrm{i=1}}^{\mathrm{N}} \mathrm{w_i} \emptyset_{\mathrm{i}}}{\sum_{\mathrm{i=1}}^{\mathrm{N}} \mathrm{w_i} {\emptyset_{\mathrm{i}}}^2} \tag{4.10}$$

Where S_a, spectral acceleration, S_d, spectral displacement, V, pushover base shear at δ_{roof} , δ_{roof} , pushover curve displacement, $\propto_1$, fraction of mass in pushover mode, $\propto_2$, ratio roof/pushover mode displacement, $\emptyset_i$, pushover mode shape at location i, $\emptyset_{roof}$, pushover mode shape at roof, w_i tributary weight at location i, W , total weight of the structure [Barros and Almeida, 2005].

When the displacement increases, so does the period. On the other hand, effective damping also increases the force acting on the structure decreases. In capacity spectrum methodology, the point taking place on the intersection demand curve (representing the decreased force) with the capacity curve is named as the performance point.

4th Stage: Elastic response spectrum which is converted to ADRS format is drawn in the same graph. Elastic calculation spectrum is decreased by considering the effective damping increases to be occurred because of the increasing displacements. The coincide of this with the capacity of the building is named as the ``performance point``.

Damping occurring when the structure's plasticity is forced beyond the limit is a kind of damping having viscous (proportioned with the speed) is lesser part while the major part is hysteretic kind. Below stated expression is written with the assumption adding these two types of damping:

$$\zeta_{effective} = 0.05 + K\zeta_o \qquad (4.11)$$

Here, ζ specifies the proportion of damping, $\boldsymbol{K}$ is a correction factor and $\boldsymbol{K\zeta_o}$ is an equivalent damping which is useful to express consumed energy by reverting repeatable load conversions.

$$\zeta_o = \frac{1\ (Crossed\ area)}{4\pi\ \ (Trapese\ area)} \qquad (4.12)$$

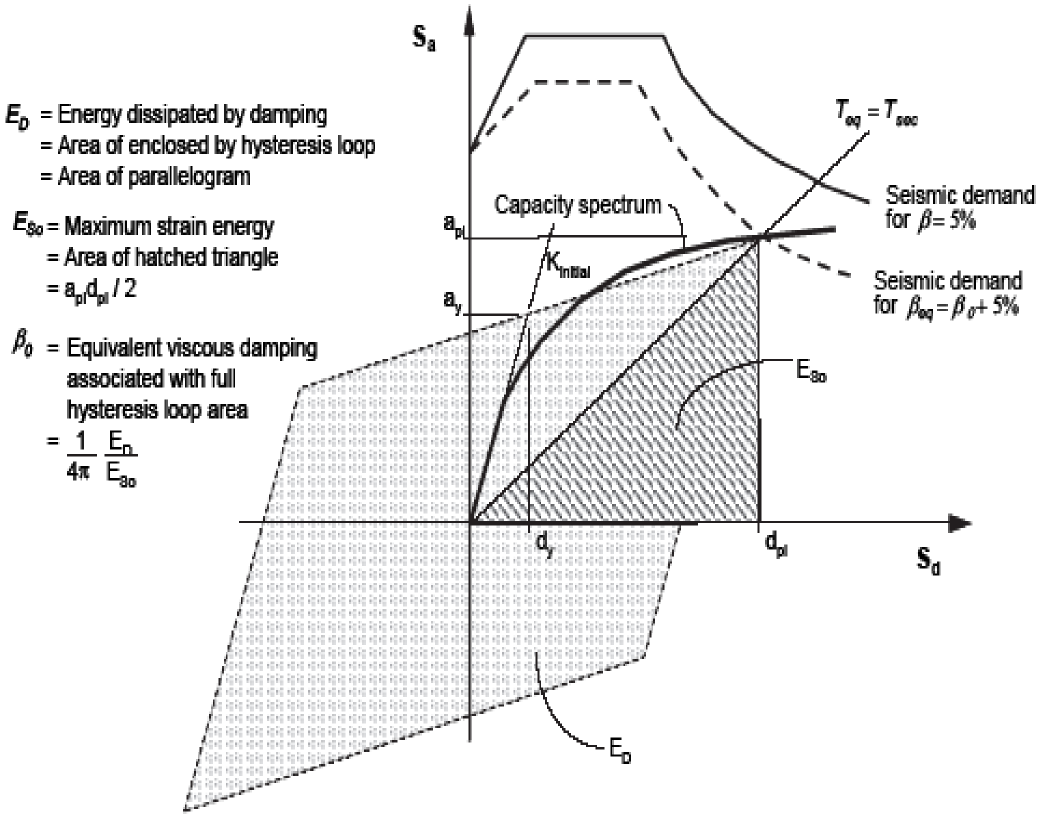


Figure 4-6 Graphical representation of the capacity-spectrum method of equivalent linearization, as presented in ATC-40

4.2.2 Displacement Coefficient Methodology (FEMA 356)

Another procedure besides the Capacity-Spectrum methodology is the Displacement Coefficient Methodology which is described in FEMA 356. This procedure is proposed to calculate a performance point by using elastic spectrum with capacity curve. Building's top point displacement (**δ_t**) corresponding to performance point is calculated using the relation given below.

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{4.13}$$

Here,

C_o = The coefficient correlating the displacement calculated for equivalent single degree system with structure's top point displacement. This coefficient can be obtained using the result of modal analysis and the values taking place used depend on number of floors of the building [Table 4.1].

C_1= Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response:

$$1.0 \quad \text{for } T_e \geq T_s \tag{4.14}$$

$$\frac{\left[1.0 + \frac{(R-1)\ T_s}{T_e}\right]}{R} \quad \text{for } T_e < T_s \tag{4.15}$$

Table 4-1 Values for modification factor C_o

	Shear Buildings		**Other Buildings**
Number of Stories	**Triangular Load Pattern**	**Uniform Load Pattern**	**Any Load Pattern**
1	1.00	1.00	1.00
2	1.20	1.15	1.20
3	1.20	1.20	1.30
5	1.30	1.20	1.40
10	1.30	1.20	1.50

T_e=Effective fundamental period of the building in the direction under consideration, in seconds.

T_s= Characteristic period of the response spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

R= Ratio of elastic strength demand to calculated yield strength coefficient which in calculated by the following Equation 4.16.

$$R = \frac{S_a}{V_y/W} C_m \tag{4.16}$$

C_2= Modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. Alternatively, use of C_2 = 1.0 shall be permitted for nonlinear procedures.

Table 4-2 Values for Modification factor C_2

	T<0.1	**T<0.1**	**T>Ts**	**T>Ts**
Structural performance Level	**Framing Type 1**	**Framing Type 2**	**Framing Type 1**	**Framing Type 2**
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

C_3= Modification factor to represent increased displacements due to dynamic P-Δ effects. For buildings with positive post-yield stiffness, shall be set equal to 1.0. For buildings with negative post-yield stiffness;

$$C_3 = 1.0 + \frac{a\,(R-1)^{3/2}}{T_e} \tag{4.17}$$

S_a= Response spectrum acceleration, at the effective fundamental period and damping ratio of the building in the direction under consideration, ***g***.

g = acceleration of gravity

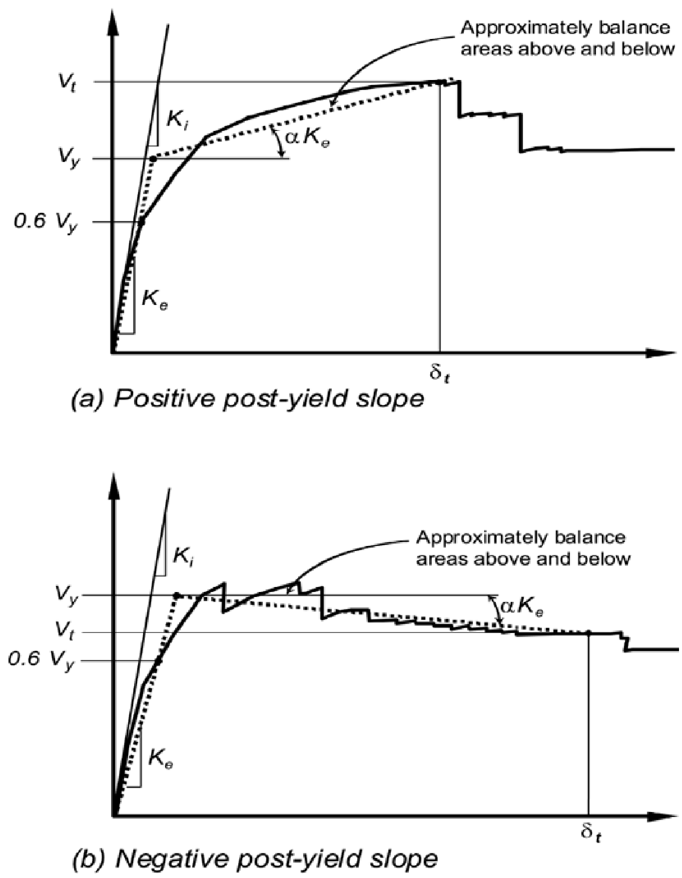


Figure 4-7 Idealized force-displacement curves [FEMA 356]

The effective fundamental period in the direction under consideration shall be based on the idealized force displacement curve. The effective fundamental period, shall be calculated in accordance with equation 4.18:

$$T_e = T_i\sqrt{\frac{K_i}{K_e}} \tag{4.18}$$

T_i= Elastic fundamental period (in seconds) in the direction under consideration calculated by elastic dynamic analysis

K_i= Elastic lateral stiffness of the building in the direction under consideration

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

4.2.3 Displacement Demand using the 2007 Turkish Seismic Code

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

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

$$S_{de1} = \frac{S_{ae}}{(\omega_1)^2} \tag{4.19}$$

S_{ae} : Elastic spectral acceleration of the corresponding period,

ω_1 : Frequency of corresponding period.

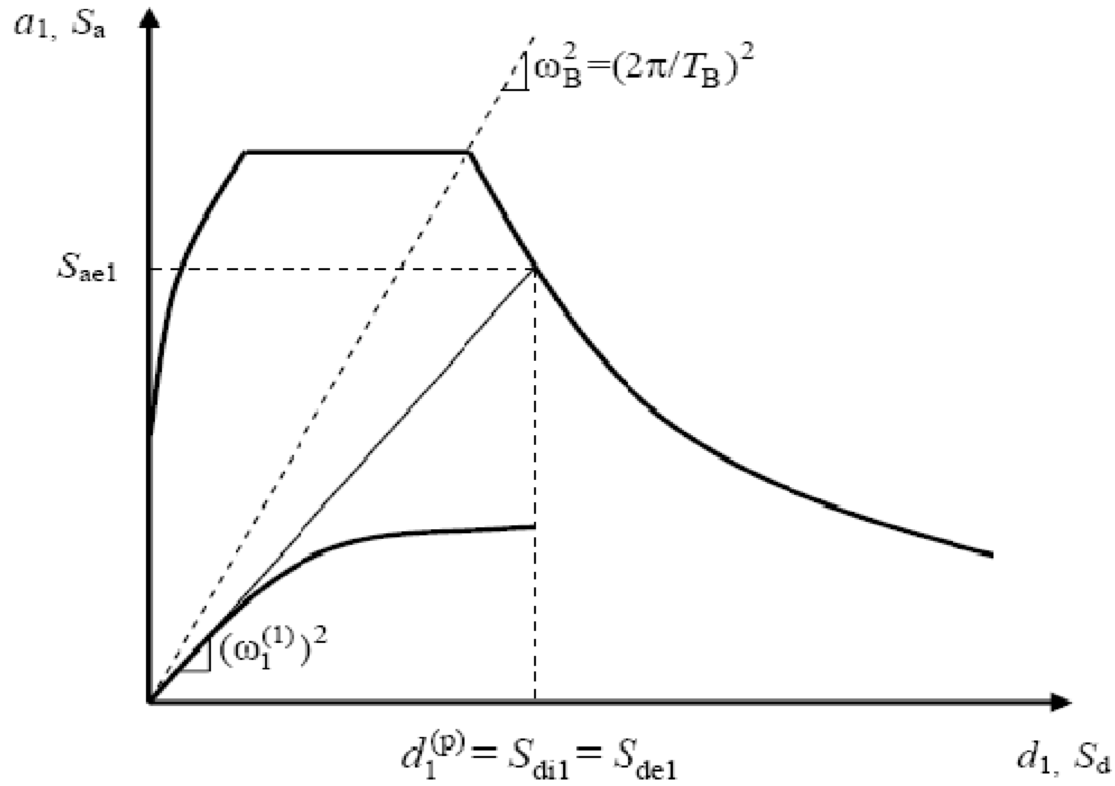


Figure 4-8 Capacity and demand curve for T ≥ T_B

2. If the period of corresponding direction (***T***) is greater than the characteristic period of acceleration spectrum (***T_B***), inelastic spectral displacement is assumed to be equal to elastic spectral displacement. [Figure 4.8]

$$S_{di} = S_{de} \tag{4.20}$$

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

$$S_{di1} = C_{R1} * S_{de1} \tag{4.21}$$

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

$$C_{R1} = \frac{1+(R_{y1}-1)T_B/T_1}{R_{y1}} \geq 1 \tag{4.22}$$

$$R_{y1} = \frac{S_{ae1}}{a_{y1}} \tag{4.23}$$

4. For first iteration $\boldsymbol{a^0_{y1}}$ is calculated as shown in Figure 4.9(a), by equal area principle, i.e. the area under the capacity curve and over the capacity curve are equal. For other iterations, $\boldsymbol{a_{y1}}$ is calculated from Figure 4.9(b) again by equal area principle, as well as $\boldsymbol{R_{y1}}$ and $\boldsymbol{C_{r1}}$ values.

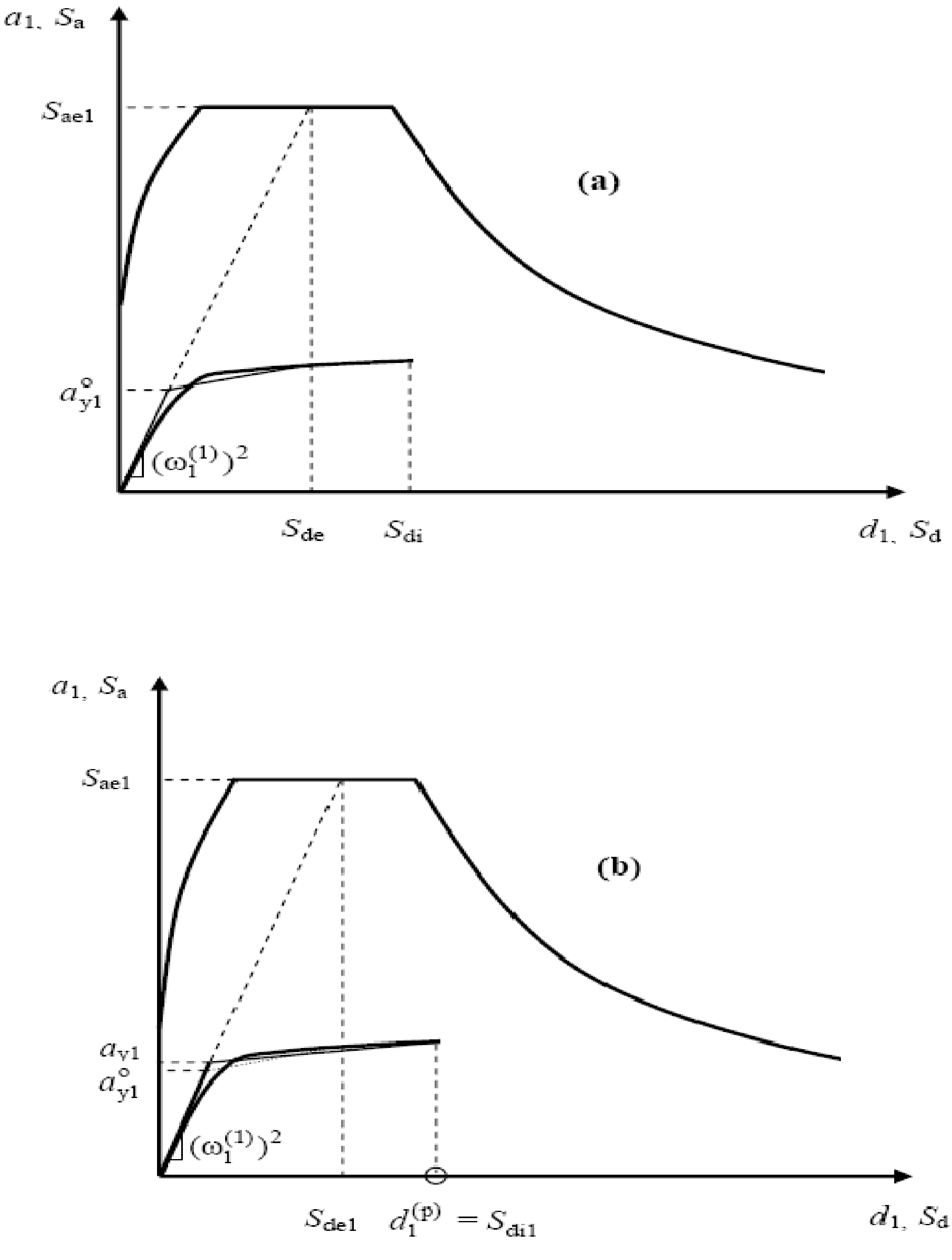


Figure 4-9 Calculation of a^{0}_{y1} and a_{y1}

5. The iterations are repeated until obtaining close results between iterations.
6. From last iteration, inelastic spectral demand is obtained (S_{die}) and converted to the inelastic displacement demand. This inelastic demand is called as target displacement [Düzce, 2006].

4.3 Linear Static Analysis (2007 Turkish Seismic Rehabilitation Code)

This calculation methodology is mostly used for force based assessment methodology. Equivalent static lateral force analysis and modal response spectrum analysis can be employed for performance assessment. Equivalent static lateral force analysis is limited to 8 story buildings with total height not exceeding 25 m, and not possessing tensional irregularity. For buildings with more than two stories, 85% of the total mass is considered in calculating the base shear force. Modal response spectrum analysis can be applied to all buildings without any restrictions. The signs of internal member forces and capacities under an earthquake excitation direction are taken as the signs consistent with the dominant mode shape in this direction [Sucuoglu, 2006].

The linear method can be regarded as an extension of the method used for the newly designed buildings to the existing buildings. The Code assumes a specific seismic load reduction factor R_a by requiring precautions for obtaining a structural system of high ductility. However, in existing building demand and capacity ratio of the cross sections evaluated and compared to their limiting values given in the Code [Uygun and Celep, 2007]. The main reason of the difference is due to variations of ductility in the members of the existing buildings. It is possible to ensure a specific ductility level in the buildings to be designed. However, one has to take into account the present level of ductility in the existing building.

Collection of the data to reflect the characteristics of the existing structure in a correctly and sufficiently is, without doubt, one of the most important components of structural resolution and assessment processes. In the light of these collected data, the capacities of the elements forming the structure will be determined and these capacities will be compared with the calculated effects later on [Sucuoglu, 2006].

In most of the detailed assessment methodologies proposed in the literature of FEMA 356 and TEC 2007, some classifications have been done depending on the scope and reliability. Mostly, three knowledge levels have been determined and a coefficient is proposed for each knowledge level (Table 4.3). Obtained knowledge level becomes effective in the determination of the calculation methodology to be applied.

Table 4-3 Knowledge Level Coefficient

Information Level	**FEMA 356**	**TEC 2007**
Limited	0.75	0.75
Moderate	0.75 or 1.00	0.90
Comprehensive	1.00	1.00

- **Limited knowledge level**

Structural plans are determined by field studies. The location of all structural members and partition walls are marked on the story plans. Foundation system is identified by excavating inspection pits in sufficient number. The collected topological information has to be adequate for constructing the analytical building model. It is assumed that the reinforcement details in concrete members confirm to the design code enforced at the year of construction. In order to confirm this assumption, reinforcement has to be inspected visually in 10% of columns and 5% of beams at each story by removing the concrete cover [TEC 2007].

- **Moderate knowledge level**

Essentially the same as the limited knowledge level, however reinforcement is inspected from 20% of columns and 10% of beams in each story. Moreover, a minimum of three concrete core samples are taken from the columns and walls, where the minimum total number is nine [TEC 2007].

- **Comprehensive knowledge level**

As-built structural drawings have to be available. They are verified by field measurements, and the differences are corrected. Other field measurements related to building geometry, reinforcement details and material properties are similar to those defined for the moderate knowledge level. However existing steel strength is verified by testing three steel specimens taken from the building. Stress-strain behavior of reinforcing steel is also determined by testing these specimens [TEC 2007].

Structural model of the building must be prepared by having sufficient detail in order to calculate internal forces and deformations on the elements in the light of the obtained data. The values obtained in the result of having the assessment explained in Chapter 6 used for the data collected for the elements, materials and cross section features.

Internal forces and deformations occurring in the elements are calculated by examining the obtained model of the structure with the earthquake loads affecting the building and under the common effects of the other loads. Since linear analyses, which form the basis for the assessment of force based methodologies are generally done being in compliance with the provisions of the Earthquake Regulation, maximum effects occurring on the cross sections must be calculated by using the load combinations specified in the regulation. For above mentions to determine the earthquake effects on the structure, spectral acceleration coefficient is used as shown in Figure 4.10. In Figure 4.11 shown the directions of earthquake loading effect on actual mass center and shifted mass center according to section 2.4 in Turkish Earthquake Regulation.

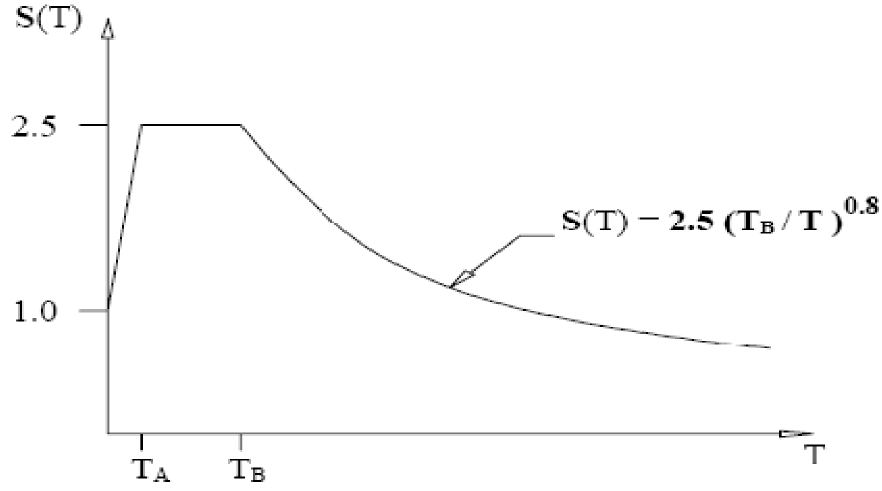


Figure 4-10 Elastic acceleration spectrum [TEC 2007]

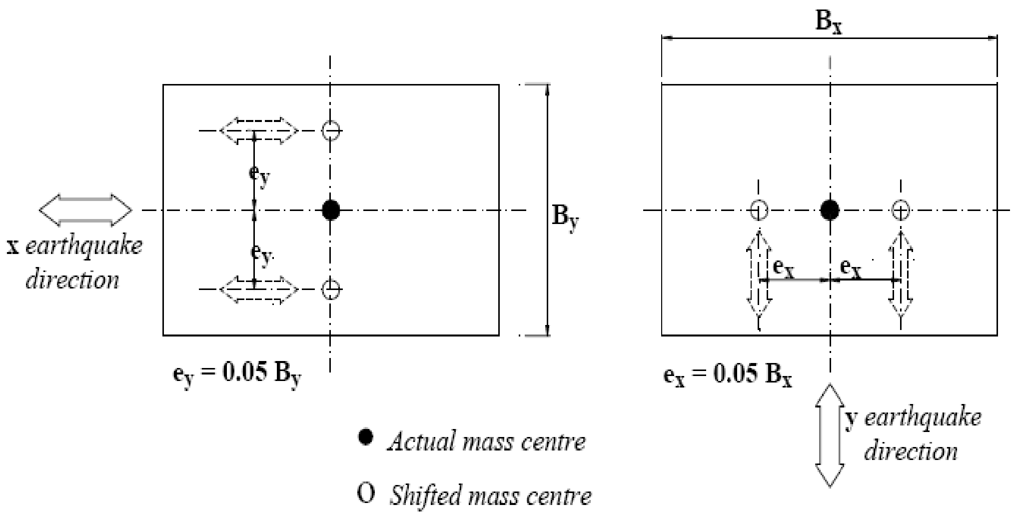


Figure 4-11 X and Y direction earthquake loading [TEC 2007]

4.3.1 Performance Assessment of Reinforced Concrete Members

Damage limits are expressed in terms of the demand/capacity ratios (member ***r*** factors) for ductile members at their critical cross sections. Ductile concrete frame members are controlled by the flexural failure mode where shear capacity exceeds the shear force developed when the member reaches its flexural capacity. The demand/capacity ratio for beams, columns and shear walls is the ratio of earthquake moment to the residual capacity moment at the critical section, where the residual capacity moment is the difference between the flexural capacity and the dead load moment.

The demand/capacity ratio of reinforced (strengthened) masonry infill walls is the ratio of shear force under earthquake loads to the shear capacity. The calculated member ***r*** factors for beams, columns, shear walls are compared with the ***r*** limits given in Tables 4.4 – 4.6 to determine the member damage states in accordance with Figure 4.12. Member ***r*** factors are analogous to section ductility ratios when equal displacement rule prevails [Sucuoğlu, 2006].

Table 4-4 Demand/capacity ratios for reinforced concrete beams (*r*)

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

Table 4-5 Demand/capacity ratios for reinforced concrete columns (*r*)

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

The reinforcement ratio in beams is defined as:

$$\frac{\rho - \rho`}{\rho_b} \tag{4.24}$$

where:

ρ is the tension reinforcement ratio.

$\rho`$ is the compression reinforcement ratio.

ρ_b is the balance reinforcement ratio.

For beam sections, if $\rho - \rho' < \rho_b$ then the section will be under-reinforced. Therefore, equation 4.24 is a factor related with ductility of the sections. For increasing this values ductility decreases and r_{limit} decreases. For both beam and column sections, as shear demand increases, the section tends to behave as brittle. Hence, increasing this value decreases ductility and r_{limit}. As $N/A_c f_c$ increases, ultimate curvature capacity decreases so that the ultimate rotation capacity decreases. For this reason, the r_{limit} values are decreasing for increasing $N/A_c f_c$ values. [Düzce,2006].

Table 4-6 Demand/capacity ratios for reinforced concrete shear walls (*r*)

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

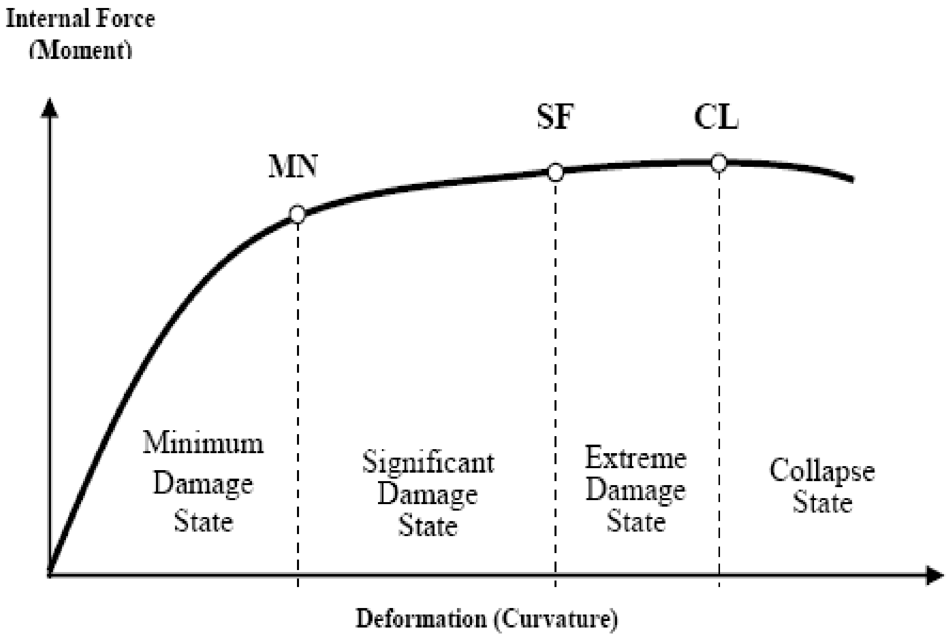


Figure 4-12 Damage limits and damage states in a ductile member

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

- **Immediate Occupancy**

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

- **Life Safety**

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

- **Collapse Prevention**

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

- **Collapse**

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

In order to assess the existing buildings the performance level for each buildings should be identified with their damage limits. Damage limits of the structural elements classified as minimum damage limit (MN), safety limit (SF) and collapse limit (CL) as shown in figure 4.12.

4.4 The Importance of Building Knowledge on the Calculation Methods

As mentioned above sections, the information obtained from the building will be used to form the analytical model of the building and to calculate the capacities of the elements. Thereafter, demand calculated under the effect of the earthquake will be compared for capacities by using a calculation methodology taking place in the Regulation and earthquake performance of the building will be decided. There are two calculation methodologies taking place in 2007 Earthquake regulation. Linear elastic calculation methodology is valid for each of the three information level. On the other hand, non-linear calculation methodology uses the details of the reinforcement and material characteristics in a much more comprehensive way.

CHAPTER 5

METHODOLOGIES USED FOR REPAIR AND STRENGTHENING OF RC BUILDING

5.1 Introduction

In this section, it will be appropriate to make a few definitions before entering repair and strengthening work methodologies. Concrete experts commonly use the terms structural repair and strengthening to describe building renovation activities. Although the two terms sound similar, they refer to slightly different concepts. Structural repair describes the process of reconstruction and renewal of a facility or its structural elements. This involves determining the origin of distress, removing damaged materials and causes of distress, as well as selecting and applying appropriate repair materials that extend a structure's life.

Structural strengthening, on the other hand, describes the process of upgrading the structural system of an existing building to improve performance under existing loads or to increase the strength of structural components to carry additional loads. For upgrade projects, design engineers must deal with structures in which every element carries a share of the existing load. The effects of strengthening or removing part or all of a structural element - such as penetrations or deteriorated materials - must be analyzed carefully to determine their influence on the global behavior of the structure. Failure to do so may overstress the structural elements surrounding the affected area, which can lead to a bigger problem and even localized failure.

To explain in brief; strengthening are the processes carried out to up-grade an undamaged structure or structure element to foreseen safety level. In this definition, the key factor is the *undamaged* condition of the building. On the other hand, repair is the processes carried out to up-grade a damaged structure or structure element to foreseen safety level. In this definition, the key factor is the *damaged* condition of the building. In damaged buildings, strengthening is done together with the general repairs [Tankut, 2008].

In order to subject a building to strengthening, it is required that there should be doubts related to the safety of the building. On the other hand, it is required that the structure must be in damaged condition in order to carry out the repair. Having strengthening in the agenda can be originated from the below stated reasons:

- Alteration in using. The structure will be used in a way other than the foreseen. For example: it can be a question to use a structure as a school whiles it has been built as a residence. In this case, since the live load and importance coefficient will be different, it is required to scrutinize the safety of the structure.
- The structure is not able to meet existing regulations.
- A doubt has occurred about the concrete strength of the structure while it was at the construction stage or after a certain period of time from its completion.
- Doubt about inadequacy of the reinforcement bars. This doubt may become visible in two ways:

1. Doubts may appear regarding the strengthening of the structure when the project is reviewed.
2. Doubt may arise that the foreseen details about the strengthening in the project have not been complied during the construction stage.

- It may be in question to add a new floor to the building or to construct many separation walls which do not have the bearing capacity.
- Building does not have sufficient lateral rigidity [Tankut, 2008].

5.2 Selection of Method

Methodologies to be used in relation with the evaluation of the existing buildings according to 2007 Turkish Earthquake Regulation are given in Section 7 of the Regulation. Developed methodologies are taken in hand in Chapter 4 with the heading of ``Seismic Performance Assessment`` with their relevant details for the purpose of determining earthquake performances of the existing structures.

At this point, the most important point to emphasize is the status ascertainments of the undamaged structure before and after the strengthening taking place in Section 7 of Turkish Earthquake Regulation. Consequently, the methodologies defined in this section of the Regulation must not be used for the ascertainment of statuses of the damaged structures and for their performance levels. Because, there are significant differences between the determination of the structural safety as a result of the assumptions for the analysis of the damaged structures and the assumptions done in the development of the regulation's methodologies. Besides, the use of the limit values in the determinations of undamaged structure elements' performance bench marks shall create mistrustful results in the evaluation of damaged structures' bearing elements. Because of these reasons, the approaches to be used in the repairs and strengthening of the damaged buildings have been left to the initiative of the designer in 2007 Turkish Earthquake Regulation [Özcebe, 2008]. Here, it should be noted that Gazi Mağusa Namik Kemal Lisesi as an undamaged building and section 7 of Turkish Earthquake Regulation can apply to determine the seismic performance before and after strengthening.

In the result of the analysis done, calculated lateral shifts for each floor must be compared with the limit values. Besides, bending and shear forces of the columns and beams must be compared with the shear forces and bending moments obtained from the analysis. At the end of the analysis and cross section control, a point of view is determined regarding the safety of the structure and the decision is taken. The decision is formed according to below stated options:

(i) There is enough structure safety. Therefore, there is no need for the repair and/or strengthening.

(ii) The structure can be used without any intervention by limiting the usage.

(iii) The structure can be reached to required structure safety with the repair/ strengthening.

(iv) Repair/strengthening is not an acceptable solution economically and practically.

If a decision has been taken for a structure's repair and/or strengthening, various options are generated in this topic. These options are evaluated in terms of cost, application convenience, construction duration and obstruction of usage. At the end of this kind of consulting, it can be possible to understand that a solution having low construction cost may bring unacceptable cost due to stoppage of the education life. Stoppage time may be much more than the cost of the strengthening.

5.3 Repair and Strengthening Methods

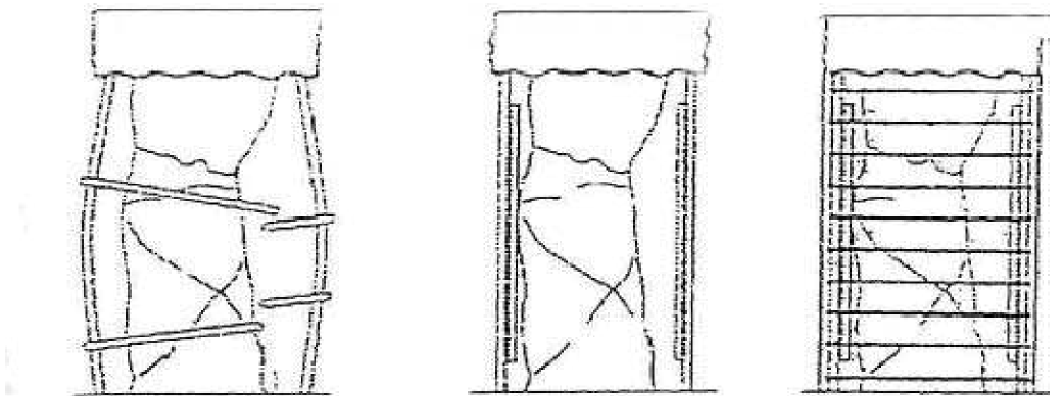
5.3.1 Repair and Strengthening of Columns

Columns are the most important elements of the structure. On the other hand, columns are also the elements which encounter the most damages during strong earthquakes. The objective of reinforcing columns as part of the load-bearing elements is to increase their resistance against axial loads, bending and shear forces, and to improve their ductility. The bending resistance of columns can be improved by increasing their cross-sectional area and by adding longitudinal reinforcement bars. The shear resistance and especially the ductility can be improved by providing stirrups with steel-belts and straps.

5.3.1.1 Local Repairs

Columns without damages in concrete or reinforcement but only with tiny cracks are repaired by injecting resin. Cement mortar injections are being used for wider cracks. Another technique used for local repairs is the vacuum method. After establishing a vacuum in the section of the crack, epoxy is applied. Just before the epoxy material gets settled, the vacuum gets abandoned. With the pressure-difference the epoxy penetrates and fills the cracks deeply. For columns with crushed concrete, reinforcement or stirrups failure, the damaged parts have to be removed and repaired.

Damaged concrete is being removed, the surfaces get roughed, new reinforcement welded to the longitudinal bars and new bonds are applied in short intervals before applying the new concrete (Figure 5.1).



(a) (b) (c)

Figure 5-1 Steps of column repair

5.3.1.2 Concrete jacketing

Sheathing is a repair and reinforcement method that is applied by increasing the concrete cross-sections and adding longitudinal reinforcement bars to improve the axial load bending moment capacity of columns. The important point here is that load is distributed from the previously existent part to the added part of the column. It can be done from one, two, three or all four sides, according to the working conditions.

Important points to be considered for concrete jacketing are the merge of the old and new concrete, and the anchorage of new and old reinforcement. The damaged section of the column has to be removed totally to the core. The distribution of forces can be enhanced by notches opened in the old concrete. By sheathing the longitudinal reinforcement before it is implemented in the story slabs the axial and shear resistance for that area can be enhanced. If an increased bending resistance of a column is desired, longitudinal reinforcement bars can be placed through drilled holes in the reinforced concrete slabs and concrete at the column – beam joint area for the anchorage of the longitudinal reinforcement. For the newly added parts the stirrups have to be bound, the concrete and longitudinal reinforcement have to be wrapped tightly. This solution will eliminate buckling of longitudinal reinforcement. The stirrups laterally support the concrete by wrapping it, and enhance the shear resistance and pressure tolerance. New stirrups are connected to the existing ones by welding or screwing with bolts.

When concrete jacketing is applied, it is paid attention that the resistance of the new material is equal or higher than that of the already present material. Additionally the layer thickness for shotcrete has to be at least 5 cm. For cast-in-place concrete this thickness has to be at least 10 cm. The diameter of the stirrups has to be not less than 8 mm and the minimum vertical spacing between stirrups have to be 20 cm.

Comprehensive experiments have been done with the propose of examining reinforced concrete columns' strength and behavior [Özden and Akgüzel, 2005]. From these experiments, the results to enlighten the application are summarized below:

- Jacketing in the way of above explanation exhibit a behavior being close to monolithness (a casting) together with the core.
- There is no disadvantage of reinforcement jacketing without lifting the load. Under monotonic and earthquake type loads rigidity and capacity of jacketing column (existing concrete and reinforcement, jacketing concrete and it's reinforcement) can be taken as 90% of the monolithic column with equivalent cross section and reinforcement (an assumption in a reliable direction).

- In the case of repair which performed after lifted the load, column capacity and rigidity of the jacketed column can be taken as 80% of monolith column. If the repair has been done without lifting the load, in this case rigidity of the jacketed column must be taken as 70% of monolith column. Only the jacketing must be taken into account when the capacity is calculated and existing damaged column must not be considered.

In Turkish earthquake regulation, there is no separation whether or not the load is to be lifted during the column's strengthening. If the continuity of column's longidutional reinforced bars can not be provided between the floors, only column ductility and shear force increases will be accepted. If longidutional reinforced bars are passed through the holes opened on the floors and if the required stirrups are placed by drilling the beams on column-beam joint zones or by doing anchorage on the beams, bending capacity of the jacketed column will be increased [Özcebe, 2003].

If column's strengthening is formed by without ribbed bars, stirrup zone effect will be very limited. While on the other hand, in cases using ribbed bar strengthening, the stirrup zone will have an important effect. The reason of this is that the clamping of ribbed bar type is a mechanical clamping because of the toots and for this reason tensile tensions form in the circumference of the strengthening. It is natural that the stirrup zone will meet these types of tensile tensions. On the other hand, the clamping of flat surfaced strengthening will not have too much effect since it is provided with adhesion and friction [Aksan, 1988].

5.3.1.3 Steel jacketing

If some of the columns taking place in a structure have problem in terms of axial load and/or stirrup zone strengthening are inadequate, axial load can be increased and stirrup zone affect can be provided with a steel support to be done. In general, steel support is configured by welding the steel sheet bars with certain intervals with the angle brackets placed on the four corners of the column. Angle brackets contribute to axial load while lateral sheet bars provide wrapping affect from the outside and prevent the buckling of the angles.

If it is desired to increase the effect of the stirrup zone, horizontal sheet bars welded to the angle brackets must be placed more frequently and they must definitely rest to the face of the column with the special mortar to be placed under them and if needed, post-tension must be applied on these sheet bars. Before welding of the horizontal sheet bars, leaning of the angles against the column must be provided with an apparatus in the type of bench clamp [Altın, 1992].

If the cohesion of the steel jacket and existing concrete has been provided well then the shear capacity of the column increases. In this type of application, it is very important not to have any voids on top and at bottoms. Steel jacket can not be effective if there is void. Details of the base and ceiling are very important [Ersoy, 1992]. With the steel jacketing method columns are repaired and strengthened by L-beams which are applied to the four corners of a column and then connected by lateral steel belts to form a skeletal case (Figure 5.2). The space between the angles and the concrete are filled with non-shrinking cement mortar or resin mortar. The steel elements are covered with shotcrete to preserve them from fire. With this method not only the carrying capacity of the column, but also its ductility is improved. Nevertheless, there is no change in rigidity.

Figure 5-2 Steel Jacketing Application on the Columns

The laboratory experiment results of the behaviour of strengthening concrete columns which are repaired off-load and steel-angle reinforced on all 4 corners under axial loads are as follows [Can, 1996];

- Ferro-concrete columns which are repaired off-load and steel-angle reinforced on all 4 corners has succeeded in terms of their resistance to axial loads. The resistance values of the columns have reached 92 % of the strength of monolithic cross-sections after reinforcement, and 89 % after repair.

- The rigidity of a bare column could be improved by 22% after repair and 32% after reinforcement.
- Columns have displayed significant improvements in ductility and energy absorption after being repaired and reinforced.

5.3.2 Repair and Strengthening of Beams

The objective of the repairs and strengthening measures for beams is to ensure sufficient bending resistance and to improve the resistance to shearing forces. The main point to be considered here is to avoid a load-bearing system with strong beam – weak column deficiency.

5.3.2.1 Local Repairs

Beams with tiny cracks are repaired by injecting epoxy or cement mortar. For beams with crushed concrete, degenerated adherence, or buckled reinforcement, the damaged parts have to be removed and re-constructed. Before starting with repairs, the beams have to be supported and propped up temporarily. The concrete that is placed beneath the present beams has to be compacted thoroughly. Additionally the shear strength of a beam can be improved by steel bars attached with longitudinal or diagonal clamps.

5.3.2.2 Concrete jacketing

To improve the bending and shear strength of a beam jacketing is applied from one, three or four sides (Figure 5.4). To achieve this, the tension reinforcement and the moment arm have to be increased. Therefore, the added concrete increases the cross-sectional area, and the added tensile reinforcement improves the bending moment of the cross section. The newly added tensile reinforcement has to be connected to the existent reinforcement by welding with fittings that are called V- or Z-links (Figure5.3). If there is no need to increase the cross-sectional area and only tensile reinforcement is to be added, sheets can be welded to the old and the new reinforcement in certain intervals. An important point to consider here is that a proper anchorage is provided by welding.

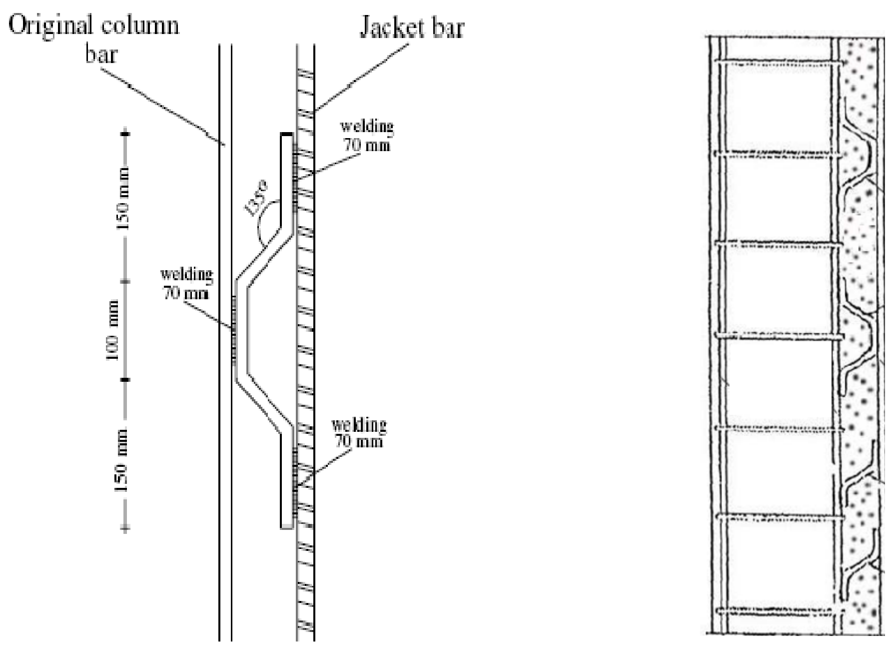


Figure 5-3 Connection of old and new reinforcement by V and Z bars

The negative moment areas at the column ends may have to be strengthened too. In that case the upper reinforcement of beams has to be placed through slots which are opened in the column and connected to the present reinforcement by welding it with V bars to make sure they work in conjunction with the present reinforcement. With additional stirrups all over the surfaces of present beams an adequate improvement in shear strength and ductility can be achieved. The stirrup handles have to be placed through slots drilled through the slab in the upper part of the sheathing to be anchored above the beams with bolts. In this way the shear force capacity of beams can be improved.

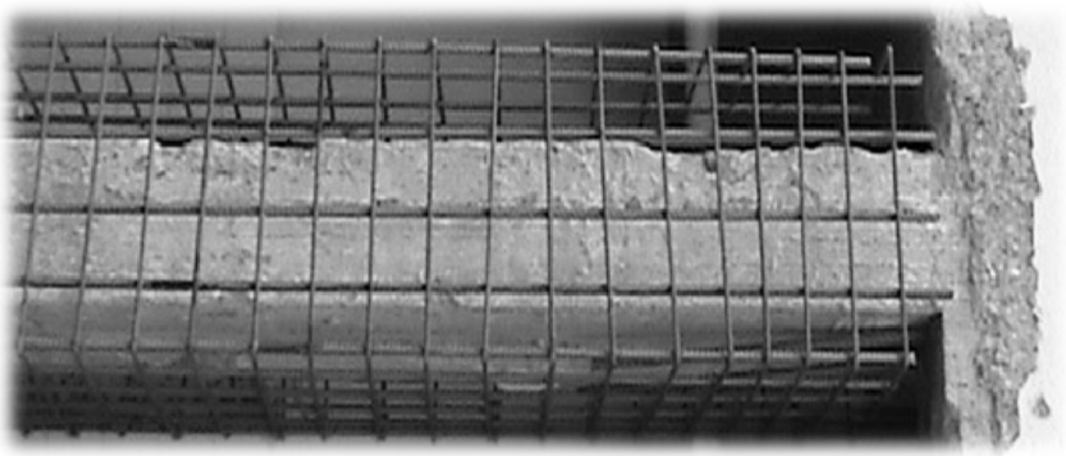


Figure 5-4 Typical Sample of Beam Jacketing

5.3.2.3 Steel jacketing

This method is implemented by adding steel sheets underneath the beams in order to improve their bending force bearing capacity or their lateral surfaces to improve their shear force capacity. Continuous steel sheets are fastened to bolts that are previously fitted into the concrete with epoxy resin (Figure 5.5 and 5.6).

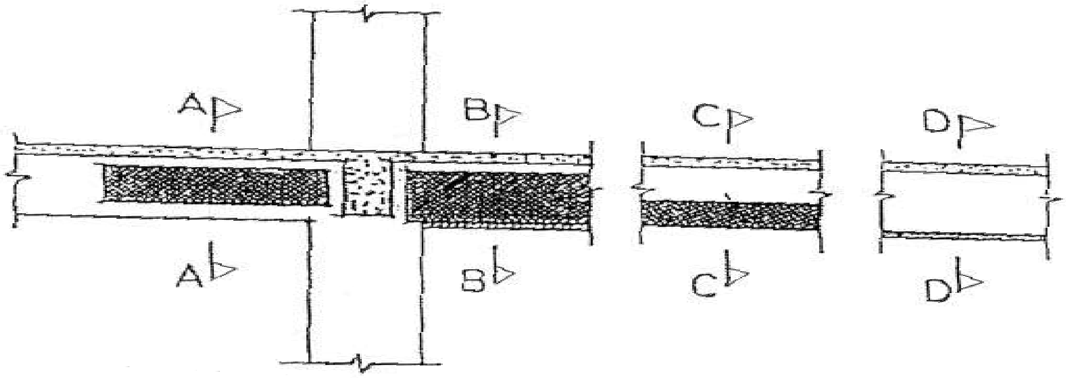


Figure 5-5 Strengthening of beams with steel sheets against bending and shear

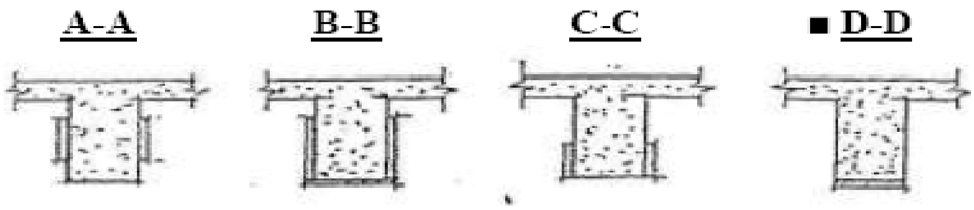


Figure 5-6 Cross-sections of steel sheets reinforced beams

Plate bonded on tensile zone layers composed of composite materials increase moment capacity of the beam and do not affect swelling and shear strength. However, when it is considered from capacity design point of view, it must not be forgotten that the increased bending capacity taking place on the ends of the beam will increase the shear strength of the beam which will act during the earthquake. In this case, shearing capacity must be controlled with the shear strength according to capacity design and if needed, strengthening should also be done in order to increase shear strength. This type of application will also increase wrapping effect [Ersoy, 1996].

In the work “Behaviour of Ferro-concrete beams strengthened against shearing”, the results of tests of beams with insufficient shear reinforcement strengthened by applying external clamps are given below [Altin, 1997]:

- This kind of strengthening measures to improve shearing strength successfully.
- While shear fractures are expected from beams with insufficient shear strength, strengthened elements tend to fail from flexure.
- Clamps have succeeded to control inclined shearing fractures by preventing them getting broader.
- It could be observed that clamps succeeded in holding together the web of the beam and its supporting plate as if stick together.
- The resistance of strengthened beams has turned out to be at nearly the same level as the reference beam with shear reinforcement.
- The increase of applied external clamps resulted in a significant decrease of the ductility of the test elements.

5.3.2.4 Carbon Fiber Polymers

There are various methodologies applied in the repair and strengthening of the beams. One of these methodologies is to increase the capacity of the beam with steel plates by using bonding agents having high strength or by using carbon fiber polymers and similar. Since the bonded plate will be cut from a certain place, it is possible to look at the topic by establishing parallelism between the issue created with this application and the cutting of reinforced concrete strengthening. The important thing is not to cut the plate at a point where it is not needed theoretically and to cut this point after passing it with a distance being equal to the depth of the beam. Since there will be tension accumulations at the place where the plate is cut, it is useful to fasten these ends to the body of the beam after clamping them. Usage of carbon fiber polymer layers (CFRP) instead of steel plate is increasing [Ersoy, 1992]. Above stated rules are also valid for this methodology. It is possible to have shear reinforcement with carbon fiber layers (Figure 5.7).

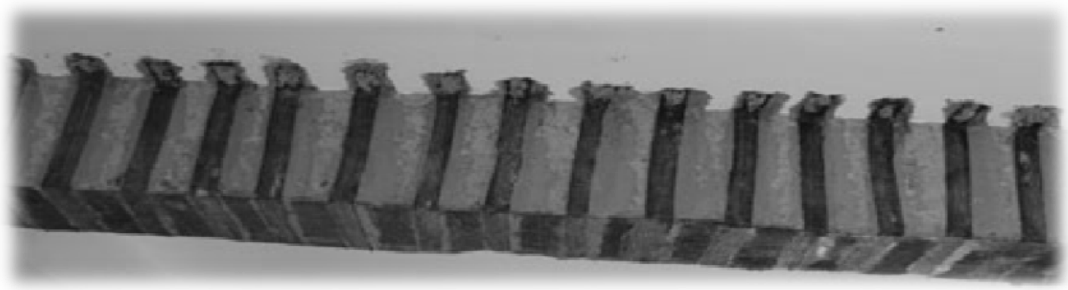


Figure 5-7 Typical Sample of beam strengthening with using CFRP

5.3.3 Repair and Strengthening of Column-Beam Joints

Nodes generally suffer shearing, adherence and anchorage damages. Heavy seismic loads cause the occurrence of plastic hinges on beams and column surfaces. Beams suffer fractures in their entire height. The surfaces of beams on both sides of internal nodes can suffer different stresses in terms of compressive and tensile stress. For that reason the anchorage of the beam reinforcement has to be made properly. Additionally, due to the loss of adherence on the column surface the reinforcement of beams comes off and the beam loses its bearing capacity.

5.3.3.1 Local Repairs

For fractures of light to medium severity it is adequate to use epoxy resins. This brings the adherence between reinforcement and concrete to its previous level. The removal and reconstruction of the damaged part is implemented if concrete is crushed or the reinforcement is yield.

5.3.3.2 Concrete jacketing

The repair and reinforcement of beam-column joints is implemented by sheathing both the column and the beam at the node where they meet. To be able to weld the new reinforcement to the present one, the concrete cover has to be removed. Longitudinal and lateral reinforcement bars and stirrups have to be connected to form skeleton cases. The rigidity of the sheathing has to be sufficient enough too. Longitudinal and lateral stirrups have to be placed at the connection node to achieve a sufficient shear resistance in that section.

5.3.3.3 Steel jacketing

This method allows a strengthening of the nodes without any change in their dimensions by using steel sheets as reinforcing equipment. With this method which is more often used for industrial type load bearing systems, the steel sheets are precast to fit the node dimensions and fixed to concrete surfaces with epoxy resins. Steel sheets have to be anchored to nodes by pre-stressed bolts. The most important issue to be considered here is the prevention of the system against fire and corrosion.

5.3.4 Repair and Strengthening of Shear Walls

The objective of repair and reinforcement of shear walls is to improve their lateral load bearing capacity or to correct irregularities.

5.3.4.1 Local Repairs

Cracks in shear walls are repaired with epoxy resins. In cases of adherence degeneration or crushed concrete, the bending and shearing resistance of shear walls can be restored to its previous level by epoxy resin injections. However, this will not restore the previous rigidity. Because every crack can not be filled with epoxy injections. On the other hand the necessary precautions against fire have to be taken.

5.3.4.2 Increase of Dimensions

The thickness of shear walls has to be increased if the previous resistance is insufficient. If the bending strength has to be improved, reinforced concrete edge columns should be added on both ends. In order to improve shear resistance, wall thickness has to be increased. The mostly used method to strengthen shear walls is the shotcrete technique. To ensure adequate adherence between old and new concrete, the existing concrete surface of the wall has to be roughened up. The new reinforcing bars have to be anchored to the old ones by welding. The added shell thickness has to be at least 5 cm and the thickness of edge columns has at least 10 cm. The adequate and sufficient transfer of loads to the foundations should be ensured. The distribution of rigidity and lateral forces should be monitored.

5.3.5 Repair and Strengthening of Slabs

Slabs are repaired when they are damaged. Strengthening measures are taken if the resistance of a slab appears to be insufficient or when joint areas of newly added shear walls have to be reinforced. Damages generally occur at sections of erroneous, empty spaces and landings.

In general, floors outside of console do not have issue related to bending capacity. Most common issue encountered in the floor is the deformation issue because of not having enough rigidity. In these cases, the thickness is increased by adding a concrete layer to the floor. Normal reinforcement and mesh reinforcement can be used in this additional layer. Floor is drilled with certain intervals in order to integrate the layers of old and new concrete (in general, holes with one meter intervals are enough). Bigger size bars having a little bigger diameter than these holes are hammered into these holes and an arrangement to meet horizontal shearing is provided this way.

5.3.5.1 Local Repairs

Cracks are repaired with the injection method. The material to use can be epoxy resin or cement mortar. In case of crushed concrete or broken, bended reinforcement damages, the damaged section is removed and reconstructed. The new reinforcement should be anchored accurately to the existing one.

5.3.5.2 Increase of Layer Thickness

The strengthening of slabs is achieved by increasing the layer thickness. This 'thickening' is achieved by an additional layer that is added upon or beneath the existent slab. If added upon the slab, besides the increased of the effective height, the opportunity to add reinforcing steel to bearings for negative moment is given as well as improving the rigidity and improving the diaphragm feature of the slab. If the addition is applied beneath the slab, the added tension reinforcement will improve the bending resistance of the slab. Yet the shotcrete method has to be applied. The surface is roughed up either by adhering grains of sand with epoxy resins, or by blasting sand on the surface. The connection of slab-to-shear wall is fixed with dowels. To achieve this, a deep hole is drilled through the slab to pass the shear wall vertically. Reinforcement bars are then applied through the shear wall within the level of the slab in necessary dimensions.

5.4 Improving Structural System

The improvement of the lateral forces resistance capacity of reinforced concrete constructions is effectively achieved by adding new load bearing elements to the existing system. The new construction element that is added changes the seismic behaviour of a building significantly and increases seismic loads. For that reason, the strengthened new system has to be considered as a wholly new system to be designed and solved from the beginning. It is possible to add new face walls to improve the torsion rigidity of the building to improve its resistance against torsion effects. Additionally irregularities like a soft story or rigid short columns can be treated by adding face walls. High torsion effects can be minimized by reducing the distance between the centre of mass and the centre of rigidity.

5.4.1 Addition of New Walls or Columns

Strengthening is achieved by adding new shear-walls. While adding new shear walls core should be taken for a proper connection that suits the slabs and roof diaphragms as well as the foundations. The transfer of shear forces between slab and shear wall is achieved by tie bars applied as L-bars according to the plan that are anchored to the slab. In this application both, cast-in-place concrete and shotcrete can be used. Some static plans are shown irregular columns distributed in the load bearing system, while with that kind of distribution the building will suffer immense torsion impacts. However, by adding columns in a way that the distance between the centre of mass and the centre of rigidity is minimized, the torsion caused by irregularity of the building will be decreased. If buildings have different slab elevation and haven't got any dilatation joint between these slabs, it can be reinforced by adding shear walls or new foundations. If depending on the land topography the foundation bases are on different levels, the building can be reinforced with new shear walls and additional foundations.

CHAPTER 6

ASSESSMENT OF GMNKL

6.1 Introduction

All properties of concrete, reinforcing bars, load-bearing elements and structural system of GMNKL are assessed before evaluating the seismic performance assessment. For these purposes several tests have been done. Half cell potential test, core test, compressive strength test and Schmidt hammer test results were evaluated. In this chapter all these tests results and load-bearing member's properties are given. GMNKL consist of 4 different main blocks as shown on the satellite view in Figure 6.1.

Figure 6-1 GMNKL block name

6.1 Material Properties of GMNKL

6.1.1 Half Cell Potential Testing

Reinforce bars in good quality concrete is protected from high-alkalinity pore water which, in the presence of oxygen, passivates the reinforcement. The loss of alkalinity due to carbonation of concrete or the penetration of chloride ions (arising from either marine or de-icing salts, or in some cases present in-situ from the use of a calcium chloride additive) can destroy the passive film (ACI, 1985; Arup, 1983). In the presence of oxygen and humidity, corrosion of steel in concrete starts. A characteristic feature for the corrosion of steel in concrete is the development of macro cells, that is, the co-existence of passive and corroding areas on the same reinforcement bar forming a short-circuited galvanic cell, with the corroding area as the anode and the passive surface as the cathode.

The voltage of such a cell can reach as high as 0.5 V or more, especially where chloride ions are present. The resulting current flow (which is directly proportional to the mass lost by the steel) is determined by the electrical resistance of the concrete and the anodic and cathodic reaction resistance [Elsener and Bohni, 1987]. The current flow in the concrete is accompanied by an electrical field which can be measured at the concrete surface, resulting in equipotential lines that allow the location of the most corroding zones at the most negative values. This is the basis of potential mapping, the principal electrochemical technique applied to the routine inspection of reinforced concrete structures [Berkeley and Pahmanaban, 1987].

This test method covers the estimation of the electrical half-cell potential of uncoated reinforcing steel in field and laboratory concrete, for the purpose of determining the corrosion activity of the reinforcing steel (ASTMC876-91). A Half

Cell gauge measures the condition and potential corrosion of reinforced bars in concrete. Steel corrosion is an electrochemical process which contains anodic and cathodic areas on the reinforcing bars. The use of the technique is described by ASTM C876-91.

When surface potentials are taken, they are measured remote from the reinforcement due to the concrete cover. The potentials measured are therefore affected by the ohmic potential drop in the concrete. Several factors have a significant effect on the potentials measured [Newman, 2003]. These factors are;

a. Concrete cover depth
b. Concrete resistivity
c. High resistive surface layers
d. Polarization effects

According to the ASTM C 876 method and Table 6.1 corrosion can only be identified with 95 % certainty at potentials more negative than –350 mV. Experience has shown, however, that passive structures tend to show values more positive than –200 mV and often positive potentials. Potentials more negative than –200 mV may be an indicator of the onset of corrosion. Corrosion potential readings for GMNKL building are as shown in Table 6.2 and in Figure 6.3 corrosion occurred on the column reinforcing bars in S 56 column.

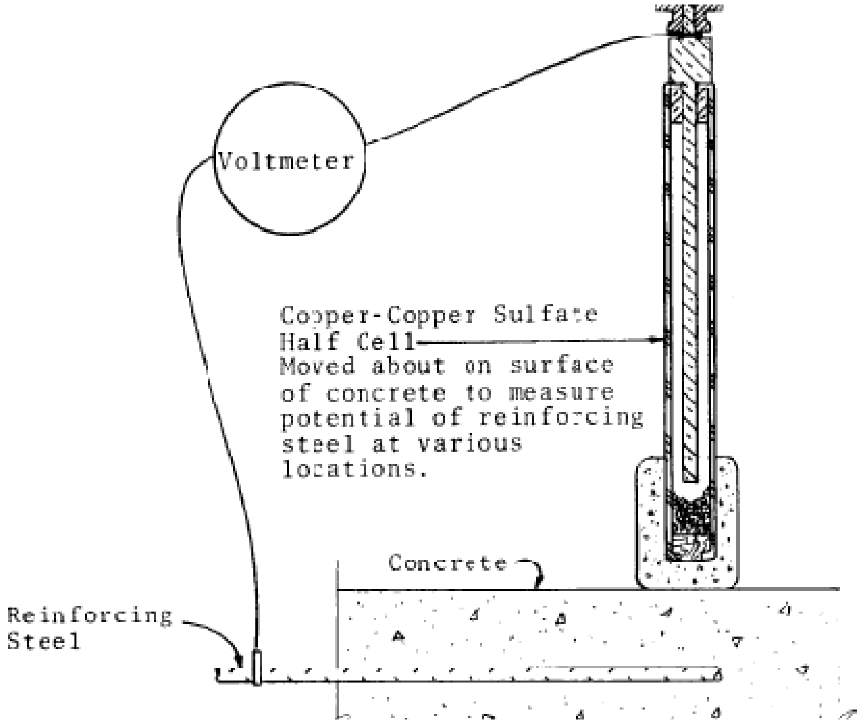


Figure 6-2 Copper-Copper Sulfate Half Cell Circuitry.

The patterns formed by the contours can often be a better guide in these cases. It should be noted that the silver/silver chloride half-cell produces results some 50–70 mV more positive than a copper/copper sulphate cell (Figure 6.2)[Newman, 2003].

Table 6-1 Probability of corrosion according to half-cell readings

Half-cell potential reading, vs. $Cu/CuSO_4$	Corrosion activity
less negative than -0.200 V	90% probability of no corrosion
between -0.200 V and -0.350 V	increasing probability of corrosion
more negative than -0.350 V	90% probability of corrosion

Figure 6-3 Corrosion occurred on the column reinforcing bars in S 56 column of GMNKL building

Table 6-2 Corrosion potential values of GMNKL building

Block	Name of column	Average corrosion potential value
B	S53	-94.00
B	S56	-435.07
B	S71	-388.64
B	S79	-145.71
B	S77	-110.57
A	S30	-43.79
A	S24	-192.00

6.1.2 Core Test

This method is used for quality control by destructively taking samples from hardened concrete (i.e. if there are deep cracks, freeze-thaw damages, surface fractures which indicate impaired solidity and if test results of any non-destructive method indicate any suspicions about the strength).

Strength tests on core samples enable us to quantify the resistance against compression inside the hardened concrete. The determination of the resistance inside hardened concrete should include both, destructive and non-destructive test methods. Before a test sample from the hardened concrete is taken, the surface hardness should be examined with a test hammer (Schmidt-Hammer) to determine the ($\boldsymbol{R_m}$) rebound values of the concrete where the sample is going to be taken [Simsek, 2007]. The findings of the surface hardness tests are considerably influenced by cast vibrations, carbonated surface, aggregated particles and maintenance. Without the findings of a test like the strength test on a core sample cut out of the concrete, information gathered by surface hardness tests is not sufficient to determine the hardness of concrete. As stated above, while examining the strength tolerance of a given part of a construction, there will be considerable divergences between the chart values returned by the ($\boldsymbol{R_m}$) values of surface hardness testing devices and the real resistance values of that building [Bungey and Millard, 1996].

There should be an established chart comparing the core strength values and the test hammer readings at the point where cores are taken in order to find a relation between core strength and rebound hammer values. With this approach it is possible to test many more construction parts by taking fewer samples from same class of concrete. The concrete resistance has been quantified by taking a total of 9 cores from all of the blocks of building and findings were used during the analysis.

Table 6-3 Compressive strength of core samples including reinforced bars

Block	Name of Column	Compressive Strength (MPa)
D	S110	29.66
D	S96	43.10
D	S88	24.19
B	S53	17.87
B	S71	42.70
B	S79	38.70
B	S77	37.20
A	S30	33.60
A	S24	32.12

For a single bar

Corrected strength = Core strength × $[1.0 + 1.5(\varphi_r / \varphi_c) \times (h/l)]$

where φ_r = bar diameter

φ_c = core diameter

h = distance of bar axis from nearer end of core

l = core length

For multiple bars

Corrected strength = core strength × $[1.0 + 1.5\sum(\varphi_r \times h)/(\varphi_c \times l)]$

If the spacing of two bars is less than the diameter of the largest bar, only the bar with the higher value of $(\varphi_r \times h)$ should be considered.

The following paragraphs give the progressive steps to convert a core strength value to either an estimate of in-situ strength or potential strength.

Estimation of
In-situ Strength

Horizontal direction

Core strength x 2.5/(1.5 + 1/ λ)

Vertical direction

Core strength x 2.3/(1.5 + 1/ λ)

Note that λ = core (length/diameter)

Estimation of
Potential Strength

Horizontal direction

Core strength x 3.25/(1.5 + 1/ λ)

Vertical direction

Core strength x 3.0/(1.5 + 1/ λ)

Note that λ = core (length/diameter)

Table 6-4 Compressive strength of core samples without reinforcing bars inside

Block	Name of Column	Compressive Strength (MPa)
D	S110	34.75
D	S96	51.68
D	S88	28.90
B	S53	19.02
B	S71	42.69
B	S79	38.69
B	S77	39.19
A	S30	33.59
A	S24	32.12

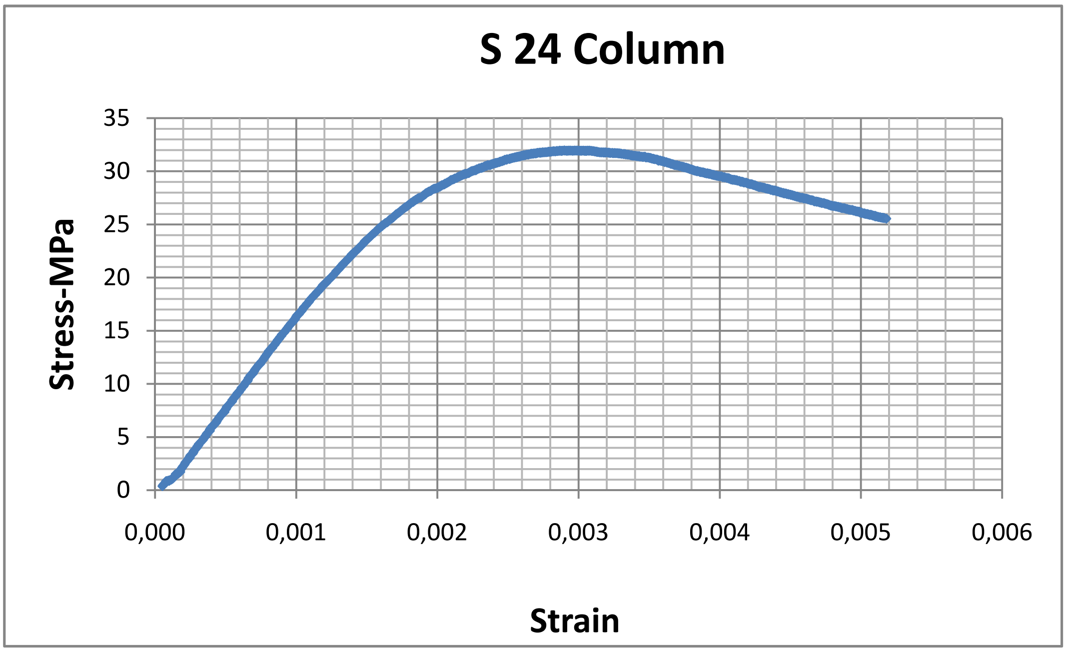


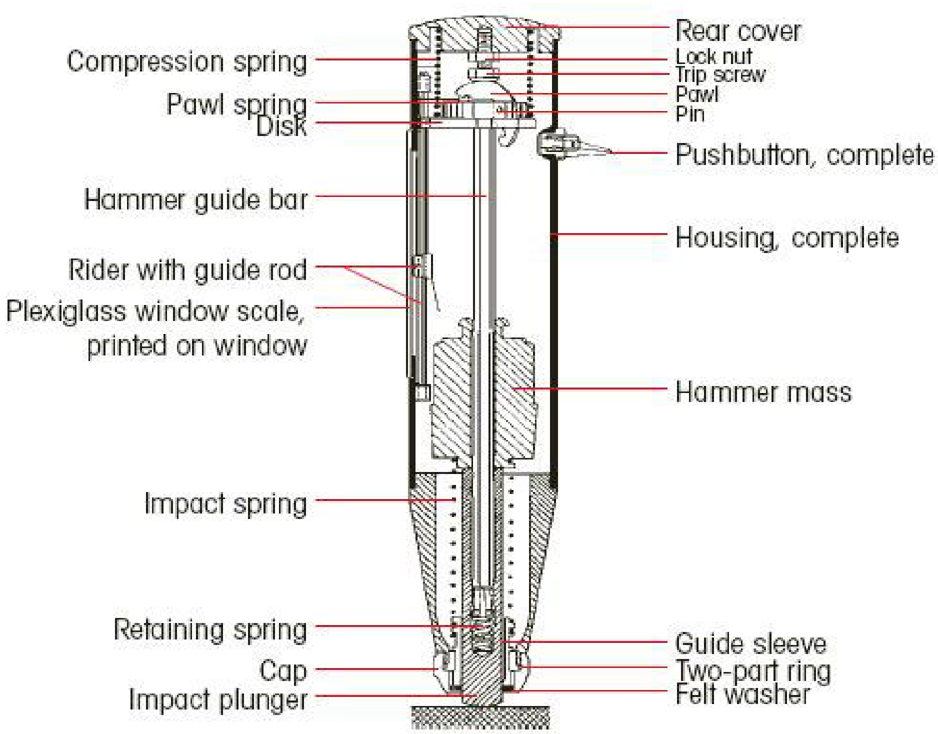
Figure 6-4 Stress-Strain diagram for S-24 Column which was located in block A

As shown in Figure 6.4, S 24 column core sample compressive strength determined by using stress-strain methods, for determine ultimate strain of concrete (ε_{cu}).

6.1.3 Schmidt Hammer Test

This test is also known as rebound hammer, impact hammer or accelerometer test. Schmidt hammer is used both in the laboratory and also on the field. ASTM C 805-02 describes this test (Figure 6.5).

According to many researches, there is a general correlation between the uniaxial strength of the concrete and hammer's rebound number. Therewith, various researchers do not believe the strength determination's sensitivity by using rebound reading [Simsek, 2007]. It is not possible to use the hammer instead of uniaxial experiments. However, the hammer can be used in order to determine the quality and homogeneity of the concrete strength and for the comparisons of the concretes with each other. Rebound method has gained serious adequacy in many



countries to take place in ASTM and BS standards.

Figure 6-5 Picture of Schmidt Hammer

The purpose of perform Schmidt Hammer test is the ascertainment for the strength of concrete and to determine the areas in which this method can be used to evaluate the results. More than one hammer type can be used for the measurement of the surface hardness and this method changes according to the characteristics of the experiment place or element. Model 58 C 0181 N Standard Test Hammer was used throughout the experimental study of Namik Kemal Lisesi. Results of experiments are as shown in Table 6.5. In Figure 6.6, relation between compressive strength of cylinders and rebound number readings are given.

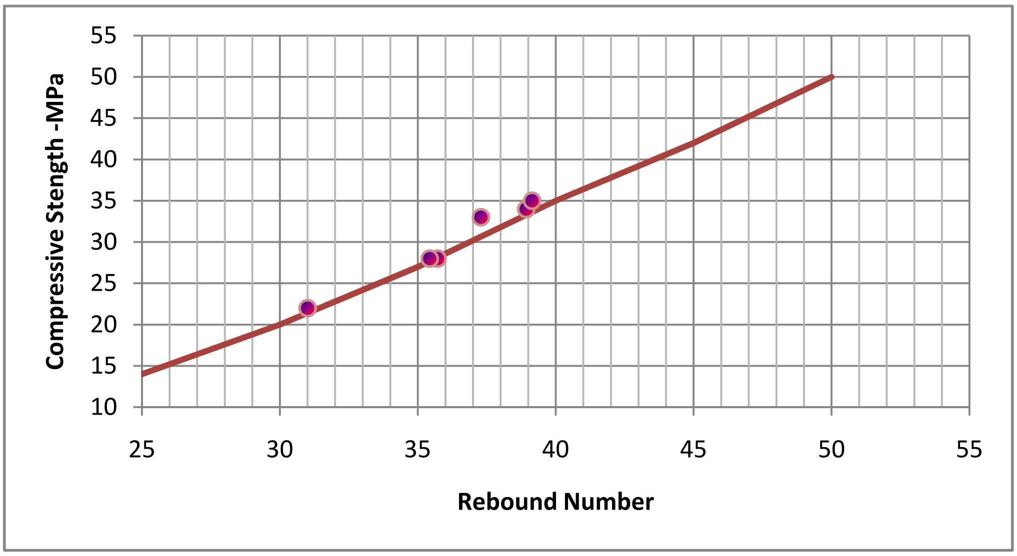


Figure 6-6 Relation between compressive strength of cylinders and rebound number readings with the hammer horizontal on a dry surface of concrete

Table 6-5 Correlation of Schmidt Hammer Rebound number

Block	Name of Column	Average Rebound Values	Correlation Values (MPa)
B	S53	37.29	33
B	S56	35.71	28
B	S71	18.64	5
B	S79	38.93	34
B	S77	39.14	35
A	S30	31.00	22
A	S24	35.43	28

6.2 Structural Assessment

The load-bearing system of a building has to be able to safely transfer external loads and its own weight to the ground. The subjects of assessing and modelling structure accurately have been examined in previous chapters. The characteristics of the construction materials are registered via destructive and non-destructive tests. The number of floors, symmetry and purpose of the building, which can be determined by the external view, are important steps in the assessment of a building. The seismic hazard where the building is, the occupancy importance factor, the effective ground acceleration coefficient, soil characteristics and the load combinations which are used for the construction or the assessment of a building are discussed in the following sections.

As stated in Chapter 7 of the seismic code 2007 “The Assessment and Strengthening of Present Buildings”, article 7.2.2 in the paragraph about “information levels”, for GMNKL, moderate information level has been chosen. The reasons for that choice are as follows;

- **GEOMETRY OF THE BUILDING**

    1. As the design project of the building is not available, the static project obtained after the survey of the building has been inspected.
    2. The obtained data contains the locations, span length, heights and dimensions of all reinforced concrete elements and walls.
    3. The building-geometry contains the necessary details to calculate the mass of the building.

4. Short columns or other similar issues having been recorded in the story drawings and cross-sections.
5. Detachment, adjacency or the existences of joints are stated.

- **DETAILS OF ELEMENTS**

1. The cover of reinforced concrete elements has been skimmed off to determine the reinforcement details.
2. For elements where the cover was not removed, the number and the layout of the reinforcement bars have been found with a detector.

- **MATERIAL CHARACTERISTICS**

1. Nine samples were taken and tested according to the requirements of TS-10465, to be one for every 400 m^2.
2. The concrete strength distribution of the building has been verified by concrete-hammer readings which were aligned with core results.
3. The reinforcement status was determined by visual examination for elements where the covering was removed.
4. Corroded elements were stated in the project and considered in the calculations.

- **BEAMS**

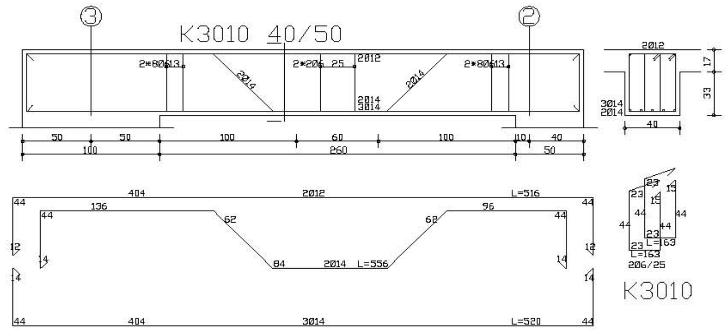
Beams are important element of structure. It can be said that beams have two primary functions in the load-bearing system. The first function is to transfer vertical dead loads from the slabs, and if existent, the loads of walls on them, to columns or shear walls to which they are anchored. These loads act vertically. The second one is to transfer horizontal loads especially resulting from earthquakes and wind to vertical load bearing elements together with the slabs. According to the practices stated above, Table 6.6 derived for the classification of beams. Beams K 3010, K3016 and K3083 assessed with using both destructive and non- destructive (ferroscan) methods (Figure 6.7). Ferroscan was used to assess Beams K3060 and K3048.

Table 6-6 Classification of beams

Location of the beams	Block Name	Size of Beams	Span length of Beams (m)			
			0--2	2--4	4--6	6--8
Exterior Beams	**A**	40/50		K3038	K3045	
				K3006	K3000	
				K3032	K3007	
				K3053	K3003	
				K3055	K3004	
				K3039	K3052	
				K3046		
				K3008		
	B	40/50		K3065	K3072	
				K3027	K3028	
				K3071	K3022	
				K3088	K3031	
				K3086	K3025	
				K3078	K3079	
				K3085		
				K3023		
	C	40/50	K3011	K3010	K3012	K3017
			K3019	K3018	K3020	K3015
						K3014
						K3016
		50/50		K3059		
				K3061		
		40/70		K3013		
				K3021		
				K3056		
				K3058		
Simply Supported	**A**	40/50		K3009		
	B	40/50		K3024		
	C	40/50			K3063	K3062
						K3064

Location of the beams	Block Name	Size of beams	Span length of Beams (m) 0--2	2--4	4--6	6--8
Interior Beams	A	40/50		K3047	K3041	K3043
				K3040	K3047	K3044
				K3054	K3049	K3036
				K3005	K3001	K3037
				K3002	K3034	K3051
				K3033	K3048	K3050
					K3035	
					K3042	
	B	40/50		K3084	K3076	K3074
				K3077	K3096	K3073
				K3087	K3082	K3067
				K3026	K3029	K3066
				K3030	K3069	K3080
				K3070	K3083	K3081
					K3068	
					K3075	
	C	40/70			K3057	
		50/50			K3060	

K3060	used feroscan
K3010	assesment + used feroscan



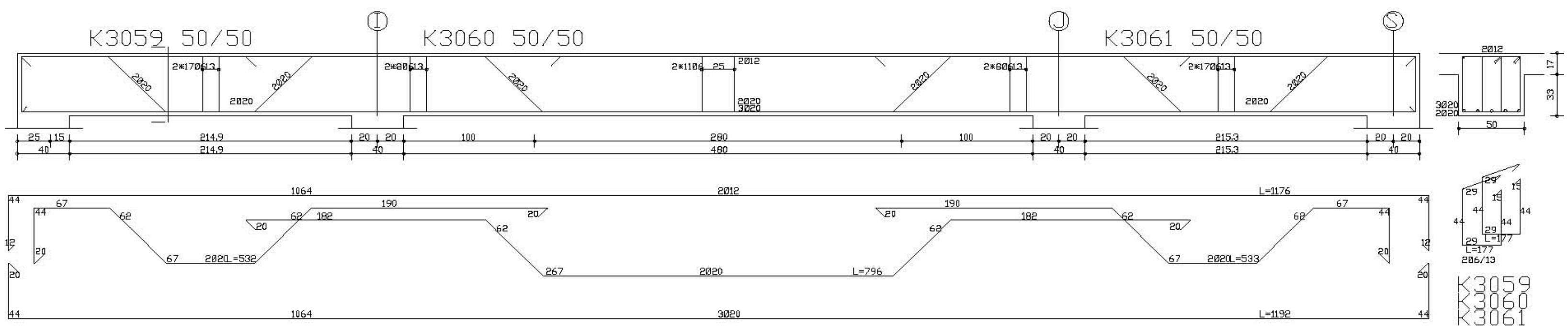


Figure 6-7 Details of beam K3010 and K3060

- **COLUMNS**

Reinforced concrete columns are the most important elements of framed structure systems. Therefore columns have to carry axial loads, bending moments and shear forces, and in some situation torsion. The positioning of the columns, which are one of the most important elements of a building, their dimensions and reinforcements are given in Figure B.1, B.2 and B.3 of Appendix B. It has been observed that GMNKL building has three kinds of columns as shown in Figure 6.8.

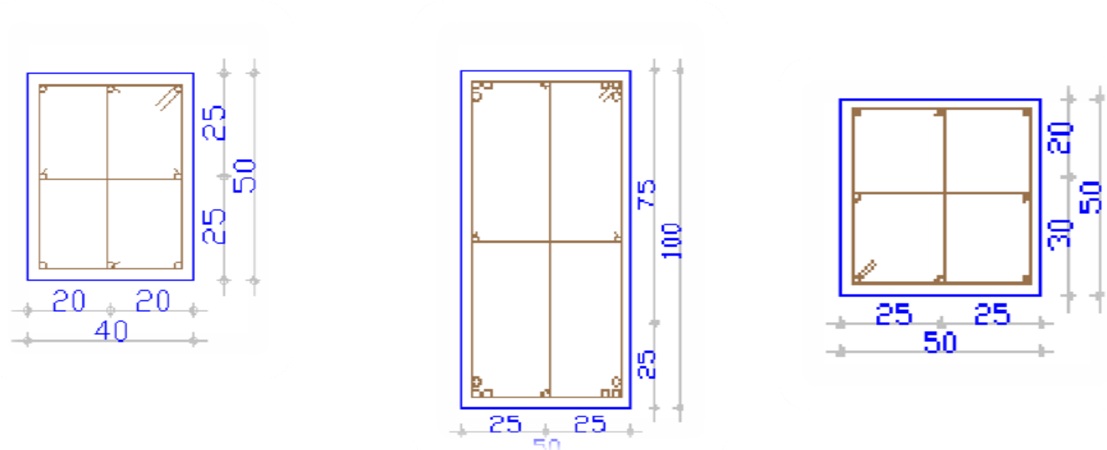


Figure 6-8 Column sizes of GMNKL

CHAPTER 7

ANALYSIS

7.1 Introduction

In this study, a three storey school building named Gazi Mağusa Namık kemal Lisesi is evaluated according to 2007 Turkish Earthquake Code in detail. This building was studied in order to verify the consistency of analysis and design procedures in the code. Each storey is 4.77 m in height with 1600 m² floor area. This building has 509 load-bearing elements (240 columns and 269 beams). Each storey has different plans. All plans of GMNKL before and after strengthening are given in Appendix B. Assessment steps and results of GMNKL were mentioned in Chapter 6. Linear and Non-Linear Analysis calculation were determined by using existing building properties and earthquake parameters, which are shown below in Table 7.1. Earthquake zone has been taken 2 to be more realistic result [Can, 1997, Yücemen, 1997]. Compressive strength of concrete is 34 MPa and reinforcement bars are S220 class for all members, on the other hand concrete used for strengthening is decided to be 25 MPa and S420 class reinforcement bars. Storey masses, mass center coordinates and mass moment of inertias are given in Table 7.2.

Table 7-1 Existing properties and code parameters of the building

Existing Building Properties	
Knowledge level	Moderate
Knowledge level factor	0.9
Existing concrete compressive strength	34 MPa
Existing steel reinforcement tensile strength	220 MPa

Earthquake Code Parameters	
Seismic Zone	2
Seismic Zone Factor (***A_o***)	0.3
Building Importance Factor (***I***)	1.4
Soil Class	Z4 (T_A=0.2 T_B=0.9)
Live Load Participation Factor	0.6

Table 7-2 Story equivalent lateral forces and general data

Story	g_i	q_i	LLPC	w_i	H_i	$F_i(x)$	$F_i(y)$	X_m	Y_m	X_r	Y_r
2.	1243.82	219.87	0.60	1375.75	14.31	307.10	307.10	49.00	2.53	48.97	2.21
1.	2037.20	529.59	0.60	2354.95	9.54	329.78	329.78	48.98	2.69	48.85	1.85
Ground	2081.28	538.71	0.60	2404.50	4.77	168.36	168.36	48.94	2.70	48.86	1.64

Where;

g_i :The total dead load in the i'th story of the structure [ton].

q_i : The total live load in the i'th story of the structure [ton].

LLPC: Live load participation coefficient.

w_i :The weight of the i'th story of the structure calculated using live load participation coefficient [ton].

H_i : The height of the i'th story of the structure measured from the upper surface of the foundations (in structures with rigid concrete walls around the basement, the height of the i'th story from the ground floor) [m].

$F_i(x)$:The equivalent seismic load acting on the i'th story, in equivalent seismic load method (x direction) [ton].

$F_i(y)$: The equivalent seismic load acting on the i'th story, in equivalent seismic load method (y direction) [ton].

X_m, Y_m :The coordinates of the center of gravity of the story [m].

X_r, Y_r : The coordinates of the center of rigidity (stiffness) of the story [m].

7.2 Non-Linear Static Analysis

Static Pushover analysis is one of the methodologies used in the analysis of the building under dynamic loading (such as the earthquake). In this method, analysis is carried out depending on the expected performance of building. The behavior of the building is defined for earthquake design and acceptable maximum limits of the damage. Therefore, the load-bearing elements are dimensioned according to this criterion. In the analysis based on the performance, almost all of the materials forming the load-bearing elements in building (reinforced concrete and/or steel) make plastic deformation. This situation is defined as the plastic hinge occurrence in the elements. Solution is continued until structure become statically determinate.

Positive side of this analysis type is its characteristic to give results according to the expected performance. The elements which lose their load-bearing are seen step by step and alternately. The structure which has been solved with the other techniques can be easily evaluated with the analysis based on the performance. On the other hand, this analysis type also has negative sides. Firstly, it requires attention and very well established theoretical infrastructure. If the system has complex structure computer usage is definitely required.

Pushover analysis is a very useful methodology in the repairs and strengthening of the existing structures. In this methodology, it is possible to determine which elements of the structure will collapse earlier. Displacement of the structure under earthquake load can be estimated and the elements of the structure which require strengthening can be easily ascertained. This methodology has provided the opportunity of obtaining useful values to the engineers who try to find linear solutions with the equivalent earthquake loads by handling non-linear earthquake behaviors.

ATC-40 and FEMA 356 procedures were described in detail in Chapter 4. Before and after strengthening performance of GMNKL is determined by using two load pattern (Triangular and Uniform load pattern) as shown in Figure 7.1.

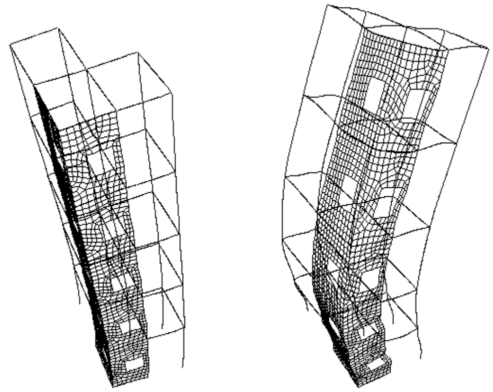


Figure 7-1 Triangular and Uniform load pattern

7.2.1 Performance Assessment Before Strengthening

Tables 7.3 and 7.4 show collapsed elements at performance point of building and difference in behavior depending on loading direction is also specified. These tables give the idealized yield point, performance point, ductility demand and collapsed elements at performance point. Shear force are given in ton and displacement are in meter. From Table 7.3 and Table 7.4 the case has been identified. Thus, negative earthquake directions F4, with uniform load pattern observed to be the worst case (Figure 7.2 and 7.3).

Table 7-3 + Earthquake direction in Triangular and Uniform load pattern before strengthening

Direction +		F1 Triangular	F1 Uniform	F2 Triangular	F2 Uniform	F3 Triangular	F3 Uniform	F4 Triangular	F4 Uniform
Yield Idealized	V (ton)	578	660	567	649	520	589	520	589
	Dis. (m)	0.0352	0.0369	0.0339	0.0356	0.0454	0.0463	0.0455	0.0463
Performance point	V (ton)	765	836	762	835	678	739	679	740
	Dis. (m)	0.1530	0.1530	0.152	0.152	0.184	0.184	0.184	0.184
Ductility Demand	Δu/Δy	4.35	4.15	4.48	4.27	4.05	3.97	4.04	3.97
Roof Dis.	θr (%)	0.0106	0.0106	0.0105	0.0105	0.0128	0.0128	0.0128	0.0128
at performance point	> Collapse	28	28	28	29	32	31	32	31
	% Collapse members	5.50	5.50	5.50	5.70	6.29	6.09	6.29	6.09
	% Collapse column of total # of column	11.67	11.67	11.67	12.08	13.33	12.92	13.33	12.92

Table 7-4 - Earthquake direction in Triangular and Uniform load pattern before strengthening

Direction		F1		F2		F3		F4	
-		Triangular	Uniform	Triangular	Uniform	Triangular	Uniform	Triangular	Uniform
	V (ton)	581	664	570	651	575	649	573	648
Yield Idealized	Dis. (m)	0.0355	0.0372	0.034	0.0358	0.0446	0.0453	0.044	0.045
	V (ton)	767	839	764	837	775	838	774	838
Performance point	Dis. (m)	0.153	0.153	0.152	0.152	0.172	0.171	0.172	0.171
Ductility Demand	$\Delta u/\Delta y$	4.31	4.11	4.47	4.25	3.86	3.77	3.91	3.80
Roof Dis.	θr (%)	0.0106	0.0106	0.0105	0.0105	0.0119	0.0119	0.0119	0.0119
at performance point	>Collapse	30	31	30	33	38	37	41	41
	% Collapse members	5.89	6.09	5.89	6.48	7.47	7.27	8.06	8.06
	% Collapse column of total # of column	12.50	12.92	12.50	13.75	15.83	15.42	17.08	17.08

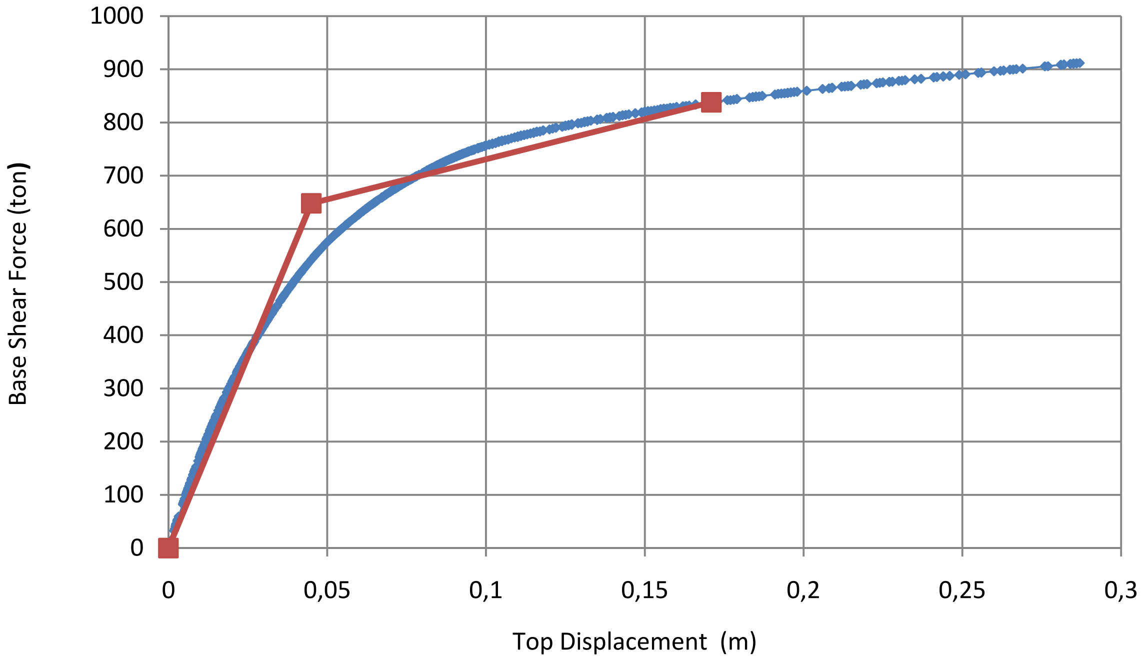


Figure 7-2 Pushover curve for existing building in X direction

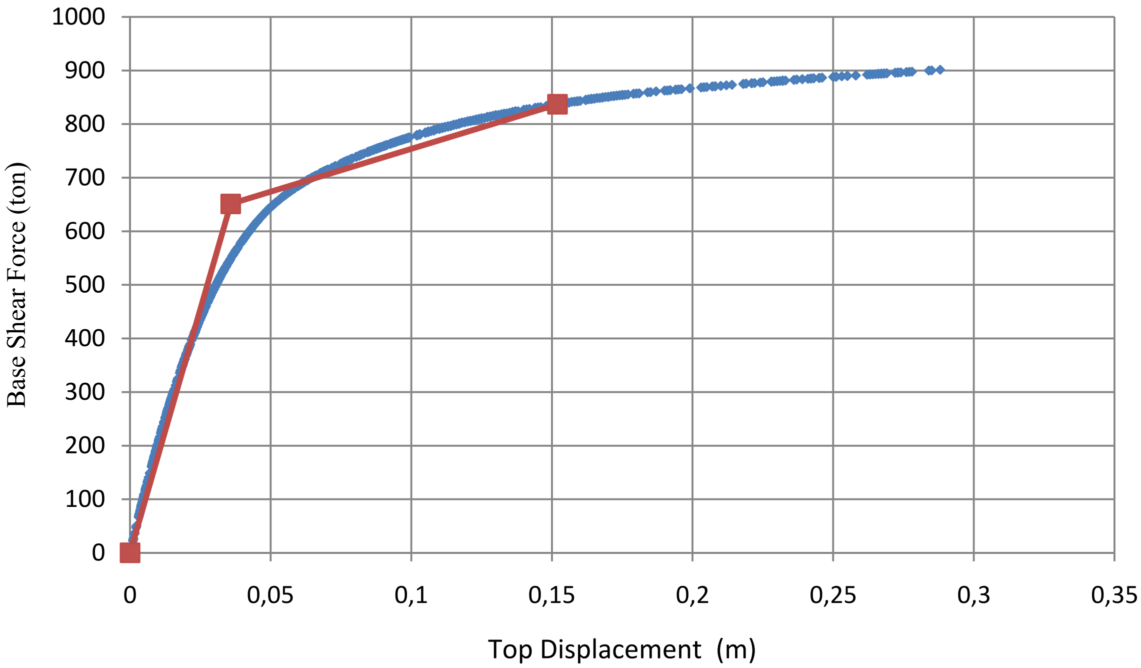


Figure 7-3 Pushover curve for existing building in Y direction

After performing pushover analysis and the Roof Displacement – Base Shear curve (Capacity Curve) obtained, target displacement for selected performance level has to be determined. The method described in the TEC 2007 was previously discussed in Chapter 4. According to the TEC 2007 earthquake code, if purpose of occupancy of the building is intensively and long-term occupied buildings and buildings preserving valuable goods (such as school buildings) (***I=1,4***) performance of building should be in immediate occupancy performance level. For any storey, for all vertical members, immediate occupancy limits should be satisfied. On the other hand, 10% of the beams at most are allowed to exceed the immediate occupancy limits for beams. Limitation of performance levels were mentioned in Chapter 4. In Tables 7.5 through 7.8 summaries of results for nonlinear procedure are presented. As it can be observed from Table 7.5 to Table 7.8, the existing system of the building under consideration cannot satisfy the target performance levels.

Table 7-5 Immediate Occupancy Performance Level in X Direction for Beams in Existing Structure

Story	Total number of beams	Beams not satisfying performance level		
		Number	Ratio %	Check
2	89	0	0.00	<10 %
1	88	0	0.00	<10 %
Ground	92	0	0.00	<10 %

Table 7-6 Immediate Occupancy Performance Level in X Direction for Columns in Existing Structure

Story	Total number of columns	Column not satisfying performance level		
		LS	CP	>C
2	80	0	0	15
1	80	2	0	10
Ground	80	0	0	16

Table 7-7 Immediate Occupancy Performance Level in Y Direction for Beams in Existing Structure

Story	Total number of beams	Beams not satisfying performance level		
		Number	Ratio %	Check
2	89	0	0.00	<10 %
1	88	0	0.00	<10 %
Ground	92	0	0.00	<10 %

Table 7-8 Immediate Occupancy Performance Level in Y Direction for Columns in Existing Structure

Story	Total number of beams	Column not satisfying performance level		
		LS	CP	>C
2	80	0	0	12
1	80	0	0	10
Ground	80	0	0	11

- **Target Displacement in the X and Y Directions**

Target displacement calculation is performed by drawing the code and capacity spectrum on the same graph. For both orthogonal directions, natural vibration periods are less than the T_B=0.9 sec of the code spectrum. Hence, an iterative procedure is applied as described in Chapter 4 and the target displacements are found as 0.135m in the X direction and 0.113m in the Y direction respectively (Figure 7.4 and 7.5).

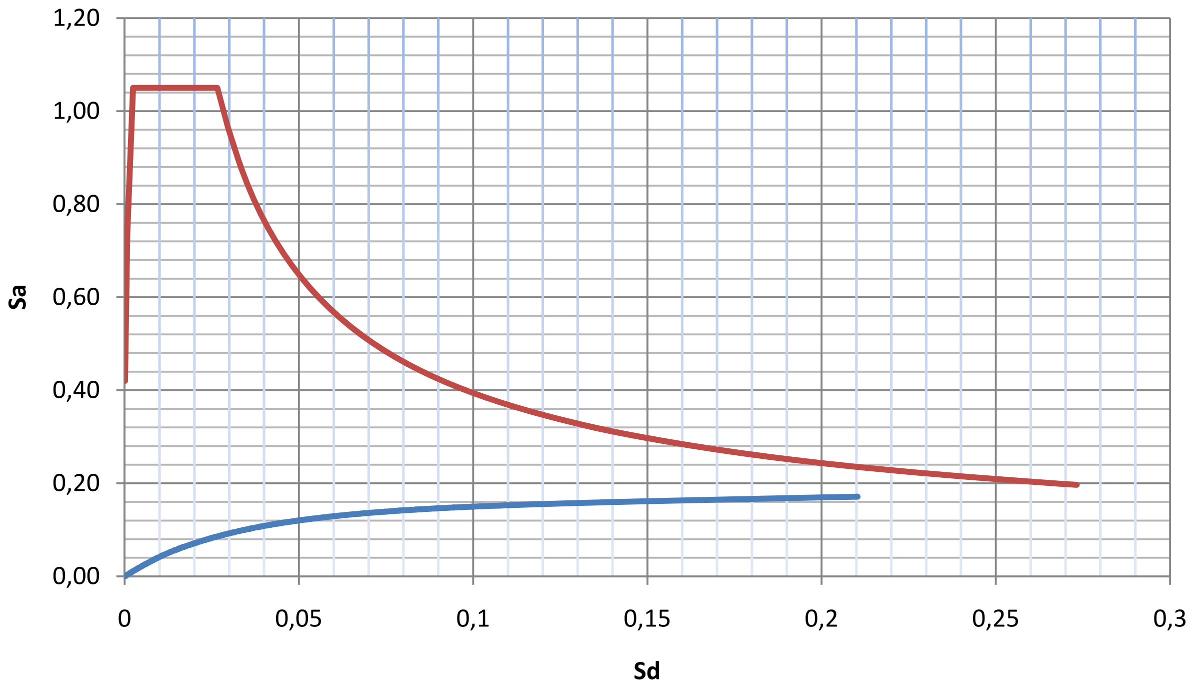


Figure 7-4 Target displacement for X direction

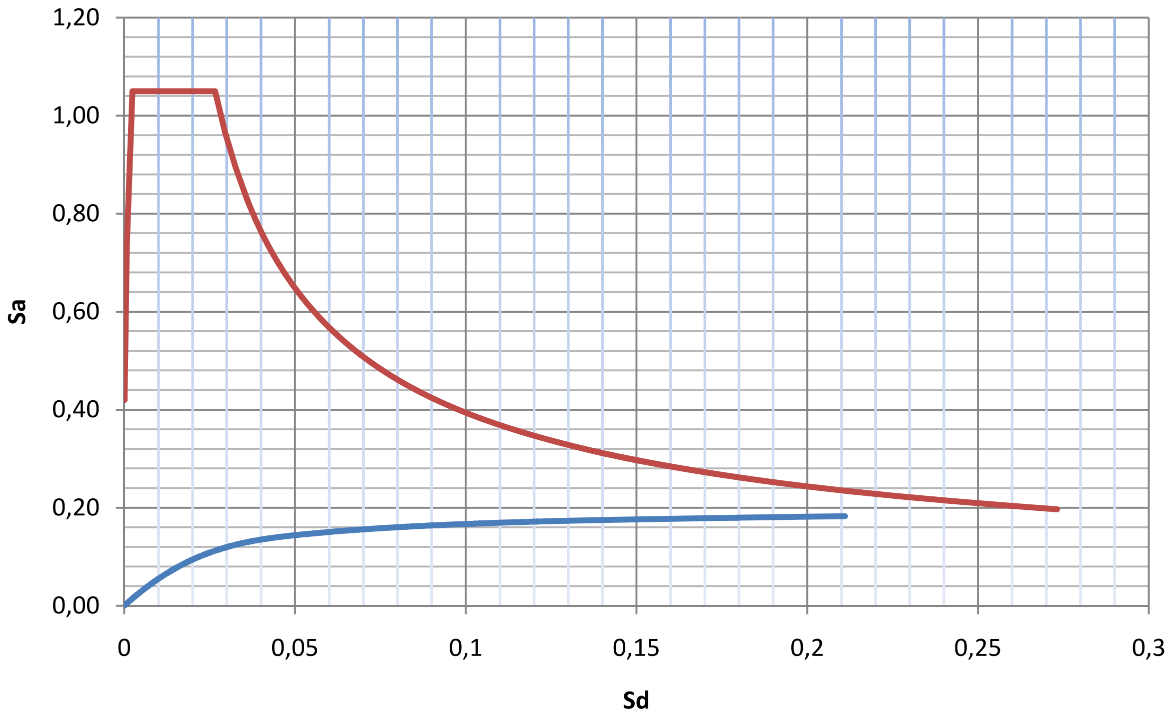


Figure 7-5 Target displacement for Y direction

7.2.2 Performance Assessment After Strengthening

7.2.2.1 Column Jacketing

One popular solution to the problem of how to strengthen old reinforced concrete (RC) structures is to place jackets around the structural elements. Jacketing was the most common technique used to increase the axial, flexural, and shear strength of existing elements. Increases in ductility and stiffness were also achieved. Jacketing was performed by adding longitudinal and transverse reinforcement or a welded wire mesh surrounding the original section and covering it with new cast in place concrete. To be able to develop yield in the longitudinal bars, continuity had to be provided at the ends of the element.

In this section all detected weak columns of existing structure have been strengthened 15 cm in their major and minor directions by jacketing techniques. For each story forty eight columns have been strengthened and selected strengthened columns are as shown in Figure 7.6. All details of location of strengthened columns plans are given in Appendix B for each story. Figure 7.7 and 7.8 show capacity curves after jacketing columns in X and Y direction.

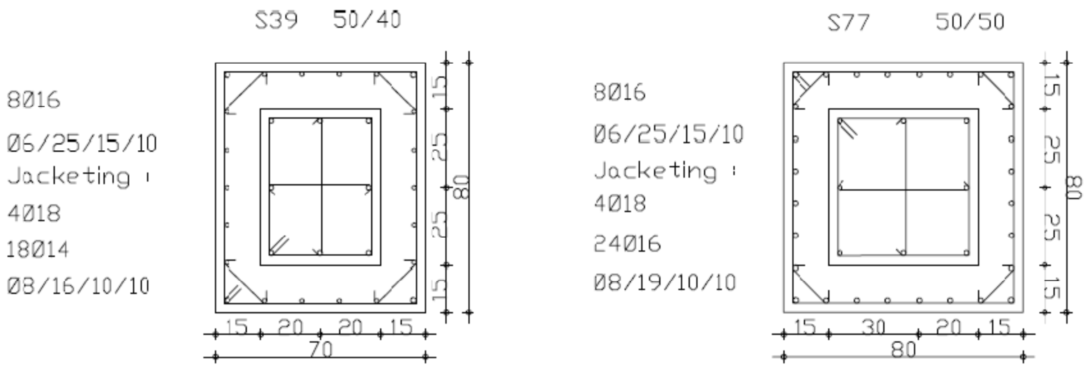


Figure 7-6 Column size and diameter of reinforcement bars after jacketing column

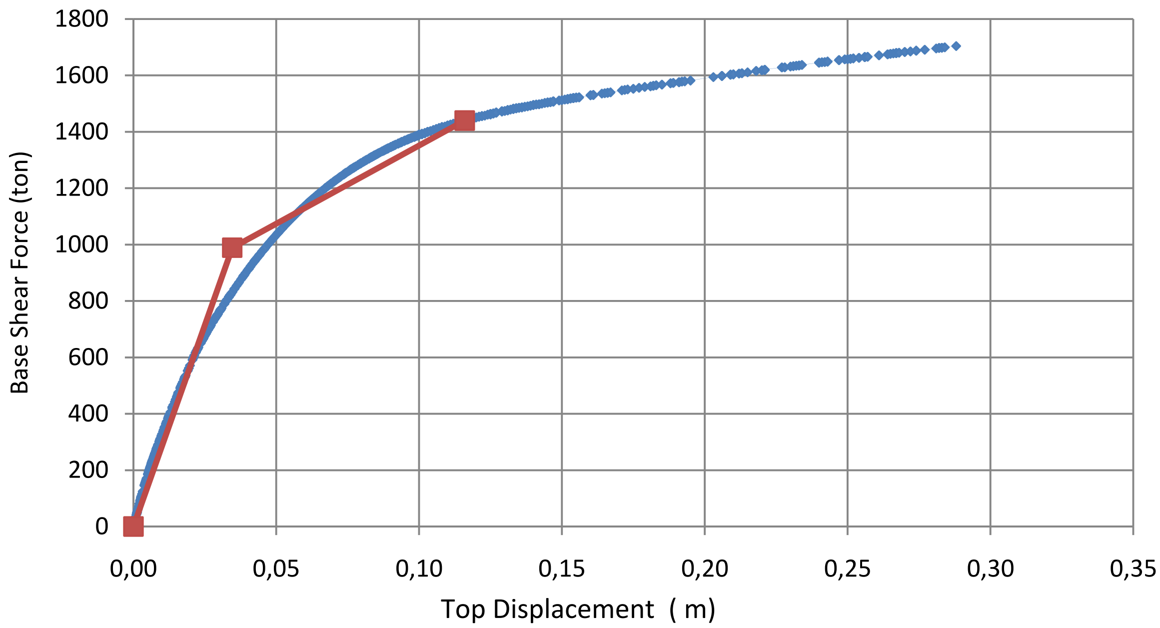


Figure 7-7 Pushover curve after jacketing columns in X direction

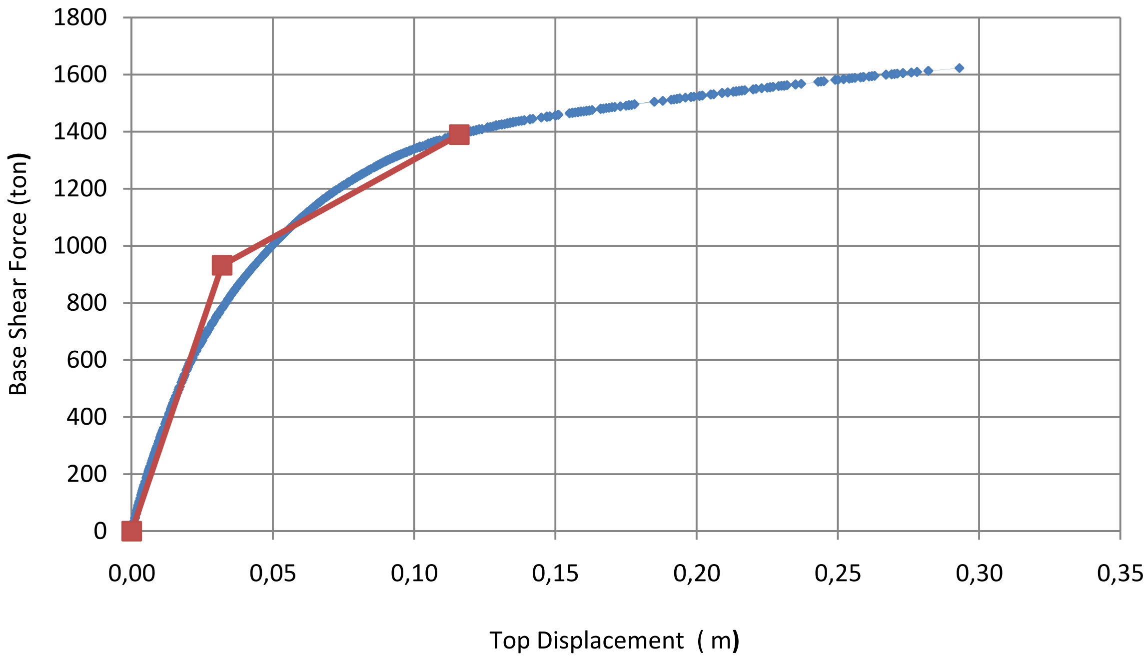


Figure 7-8 Pushover curve after jacketing columns in Y direction

After strengthening with jacketing, it is observed that according to the nonlinear static analysis the performance level of GMNKL has not become immediate occupancy performance level. This is due to collapsing of 4 columns in Y direction. In Table 7.9 to Table 7.12 member number and performance level limits were checked in each earthquake direction, which do not satisfied the performed load.

Table 7-9 Immediate Occupancy Performance Level in X Direction for Beams in Strengthening with Jacketing Structure

Story	Total number of beams	Beams not satisfying performance level		
		Number	Ratio %	Check
2	89	0	0.00	<10 %
1	88	0	0.00	<10 %
Ground	92	0	0.00	<10 %

Table 7-10 Immediate Occupancy Performance Level in X Direction for Columns in Strengthening with Jacketing Structure

Story	Total number of columns	Column not satisfying performance level		
		LS	CP	>C
2	80	0	0	0
1	80	0	0	0
Ground	80	0	0	0

Table 7-11 Immediate Occupancy Performance Level in Y Direction for Beams in Strengthening with Jacketing Structure

Story	Total number of beams	Beams not satisfying Performance level		
		Number	Ratio %	Check
2	89	0	0.00	<10 %
1	88	0	0.00	<10 %
Ground	92	0	0.00	<10 %

Table 7-12 Immediate Occupancy Performance Level in Y Direction for Columns in Strengthening with Jacketing Structure

Story	Total number of columns	Column not satisfying performance level		
		LS	CP	>C
2	80	0	0	1
1	80	0	0	1
Ground	80	0	0	2

Table 7-13 Story equivalent lateral forces and general data

Story	g_i	q_i	LLPC	w_i	H_i	$F_i(x)$	$F_i(y)$	X_m	Y_m	X_r	Y_r
2.	1429.80	218.30	0.6	1560.78	14.31	344.53	344.53	49.00	2.45	48.95	1.11
1.	2203.34	525.74	0.6	2518.79	9.54	349.56	349.56	48.99	2.61	48.92	1.31
Ground	2243.66	534.97	0.6	2564.64	4.77	177.96	177.96	48.95	2.61	48.91	1.54

Storey masses, mass center coordinates and mass moment of inertias are given in Table 7.13. As it is shown in figures below, after strengthening of existing structure target displacement reduced with an increase in base shear force. Figure 7.9 and 7.10 indicate show the effect of jacketing columns on performance points after and before strengthening with jacketing. There are no bearing members collapsing after strengthening of existing structure with jacketing methods up to performance points.

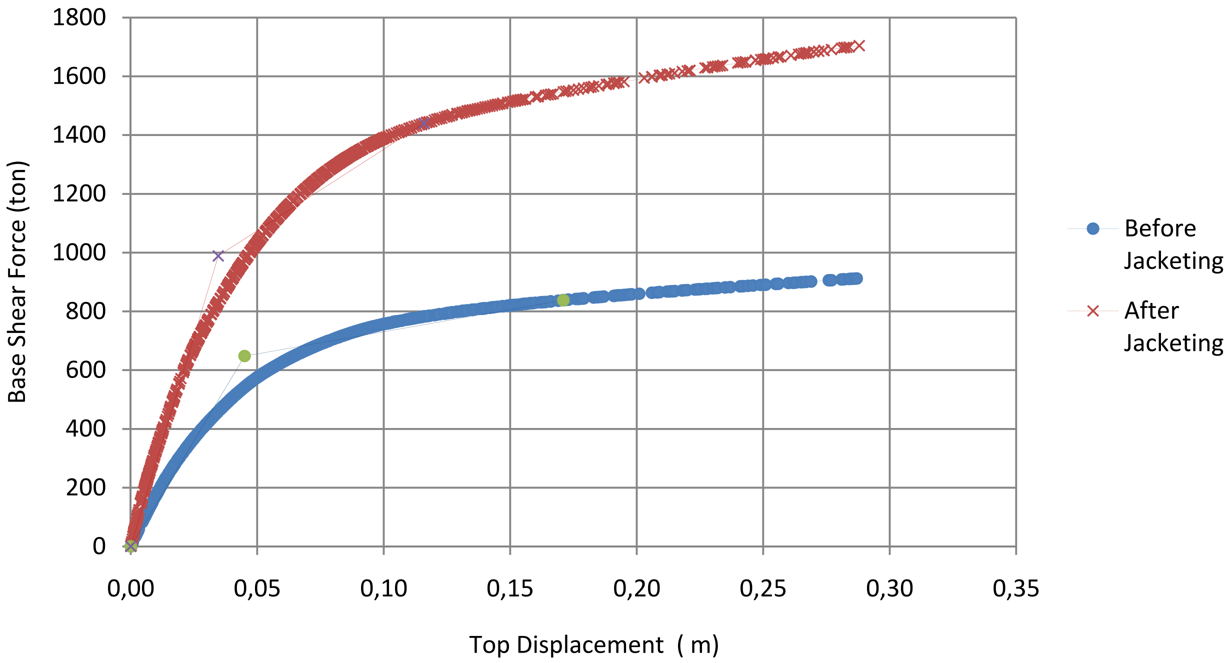


Figure 7-9 Before and after nonlinear static structural behavior with jacketing of GMNKL in X direction

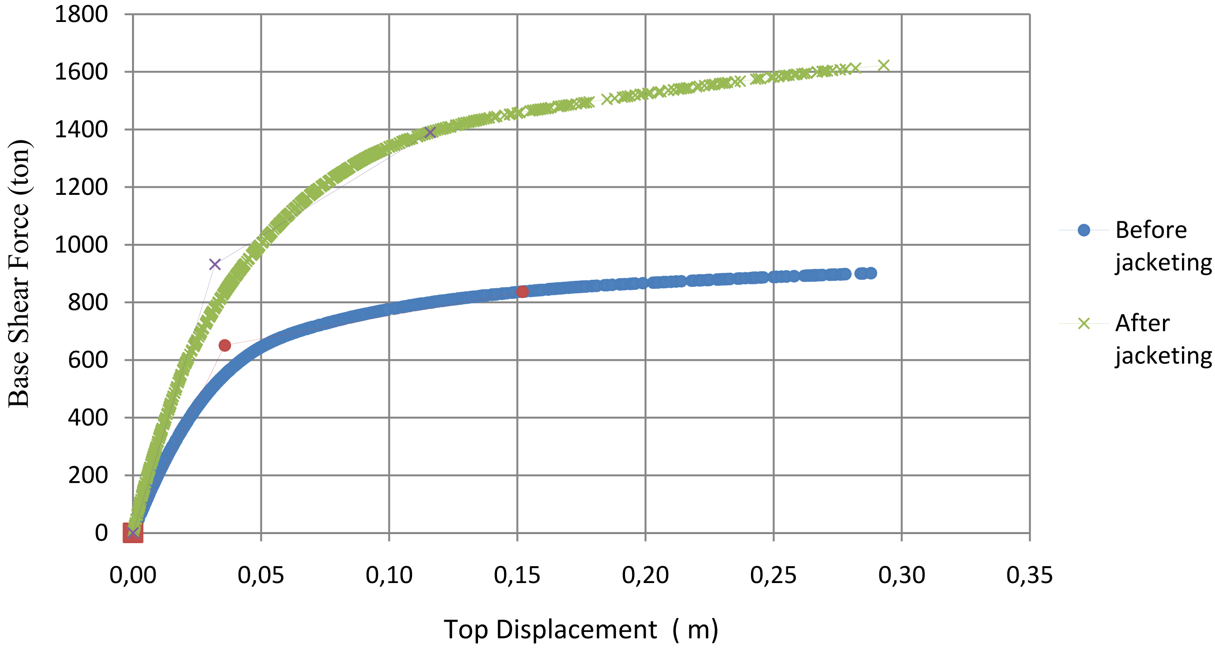


Figure 7-10 Before and after nonlinear static structural behavior with jacketing of GMNKL in Y direction

- **Target Displacement in the X and Y Directions**

Target displacement of the building calculation was performed using the capacity curves and the procedure is defined in the 2007 Turkish Earthquake Code. In Figure 7.11 and Figure 7.12 target displacements are found as 0.087 m in the X direction and 0.09 m in the Y direction, respectively.

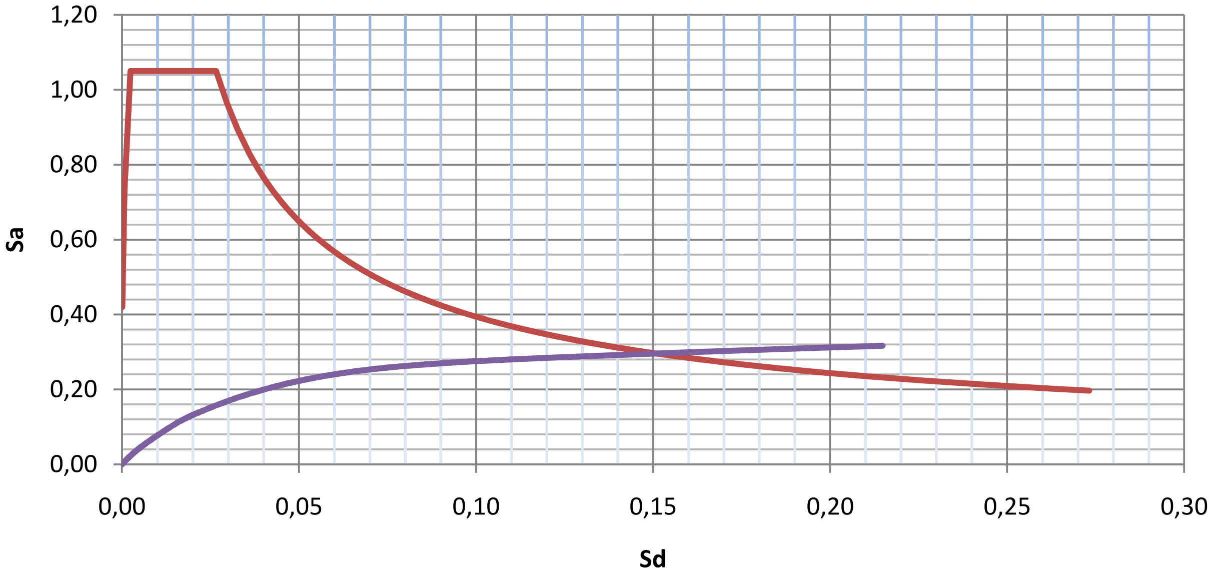


Figure 7-11 Target displacement for X direction

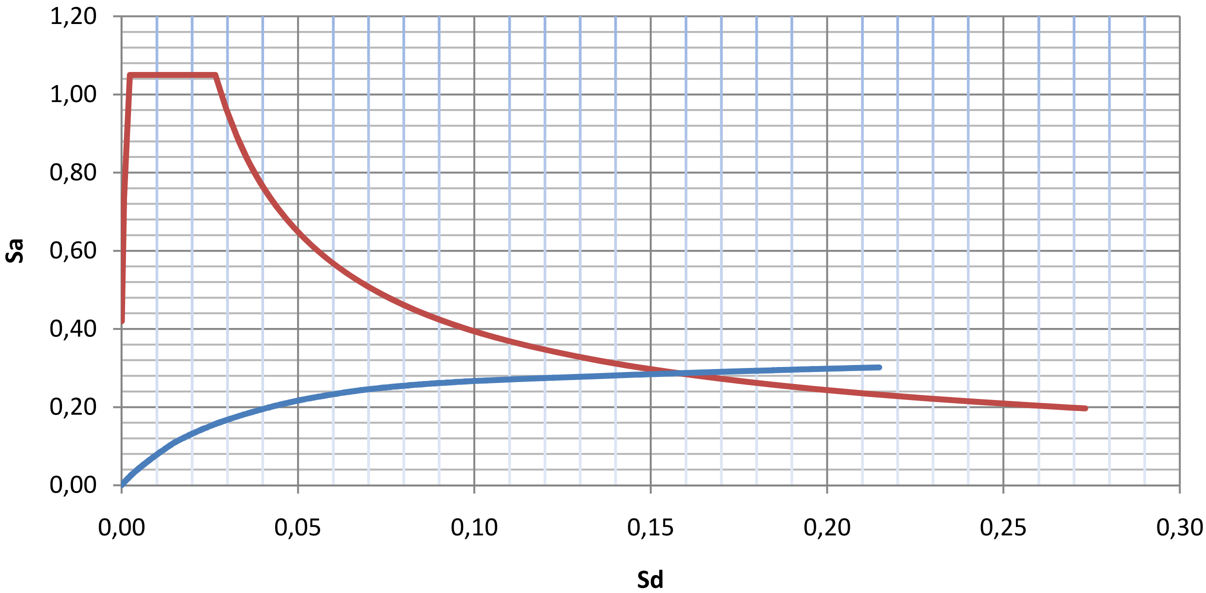


Figure 7-12 Target displacement for Y direction

7.2.2.2 Shear Wall

Concrete shear walls were often used to eliminate stiffness eccentricities in a building or to increase lateral load carrying capacity. The new walls were located in the perimeter of the structure thereby reducing interior interference. Structural walls were attached to existing columns whenever possible so that gravity forces would reduce the uplift generated at the ends of the wall due to overturning moments as lateral loads applied.

There are no shear walls in the structural system. This makes the building very flexible under the effect of lateral forces. Adding new shear walls into the existing moment resisting frames of the case study building is the basic retrofitting system. Area of shear walls and area ratio of shear wall for each story are given in Table 7.14. Added shear walls were 35x162 cm in Y direction, 35x700 cm in X direction in ground and 1st floor, 30x162 cm in Y direction and 30x700 cm in X direction in 2nd story.

Table 7-14 Area and ratio of shear walls

Story	Typical Floor Area (m^2)	Area of Shear Walls (m^2)	Shear Wall Area Ratio (%)
2	1600	20.04	1.25
1	1600	23.38	1.46
Ground	1600	23.38	1.46

The nonlinear static behavior of the structure after shear wall in Y and X earthquake directions pushover curves are given in Figure 7.13 and 7.14. Storey masses, mass center coordinates and mass moment of inertias are given in Table 7.19. Performance level of structure become immediate occupancy level after adding the shear wall. All bearing members are in IO level, shown in Table 7.15 to 7.18.

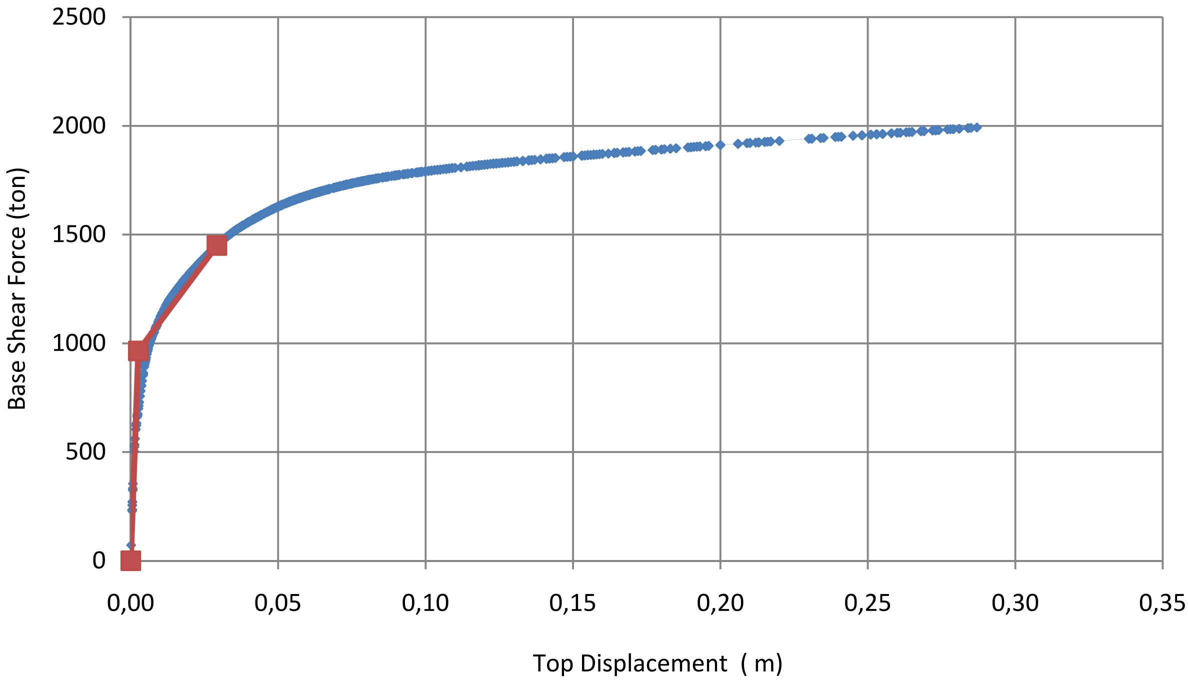


Figure 7-13 Pushover curve after shear wall in X direction

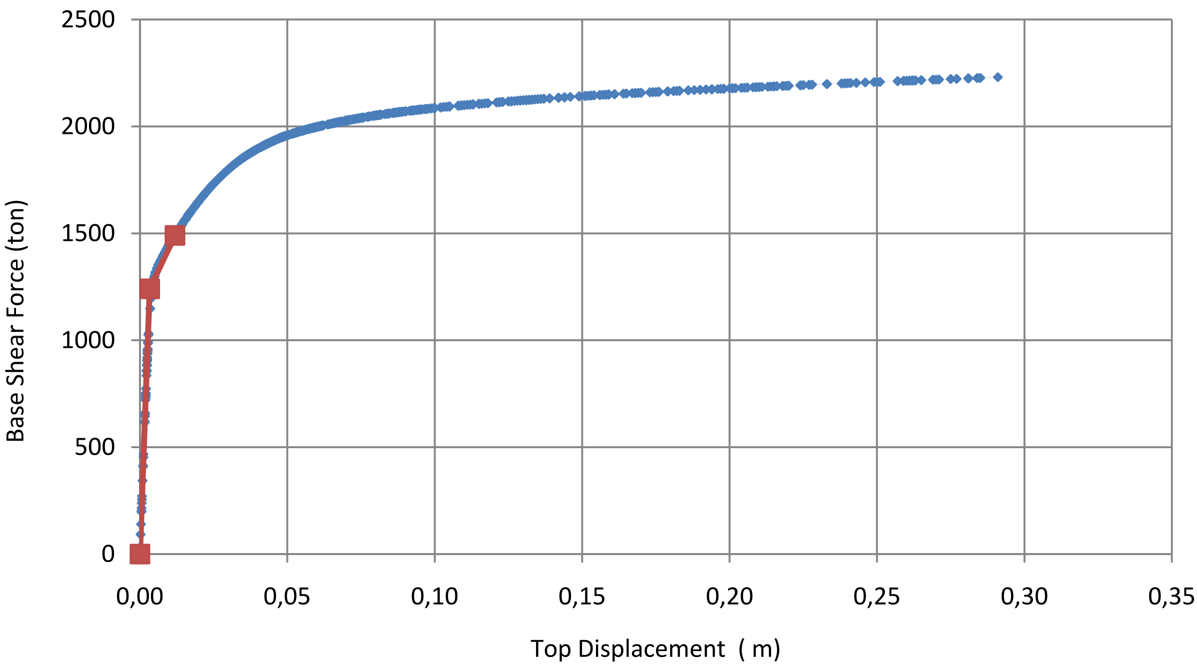


Figure 7-14 Pushover curve after shear wall in Y direction

Table 7-15 Immediate Occupancy Performance Level in X Direction for Beams in Strengthening with Shear Wall Structure

Story	Total number of beams	Beams not satisfying performance level		
		Number	Ratio %	Check
2	89	0	0.00	<10 %
1	88	0	0.00	<10 %
Ground	92	0	0.00	<10 %

Table 7-16 Immediate Occupancy Performance Level in X Direction for Columns in Strengthening with Shear Wall Structure

Story	Total number of columns	Column not satisfying performance level		
		LS	CP	>C
2	80	0	0	0
1	80	0	0	0
Ground	80	0	0	0

Table 7-17 Immediate Occupancy Performance Level in Y Direction for Beams in Strengthening with Shear Wall Structure

Story	Total number of beams	Beams not satisfying Performance level		
		Number	Ratio %	Check
2	89	0	0.00	<10 %
1	88	0	0.00	<10 %
Ground	92	0	0.00	<10 %

Table 7-18 Immediate Occupancy Performance Level in Y Direction for Columns in Strengthening with Shear Wall Structure

Story	Total number of columns	Column not satisfying performance level		
		LS	CP	>C
2	80	0	0	0
1	80	0	0	0
Ground	80	0	0	0

Table 7-19 Story equivalent lateral forces and general data

Story	g_i	q_i	LLPC	w_i	H_i	$F_i(x)$	$F_i(y)$	X_m	Y_m	X_r	Y_r
2.	1447.50	219.90	0.6	1579.44	14.31	447.38	446.55	48.96	2.49	48.99	2.06
1.	2274.61	529.53	0.6	2593.33	9.54	492.32	460.53	48.97	2.69	48.95	4.47
Ground	2311.43	534.92	0.6	2632.38	4.77	249.96	233.80	48.94	2.69	48.96	3.47

After strengthening of the existing structure with shear walls, the performance level of structure is increased. As a result of implemented strengthening system, almost entire lateral load applied to the structure was resisted by shear walls. As it can be observed from Figure 7.15 and 7.16 in X and Y direction of earthquake, after adding the shear wall the structure became stronger.

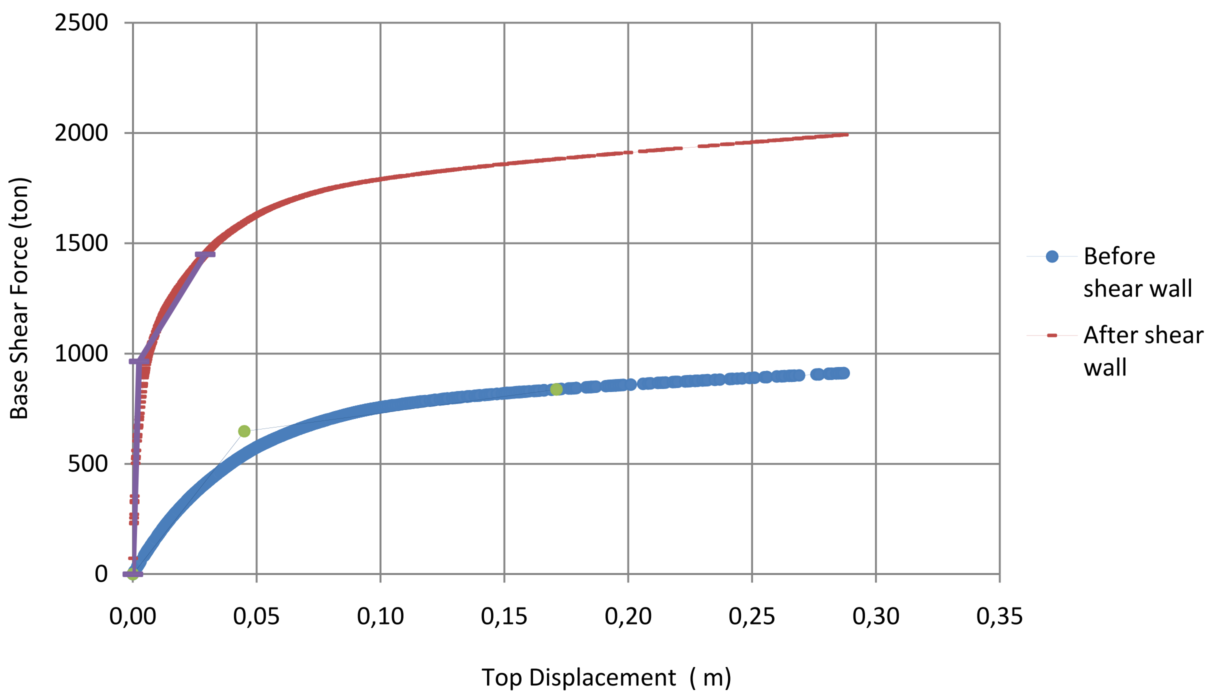


Figure 7-15 Before and after nonlinear static structural behavior with jacketing of GMNKL in X direction

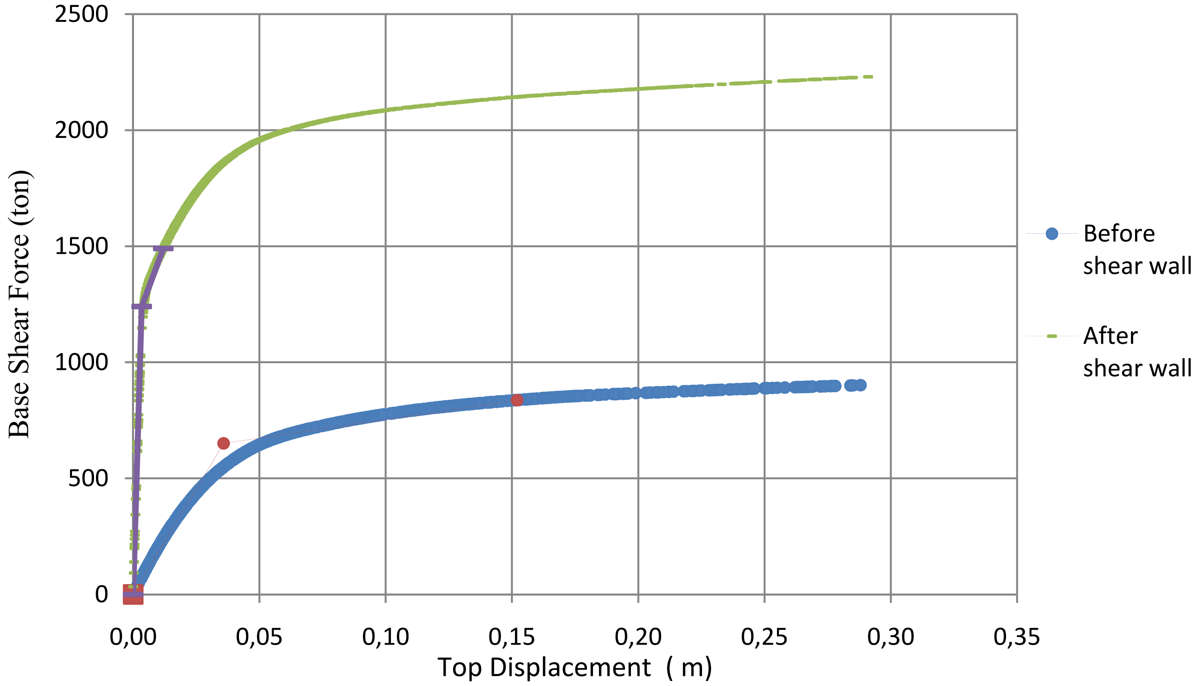


Figure 7-16 Before and after nonlinear static structural behavior with shear wall of GMNKL in Y direction

- **Target Displacement in the X and Y Directions**

Target displacements are calculated as explained previously and found as 0.016 m in the X direction and 0.014 m in the Y direction respectively as shown in Figure 7.17 and 7.18.

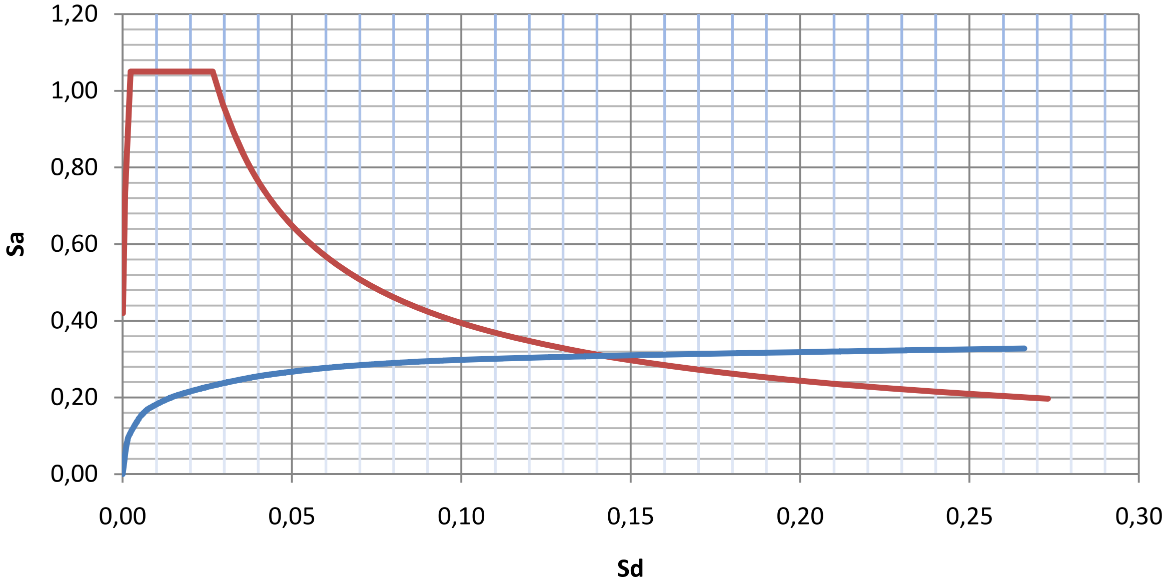


Figure 7-17 Target displacement for X direction

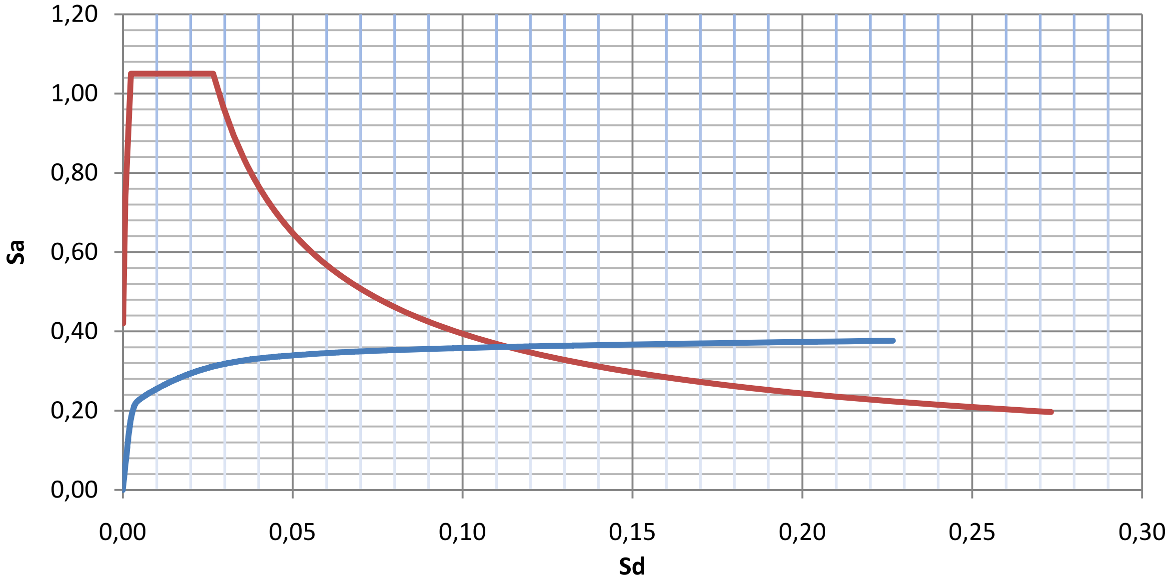


Figure 7-18 Target displacement for Y direction

7.3 Linear Elastic Analysis

Performance based design provides the behavior of the structure and helps the designers to have better idea about building's damages at different levels of earthquakes. This would also provide to select correct strengthening methods with better decision making for existing buildings [Yalciner and Yardimci, 2008]. 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 2007. In Chapter 4, limitation of damage limits of the structural elements classified as minimum damage limit (MN), safety limit (SF) and collapse limit (CL), demand/capacity ratios for reinforced concrete beams in Figure 4.4 and demand/capacity ratios for reinforced concrete columns in Figure 4.5. All before and strengthening stage linear analysis graphs are given in Appendix A.

7.3.1 Performance Assessment Before Strengthening

Table 7.19 shows collapse elements according to 2007 Turkish earthquake codes criteria of building and the differences between earthquake direction results before strengthening of GMNKL by using Linear analysis. In linear performance analysis existing structure performance was in collapse region. In the X direction, approximately 37% in 2nd, 75% to 89% in 1st, 79% to 89% in ground floor of beams are beyond the expected limit. In the Y direction, nearly 43% in 2nd, 80% to 85% in 1st, 83% to 82% in ground floor of beams are beyond the expected limit. As it was stated in Chapter 4, for the immediate occupancy performance level, in the Turkish Earthquake Code 2007 does not allow any column beyond the minimum damage range. However, in existing structure, in X direction in 2nd floor 26%, in 1st floor 23% and in ground floor 6%, in Y direction in 2nd floor nearly 35% to 29%, in 1st floor 28% to 21% and in ground floor 5% columns collapse.

Table 7-20 Earthquake direction and performance level of members before strengthening

Direction	Story	Element Type	Min. Damage	%	Evident Damage	%	Further Damage	%	Collapse	%
+ Ex	2	Beam	10	11	30	34	16	18	33	37
		Column	0	0	40	50	19	24	21	26
	1	Beam	3	3	8	9	11	13	66	75
		Column	0	0	44	55	18	23	18	23
	Ground	Beam	7	8	7	8	5	5	73	79
		Column	0	0	66	83	9	11	5	6
- Ex	2	Beam	10	11	29	33	17	19	33	37
		Column	0	0	40	50	20	25	20	25
	1	Beam	3	3	3	3	4	5	78	89
		Column	0	0	43	54	19	24	18	23
	Ground	Beam	7	8	1	1	2	2	82	89
		Column	0	0	66	83	9	11	5	6
+ Ey	2	Beam	5	6	34	38	12	13	38	43
		Column	0	0	38	48	14	18	28	35
	1	Beam	3	3	2	2	8	9	75	85
		Column	0	0	46	57	12	15	22	28
	Ground	Beam	6	7	3	3	7	8	76	83
		Column	0	0	74	93	2	3	4	5
- Ey	2	Beam	8	9	29	33	14	16	38	43
		Column	0	0	41	51	16	20	23	29
	1	Beam	3	3	12	14	3	3	70	80
		Column	0	0	51	64	12	15	17	21
	Ground	Beam	7	8	5	5	5	5	75	82
		Column	0	0	70	88	5	6	5	6

7.3.2 Performance Assessment After Strengthening

Although strengthening was carried out by using jacketing and shear wall, in linear analysis performance, stage of building was in collapse region. Most of bearing members were strengthened, but their ***r*** ratios were still above limitation. Especially for beams limitation is very ruthless in TEC 2007.

7.3.2.1 Column Jacketing

In X direction, in 1st and ground floor there were no collapse column in spite of 2nd floor 5% column collapse. Most of beams are in collapse stage. These are in 2nd floor 39% to 44% , 1st floor 74% to 86%, ground floor 72% to 84% collapse. In Y direction, only ground floor columns are not in collapse stage. In 1st floor almost all columns have damages. All performances of the members are given in detail in Table 7.20.

Table 7-201 Earthquake direction and performance level of members after strengthening with jacketing

+ **Ex**	Story	Element Type	Min. Damage	%	Evident Damage	%	Further Damage	%	Collapse	%
	2	Beam	4	4	33	37	17	19	35	39
		Column	48	60	22	28	6	8	4	5
	1	Beam	4	5	12	14	7	8	65	74
		Column	0	0	77	96	3	4	0	0
	Ground	Beam	8	9	11	12	7	8	66	72
		Column	70	88	10	13	0	0	0	0

- Ex

2	Beam	6	7	30	34	14	16	39	44
	Column	48	60	22	28	6	8	4	5
1	Beam	3	3	3	3	6	7	76	86
	Column	0	0	78	96	2	3	0	0
Ground	Beam	7	8	2	2	6	7	77	84
	Column	69	86	11	14	0	0	0	0

+ Ey

2	Beam	19	21	22	25	9	10	39	44
	Column	2	3	74	93	1	1	3	4
1	Beam	6	7	13	15	17	19	52	59
	Column	0	0	80	100	0	0	0	0
Ground	Beam	11	12	15	16	10	11	56	61
	Column	74	93	6	8	0	0	0	0

- Ey

2	Beam	13	15	28	31	4	4	44	49
	Column	2	3	70	88	2	3	6	8
1	Beam	5	6	15	17	16	18	52	59
	Column	0	0	76	95	0	0	4	5
Ground	Beam	8	9	14	15	13	14	57	62
	Column	75	94	5	6	0	0	0	0

7.3.2.2 Shear Wall

In this stage, as shown in Table 7.21 both columns and shear walls are in minimum and evident damage. However in both earthquake direction, beams are in collapse region. Based on TEC 2007, performance of GMNKL is collapse, even though all column and shear walls are not in collapse stage.

Table 7-212 Earthquake direction and performance level of members after strengthening with shear wall

+ Ex	Story	Element Type	Min. Damage	%	Evident Damage	%	Further Damage	%	Collapse	%
	2	Beam	78	88	4	4	1	1	6	7
		Column	72	90	6	8	0	0	2	3
		Shearwalls	8	80	2	20	0	0	0	0
	1	Beam	68	77	10	11	2	2	8	9
		Column	78	98	2	3	0	0	0	0
		Shearwalls	10	100	0	0	0	0	0	0
	Ground	Beam	73	79	7	8	4	4	8	9
		Column	80	100	0	0	0	0	0	0
		Shearwalls	10	100	0	0	0	0	0	0
- Ex										
	2	Beam	75	84	4	4	1	1	9	1
		Column	72	90	6	8	0	0	2	3
		Shearwalls	8	80	2	20	0	0	0	0
	1	Beam	63	72	9	10	2	2	14	16
		Column	78	98	2	3	0	0	0	0
		Shearwalls	10	100	0	0	0	0	0	0
	Ground	Beam	65	71	5	5	6	7	16	17
		Column	80	100	0	0	0	0	0	0
		Shearwalls	10	100	0	0	0	0	0	0
+ Ey										
	2	Beam	88	99	0	0	0	0	1	1
		Column	80	100	0	0	0	0	0	0
		Shearwalls	6	60	4	40	0	0	0	0
	1	Beam	78	89	0	0	0	0	10	11
		Column	80	100	0	0	0	0	0	0
		Shearwalls	10	100	0	0	0	0	0	0
	Ground	Beam	80	87	0	0	1	1	11	12
		Column	80	100	0	0	0	0	0	0
		Shearwalls	10	100	0	0	0	0	0	0

- Ey										
2	Beam	89	100	0	0	0	0	0	0
	Column	80	100	0	0	0	0	0	0
	Shearwalls	6	60	4	40	0	0	0	0
1	Beam	75	85	2	2	0	0	11	13
	Column	80	100	0	0	0	0	0	0
	Shearwalls	10	100	0	0	0	0	0	0
Ground	Beam	75	82	3	3	2	2	12	13
	Column	80	100	0	0	0	0	0	0
	Shearwalls	10	100	0	0	0	0	0	0

7.4 Comparison of ideCAD and SAP 2000

In this section, ideCAD 5.511 and SAP 2000 programs are compared by using 3 storey frame as shown in Figure 7.19. SAP 2000 is the most famous engineering program compared to ideCAD5.511. Because it has too many functions and distributed in the world wide and known as most trustworthy program. However, SAP 2000 program needs more time for analysis of structures. Modeling part of SAP 2000 is more difficult than ideCAD 5.511.

ideCAD structural 5.511 is an integrated analysis, design and detailing software for reinforced concrete buildings especially developed for structural engineering designers. ideCAD Structural 5.511 covers all phases relevant to reinforced concrete constructions, from dynamic analyses to design of reinforced concrete cross-sections. Furthermore, reinforcement details are automatically generated [ideCAD].

Creation and modification of the model, execution of the analysis, and checking and optimization of the design, and production of the output are all accomplished using this single interface. A single structural model can be used for a wide variety of different types of analysis and design. Analysis capabilities include static nonlinear analysis for material and geometric effects, including pushover analysis; nonlinear time-history analysis by modal superposition or direct integration; and buckling analysis [SAP 2000].

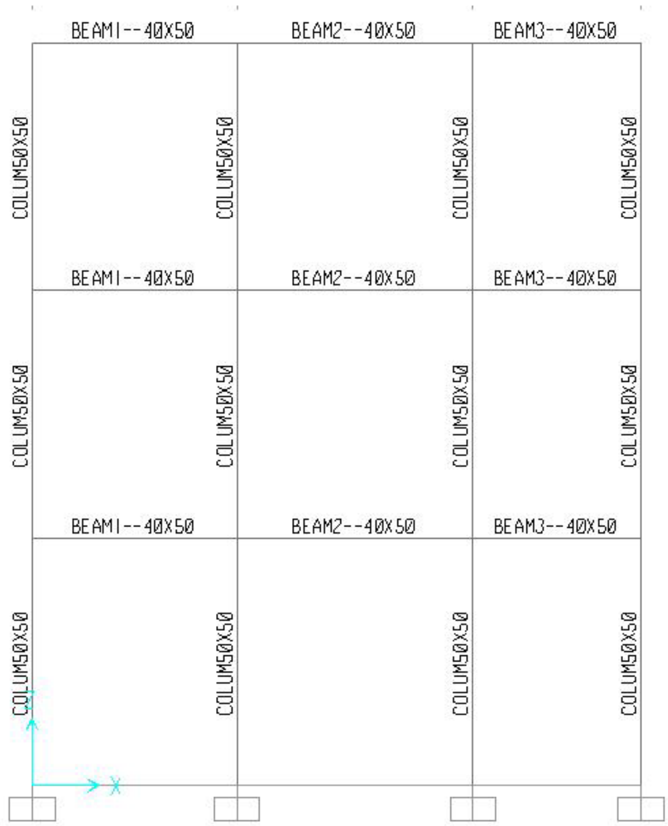


Figure 7-19 3 storey frame

This frame consist of A-6 to D-6 axes, as shown in Appendix B. Columns are 50x50 cm, beams 40x50 cm and steel reinforcing bars were given in details in Chapter 6. After analysis by using SAP 2000 and ideCAD 5.511 programs, capacity cures and idealized curves are given in Figure 7.20 and 7.21 respectively. In Table 7.22 differentiate performance point results are given. And in Figure 7.22 and 7.23 shows performance level of elements up to performance point of each program nonlinear static results.

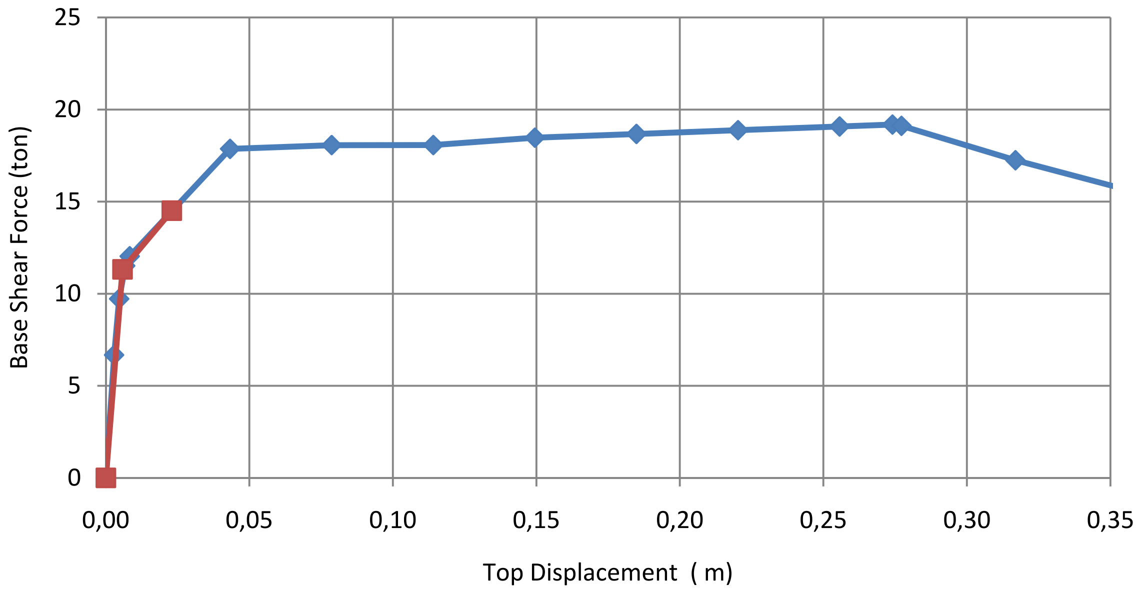


Figure 7-19 SAP 2000 capacity curve and idealized curve

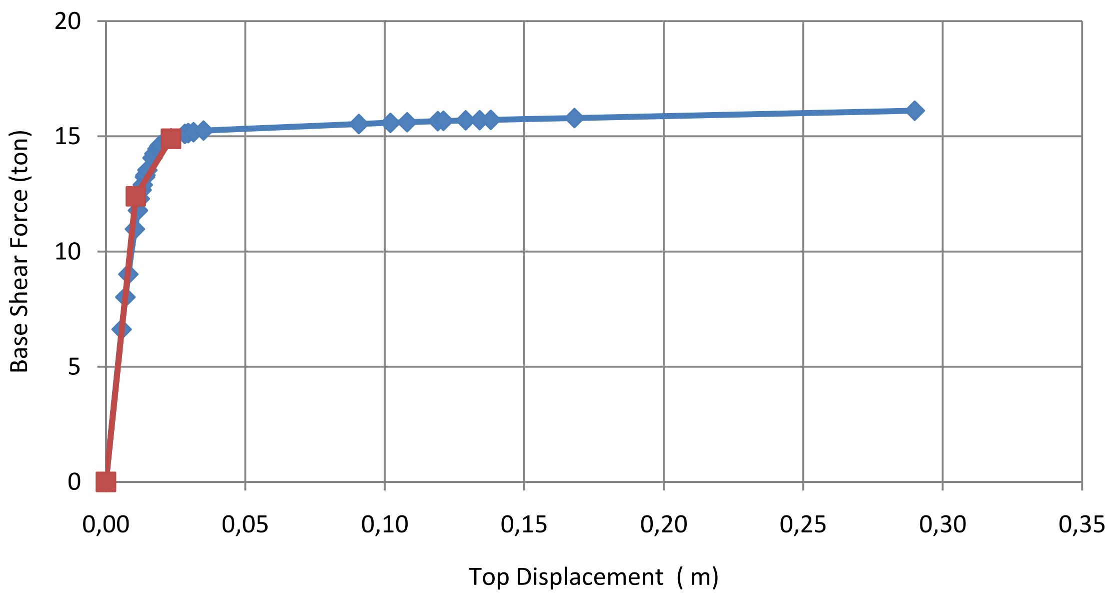


Figure 7-20 ideCAD 5.511 capacity curve and idealized curve

Table 7-22 Performance point result

	Base Shear Force (ton)	Top Displacement (m)
SAP 2000	14.512	0.0230
ideCAD	14.900	0.0233

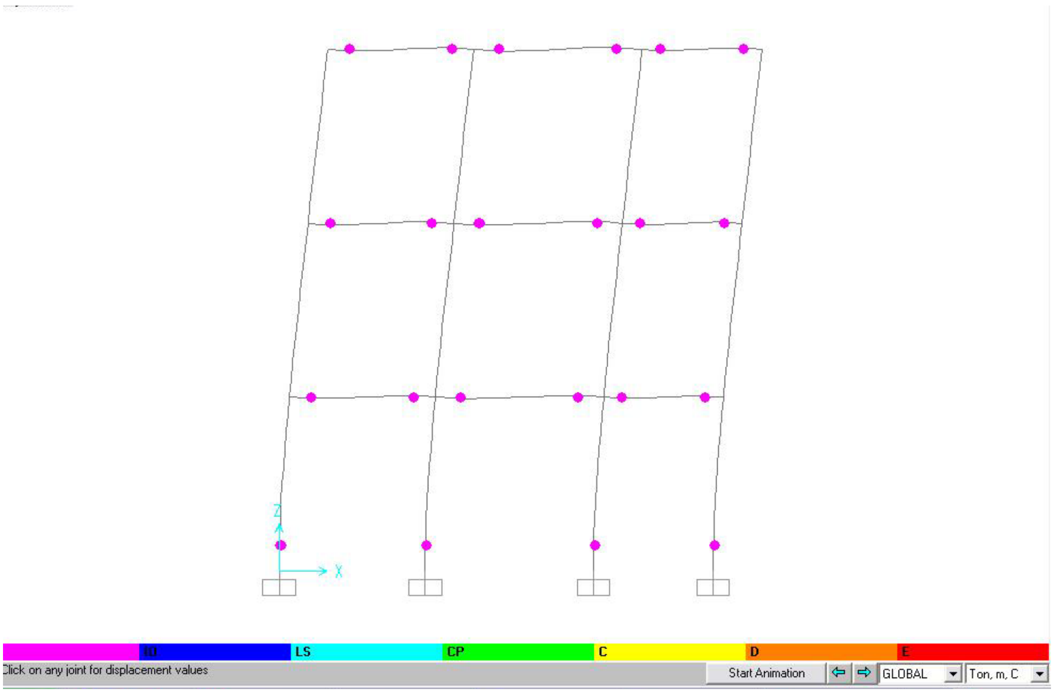


Figure 7-21 Performace level of element in SAP 2000

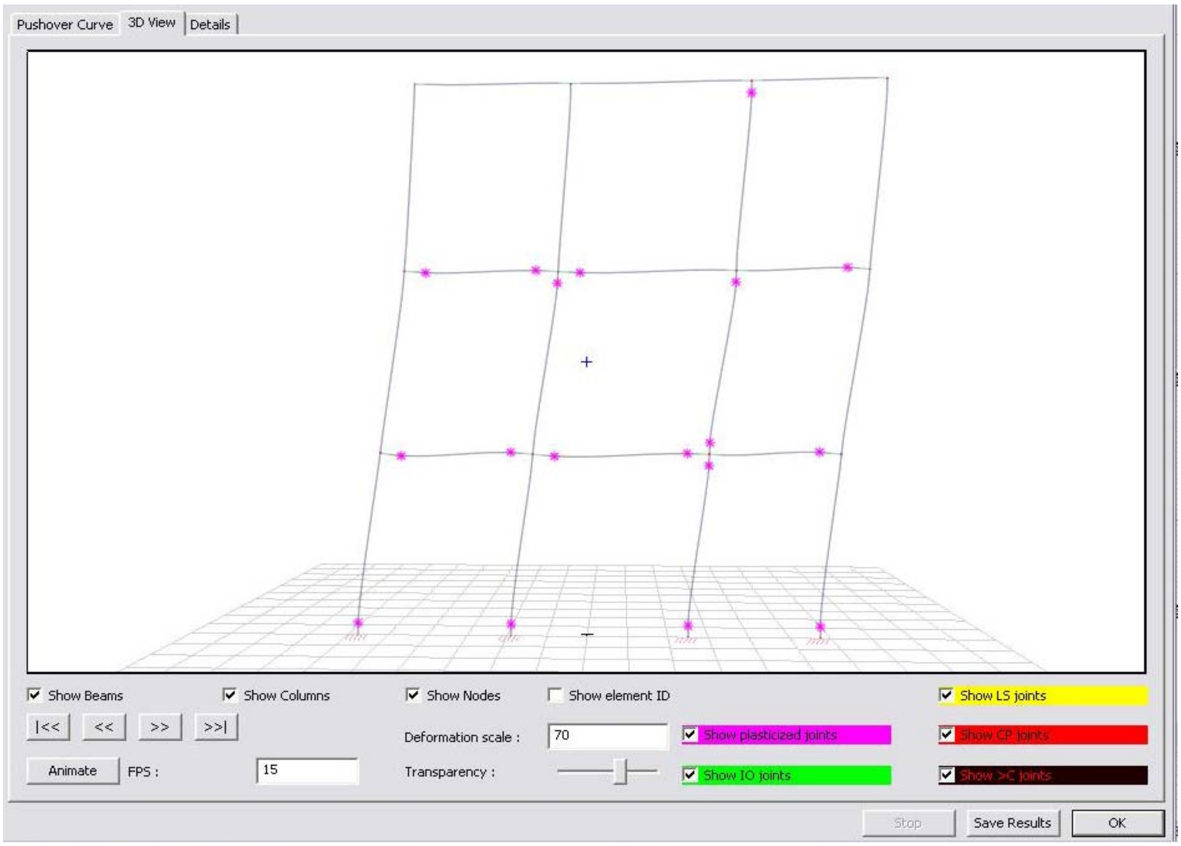


Figure 7-22 Performace level of element in ideCAD 5.511

CHAPTER 8

DISCUSSION OF RESULTS AND CONCLUSION

8.1 Summary

A reinforced concrete structure, which was constructed in 1956 as a school building, is assessed by employing different codes or guidelines and different analysis procedures in this study. The building which is located in Gazi Mağusa is a three storey reinforced concrete structure serving as a high school building.

In the previous chapters, both existing and strengthened structures are first processed with non- linear elastic analysis by employing the triangular and uniform mode shapes. Inelastic displacements were calculated as a result of this analysis and finally, before and after strengthening structural systems were assessed by using the nonlinear procedures proposed in the TEC 2007, FEMA 356 and ATC 40. By using the analysis results, the structural systems were assessed according to the TEC 2007 by placing the new shear walls in the existing moment resisting frames and jacketing columns, the existing system was strengthened.

8.2 Discussion of Results

Existing structure was in collapse level. After jacketing columns and adding shear walls, the performance level of existing structure was increased. As shown in Figure 8.1 and 8.2, FEMA 356 performance point was rising. In Chapter 7, although capacity curve increased with jacketing, performance level was in collapse stage because of collapsing of 4 columns.. On the other hand, in shear wall stage GMNKL performance level became immediate occupancy (IO) level.

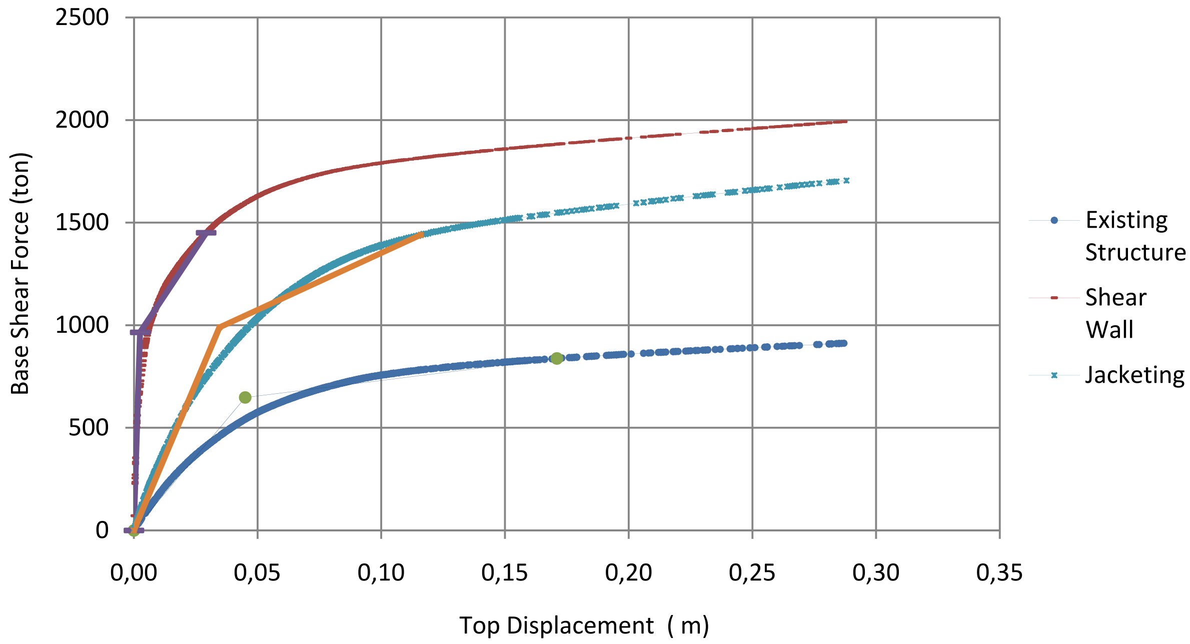


Figure 8-1 Differencing performance points between after and before strengthening methods in X direction

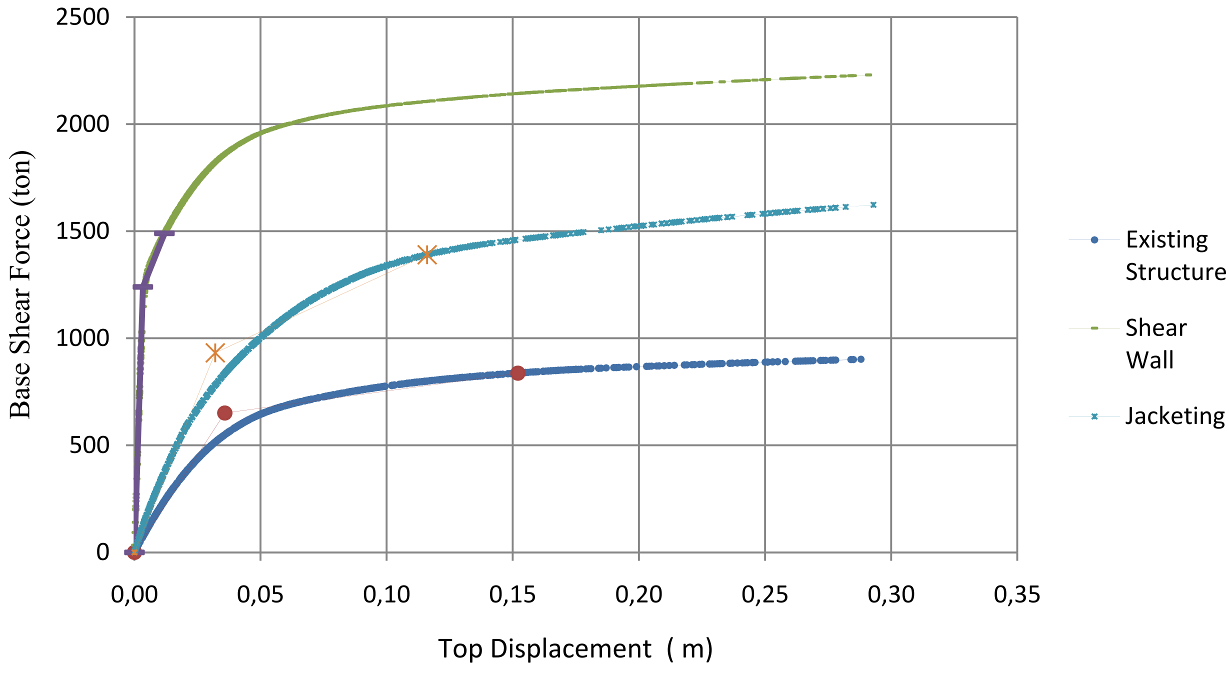


Figure 8-2 Differencing performance points between after and before strengthening methods in Y direction

In linear elastic analysis, in existing structure, jacketing and shear wall stage gave collapse level. In existing structure stage in +Ex direction 18.3%, in –Ex direction 17.92%, in +Ey direction 22.5%, in –Ey direction 18.5% of column were in collapse region. For beams, in +Ex direction 63.9%, in –Ex direction 71.7%, in +Ey direction 70.2%, in –Ey direction 69.03% of beams were in collapse region. After jacketing methods, +Ex direction 1.67%, in –Ex direction 1.67%, in +Ey direction 1.25%, in –Ey direction 4.17% of columns were in collapse region. On the other hand +Ex direction 61.7%, in –Ex direction 71.4%, in +Ey direction 54.6%, in –Ey direction 56.8% of beams were in collapse region. Other strengthening method, which is shear wall, +Ex direction 0.83%, in –Ex direction 0.83%, in +Ey direction 0%, in –Ey direction 0% of columns were in collapse region and all shear walls were in IO level. For beams +Ex direction 8.18%, in –Ex direction 14.49%, in +Ey direction 8.18%, in –Ey direction 8.55% of beams were in collapse region, for this reason shear wall was in collapse level in linear elastic analysis.

Table 8.1 shows comparison of cost between jacketing and shear wall strengthening methods. Unit prices have been taken from TRNC Ministry of Public Work and Transport, Director of Planning and Project, 2007. In spite of higher cost of shear wall, performance level was better and application time will be shorter than jacketing. For these reasons, shear wall strengthening technique is preferred instead of the jacketing method.

Table 8-1 Cost price of column jacketing and shear wall

	Concrete (m^3)	Steel (ton)	Unit price* (TL)	Total cost(TL)
Column Jacketing	260.37	34,484.00	425.00	110,631.75
Shear Wall	271.47	24,370.00	425.00	115,375.75

*Unit price= Concrete + Steel + Formwork

8.3 Conclusions

Seismic assessment of existing buildings was added to the 2007 Turkish Earthquake Code as a new chapter. Two different alternative procedures are introduced, namely linear elastic and nonlinear assessment procedures. In this study performance of both procedures are examined. Based on the research performed in this study, following conclusions were drawn:

— Both analyses, nonlinear and linear static analysis performed according to FEMA 356, ATC 40 and 2007 Turkish Earthquake Code. In nonlinear static analysis, before strengthening of existing building performance level was obtained to be but, in collapse level, after strengthening to this level became IO. In linear analysis both jacketing and shear walls were in collapse level, because of limitation of r (demand/capacity) value is very strict for beams in 2007 Turkish Earthquake Code.

— TEC 2007 gives equal importance to both procedures but the performance evaluation by using both procedures for a building can be different. This may cause conflict between owners and designers because both procedures are legally valid.

— By comparing the actual damage observed with the damage predicted with these procedures, both procedures found to overestimate the actual observed damage. Acceptance limits of both methodologies can be reassessed.

— A great number of column reinforcing bars are found to be corroding. Their formation results in delamination of concrete. This is one of the results effect performance level of structure. For stopping corrosion, epoxy coating of steel is a specialized technique which can be helpful in addition to an adequate thickness of cover concrete of low permeability.

— Although average compressive strength of concrete of existing structure was obtained to be 34 MPa, as a result of many experiments concrete was obtained to be brittle and permeability level for some of core samples were moderate and some very high.

8.4 Recommendations for Further Studies

It is expect that this study will be a point of reference to other school buildings in TRNC. As far as other assessed buildings, which were Gazimağusa Türk Maarif Koleji, Çanakkale Ortaokulu and Canbolat Ortaokulu, they had same proplems base on construction years and properties of material.

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APPENDICES

APPENDIX A

Figures plotted to Linear Performance Analysis

In Chapter 7 all Linear Performance Analysis results are given. In this section Linear Performance Analysis ***r*** (demand/capacity)-member joints graphics are given in detailly. Below tables base on earthquake direction and members of floor. In existing building graphics limitations were taking collapse (for beams r=10, for column r= 8). In strengthening section, which are jacketing and shear wall graphics limitation were taking life safety (for beam r=7, for column r=6)

A.1 Existing building

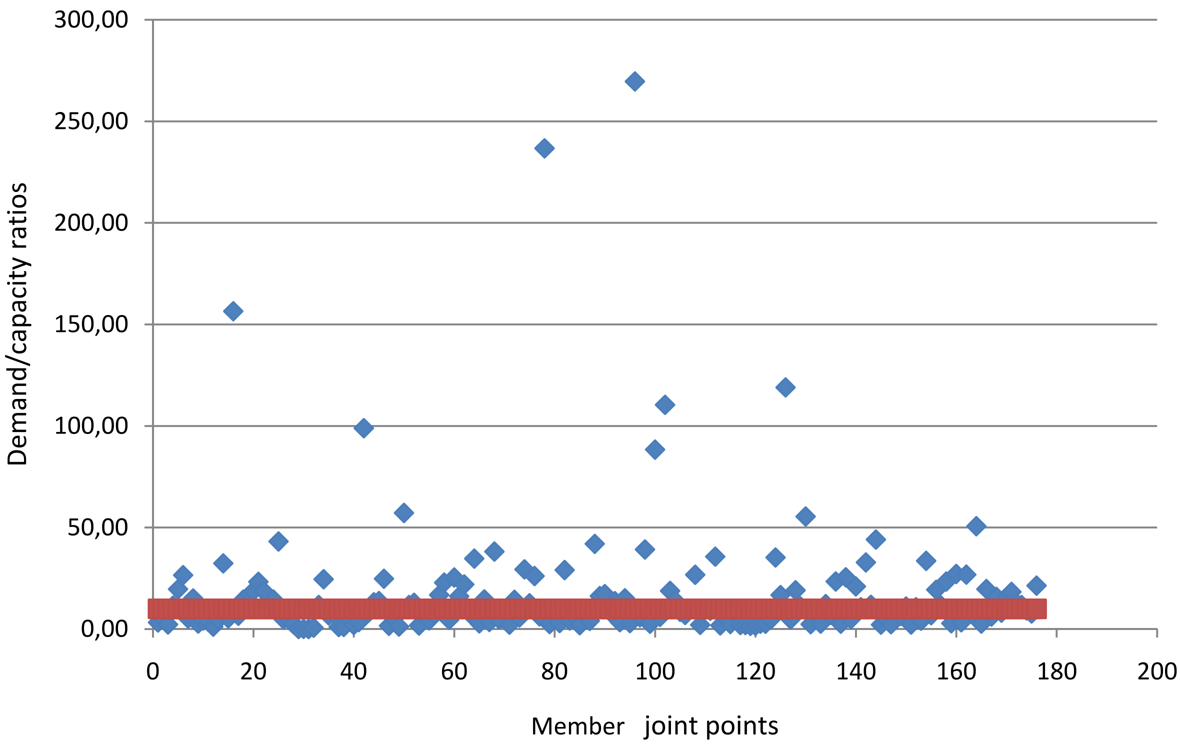

Figure A-1 +EX 2nd Floor Beam

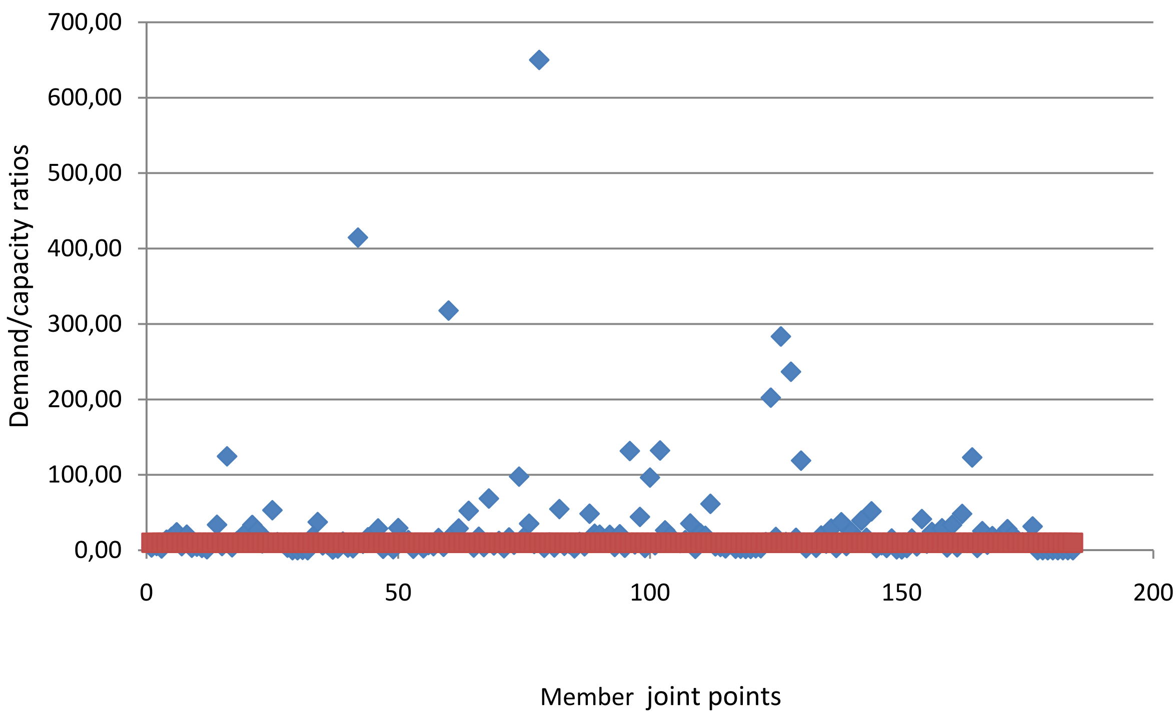

Figure A-2 +EX 1nd Floor Beam

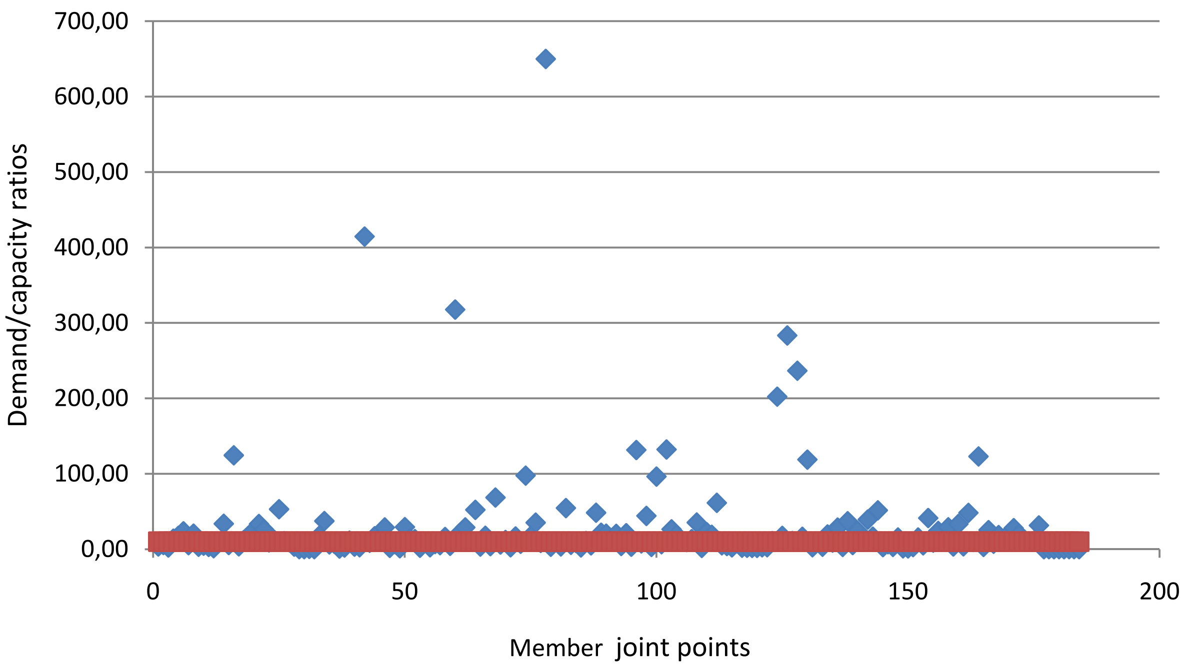

Figure A-3 +EX Ground Floor Beam

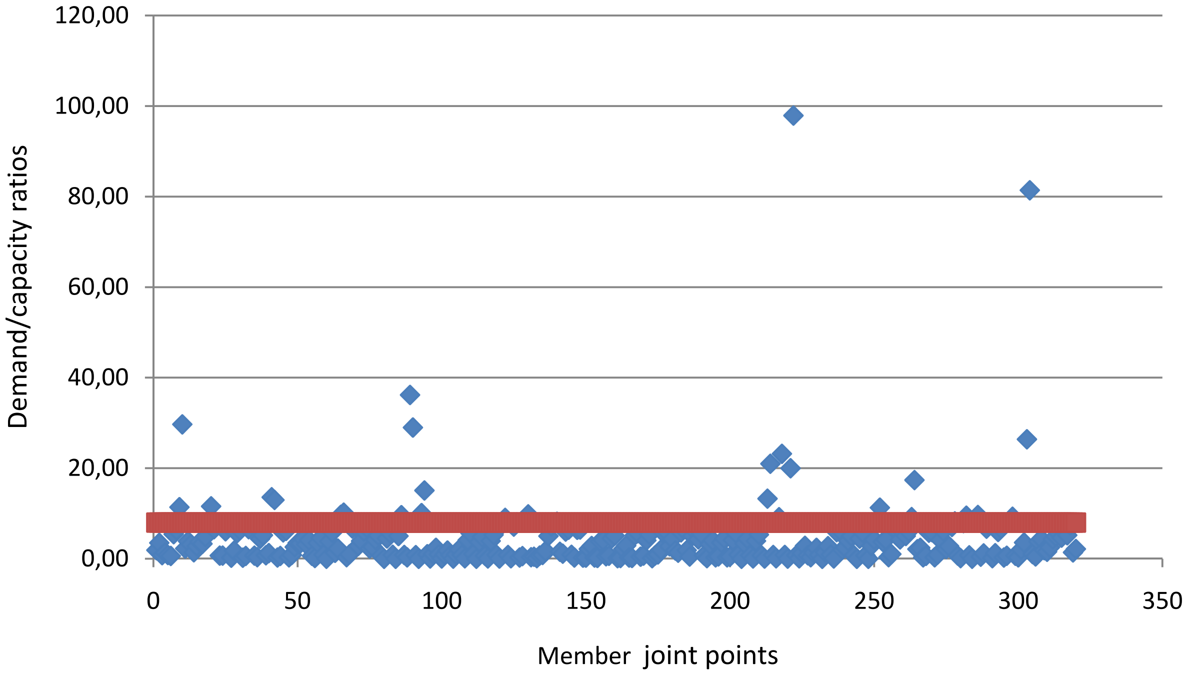

Figure A-5 +EX 2nd Floor Column

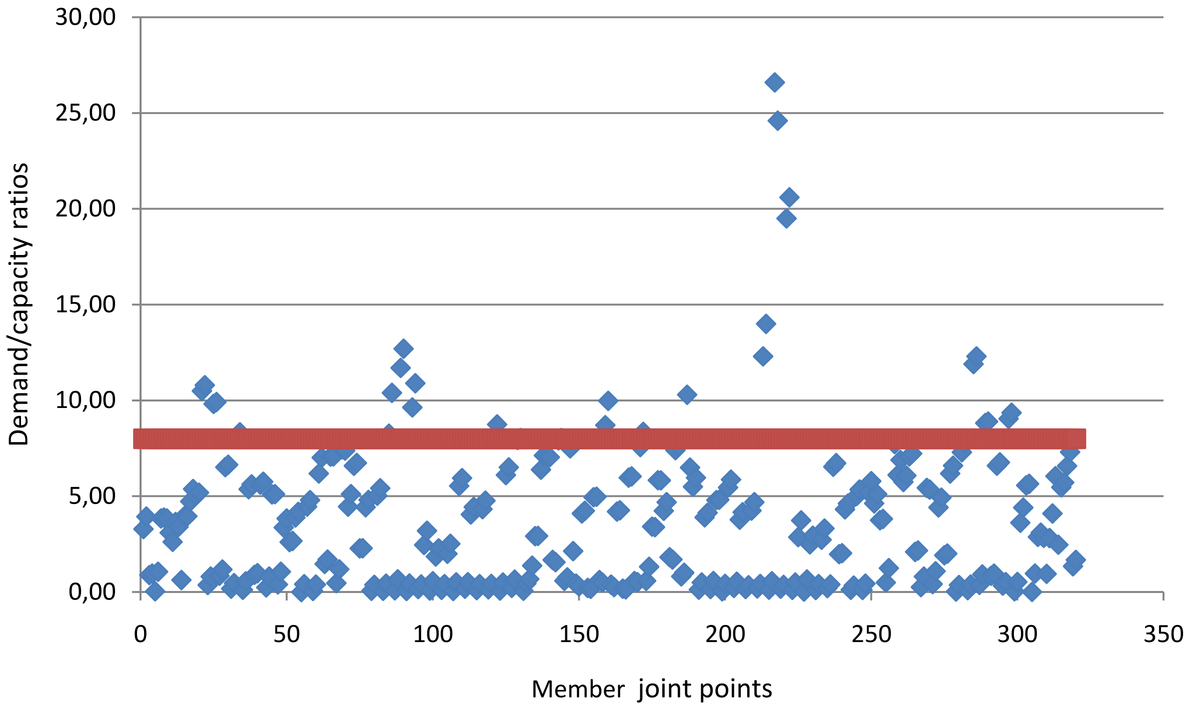

Figure A-6 +EX 1st Floor Column

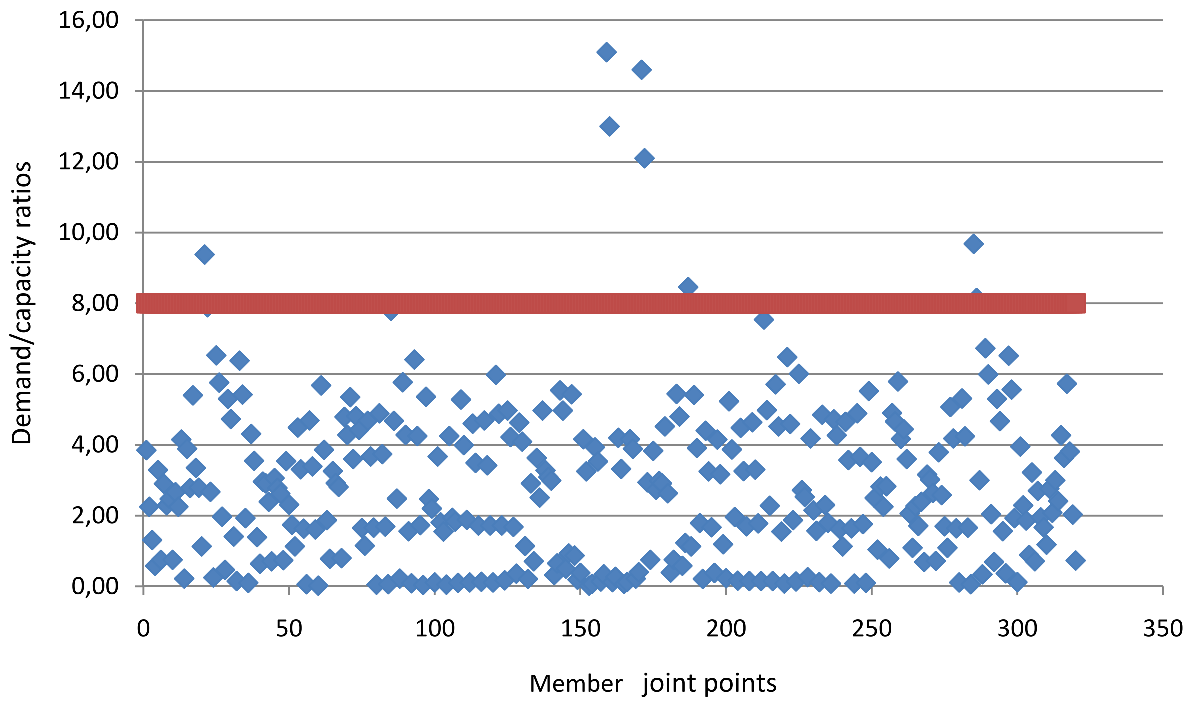

Figure A-7 +EX Ground Floor Column

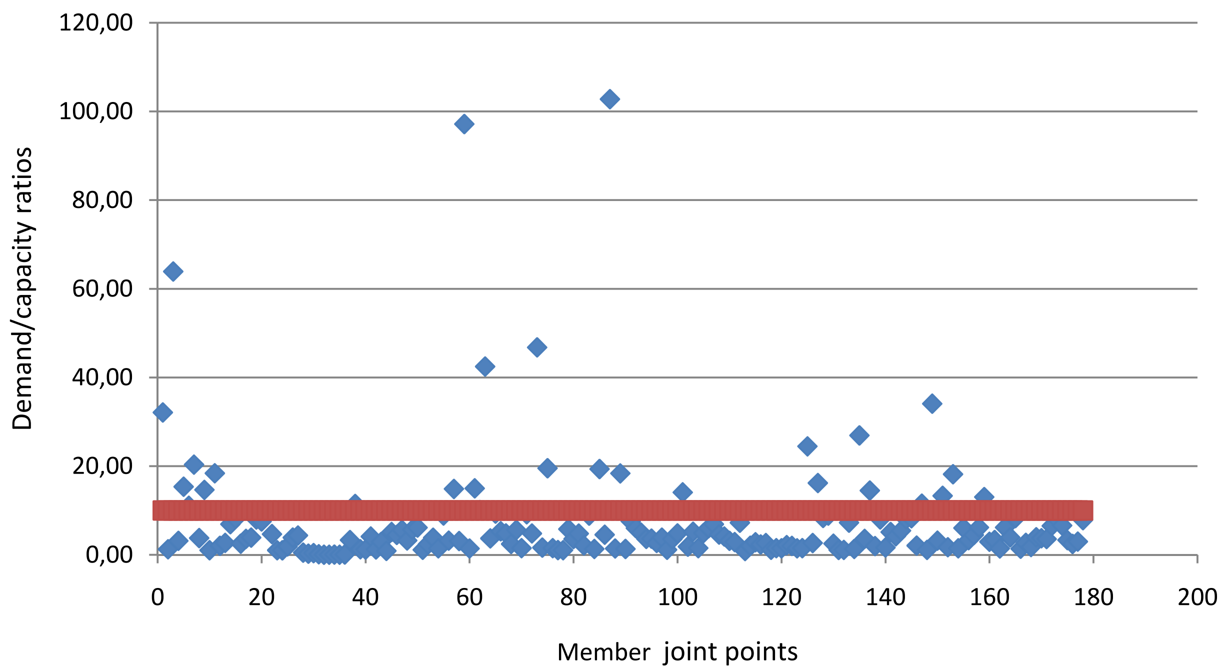

Figure A-8 -EX 2nd Floor Beam

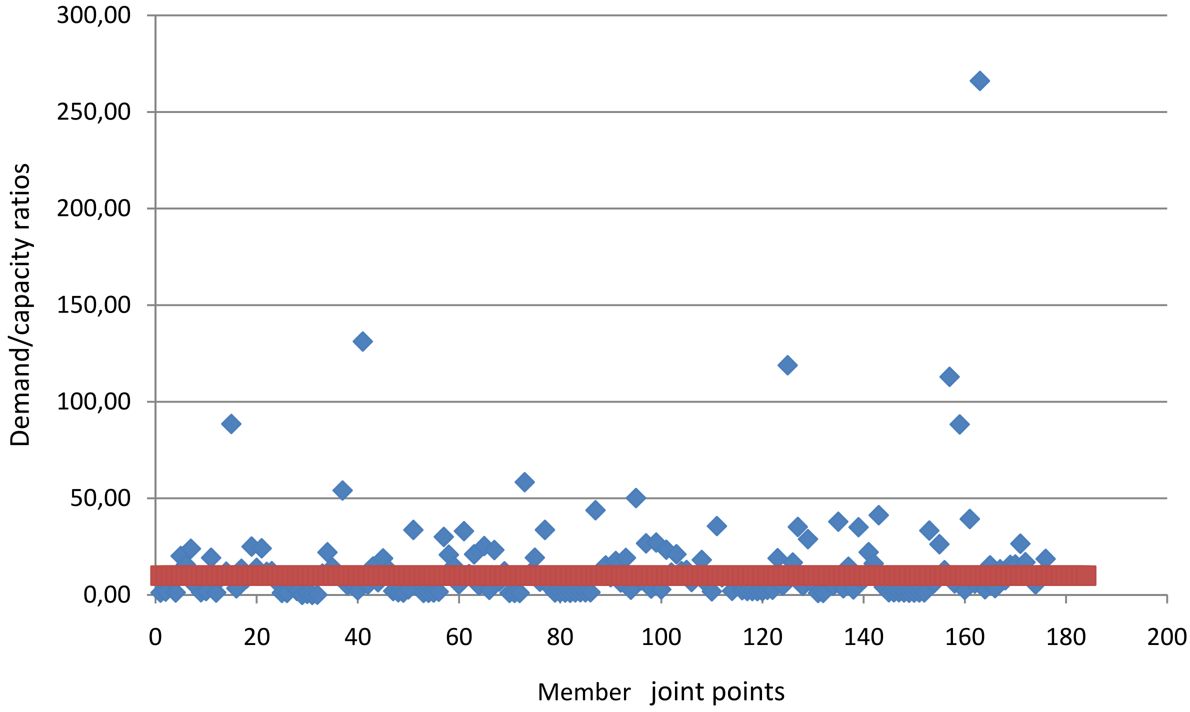

Figure A-9 -EX 1st Floor Beam

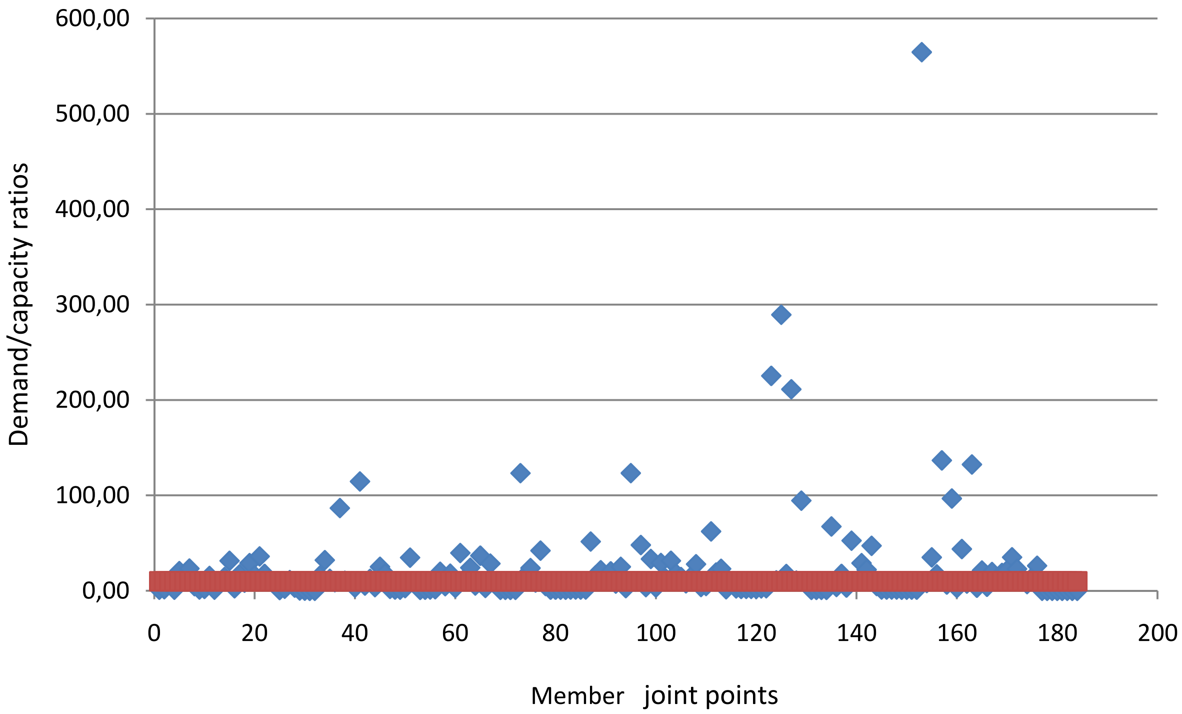

Figure A-10 -EX Ground Floor Beam

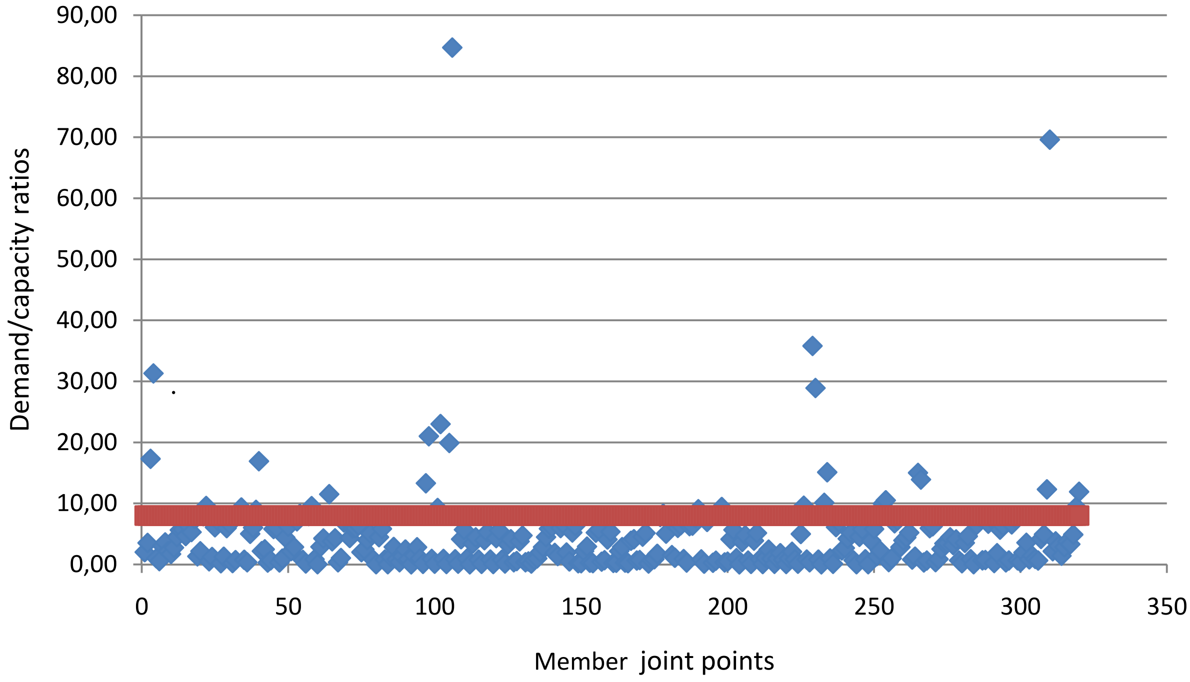

Figure A-11 -EX 2nd Floor Column

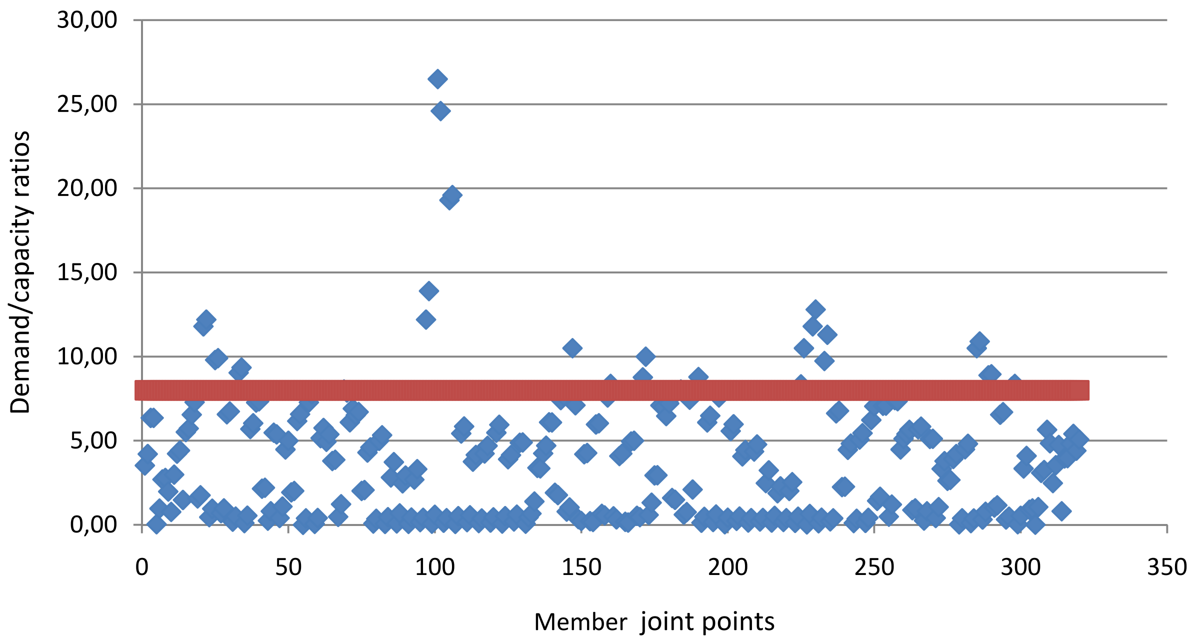

Figure A-12-EX 1st Floor Column

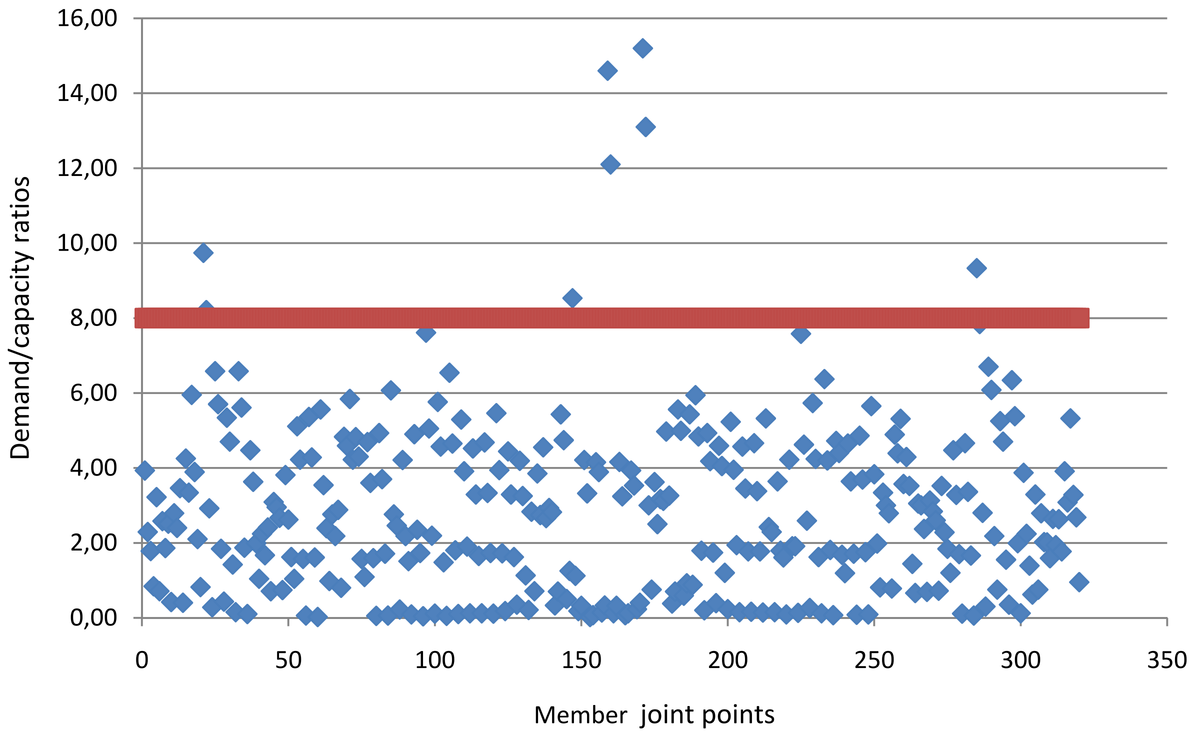

Figure A-13 -EX Ground Floor Column

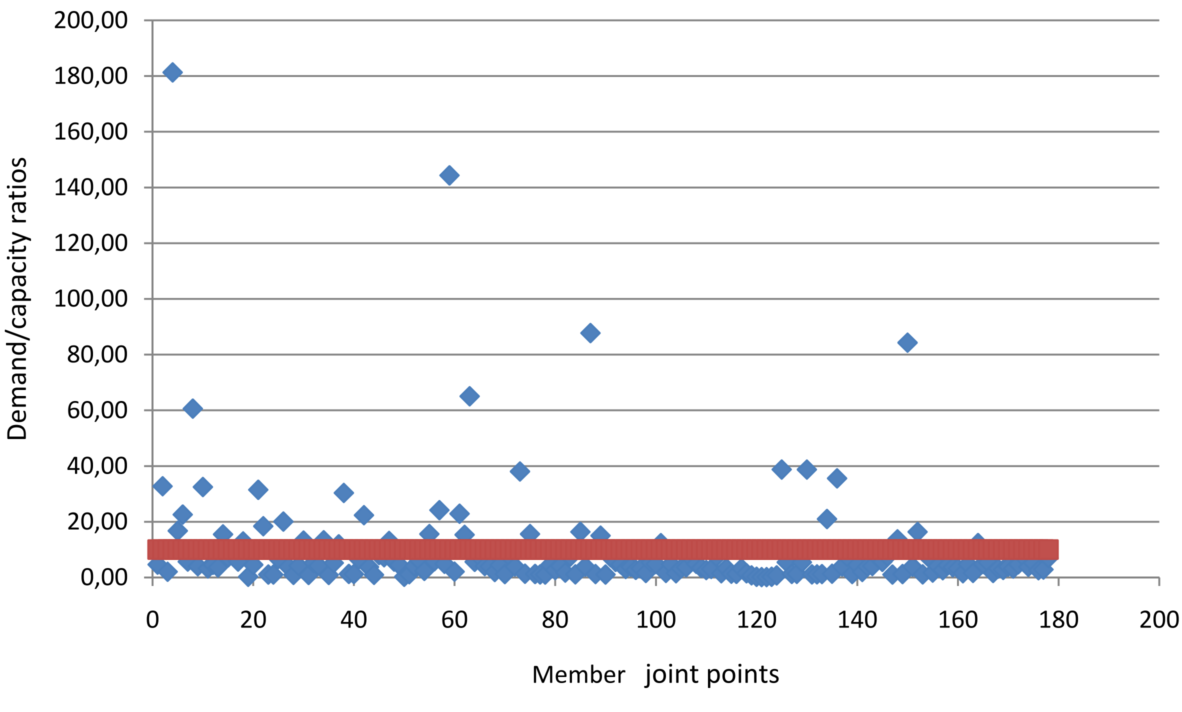

Figure A-14 +EY 2nd Floor Beam

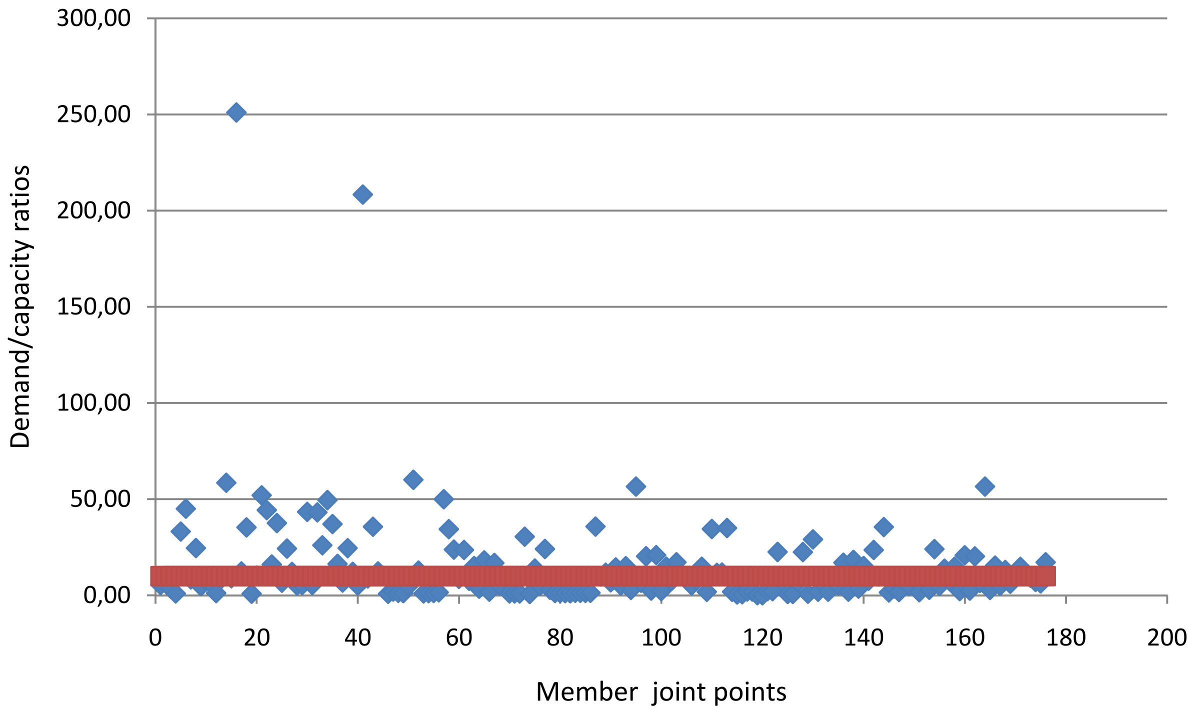

Figure A-15 +EY 1st Floor Beam

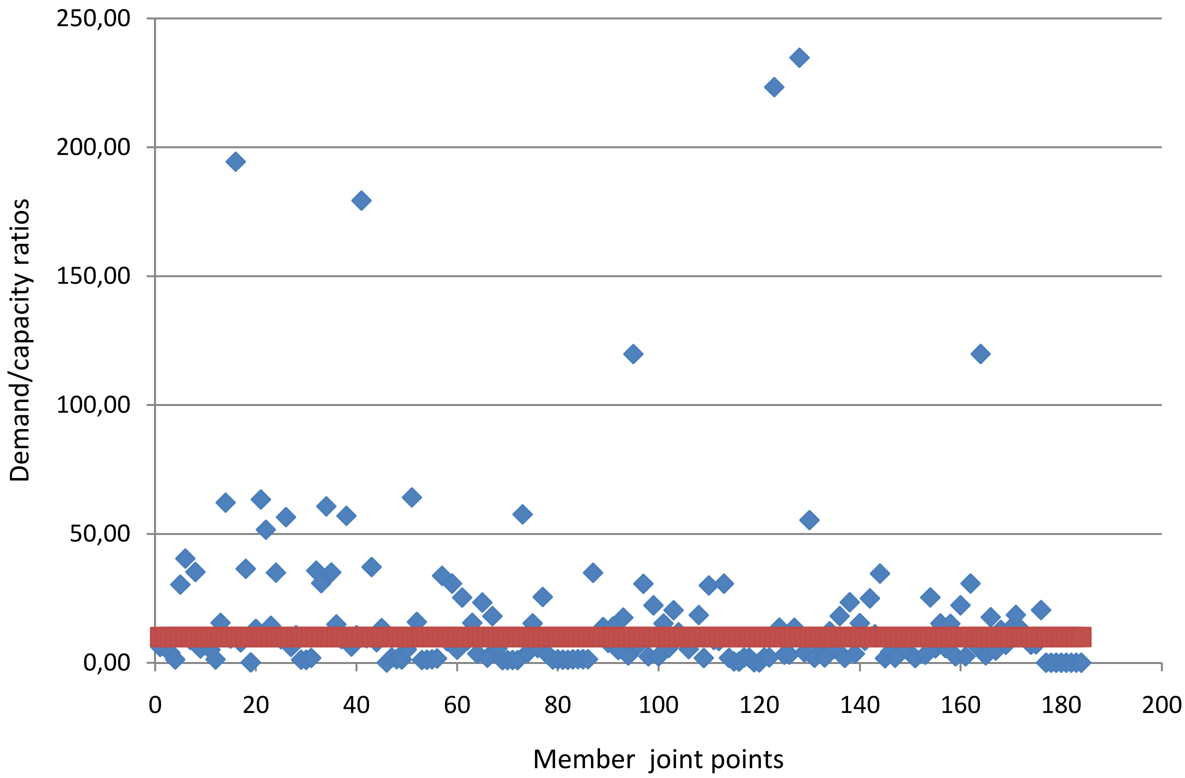

Figure A-16 +EY Ground Floor Beam

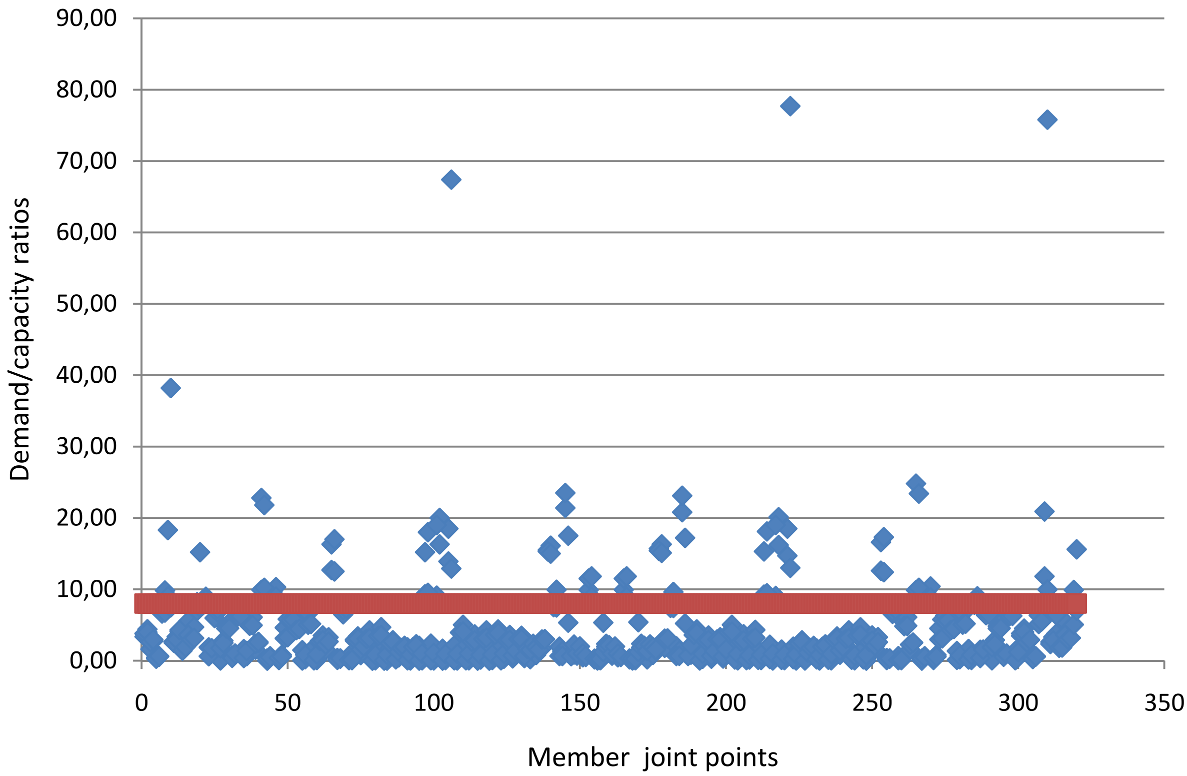

Figure A-17 +EY 2nd Floor Column

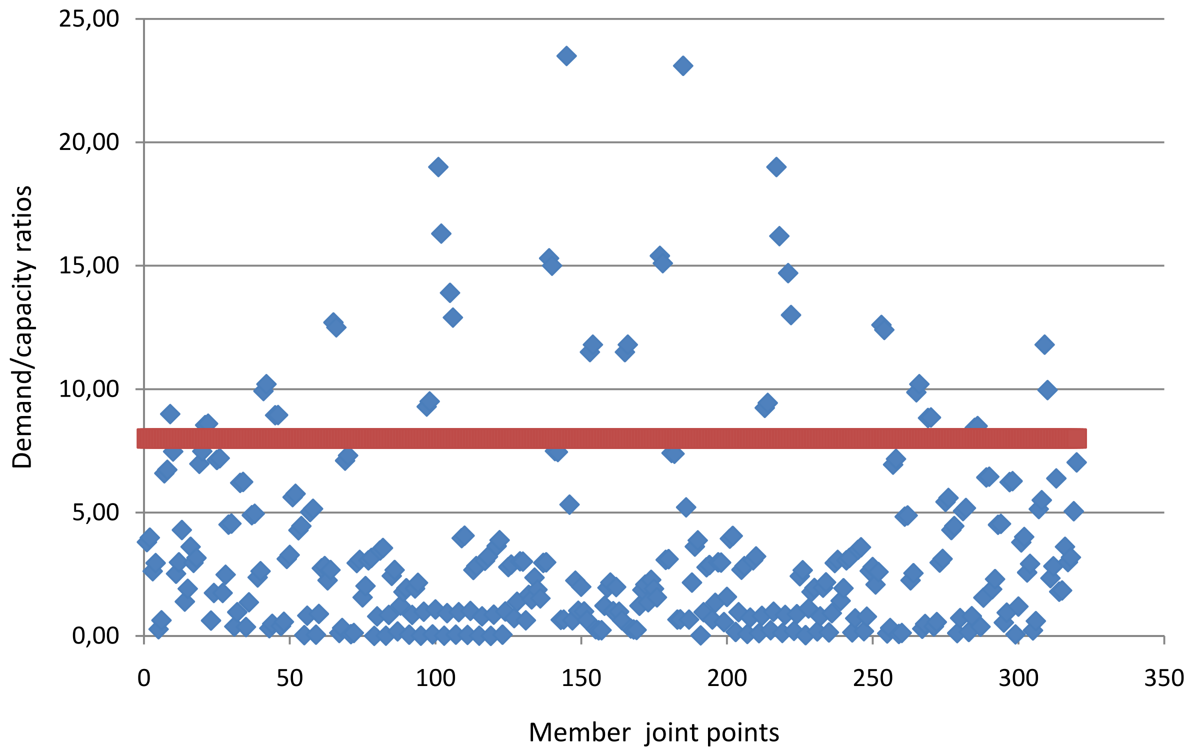

Figure A-18 +EY 1st Floor Column

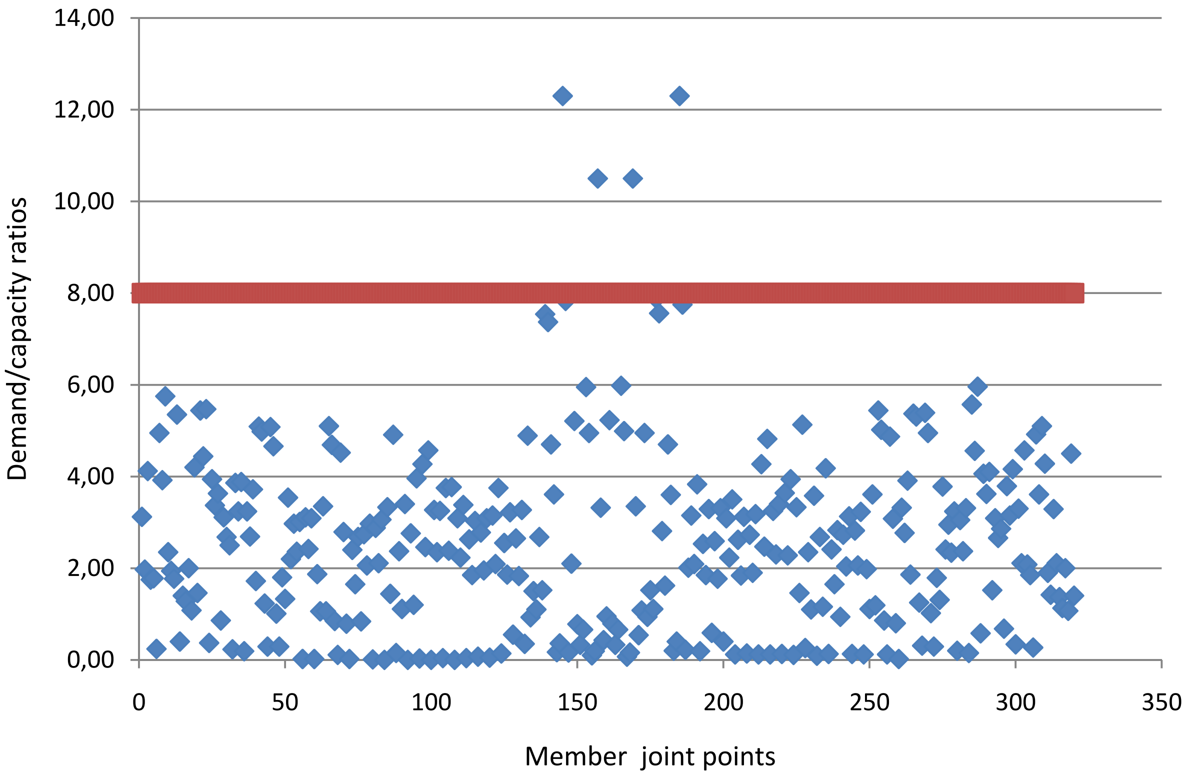

Figure A-19 +EY Ground Floor Column

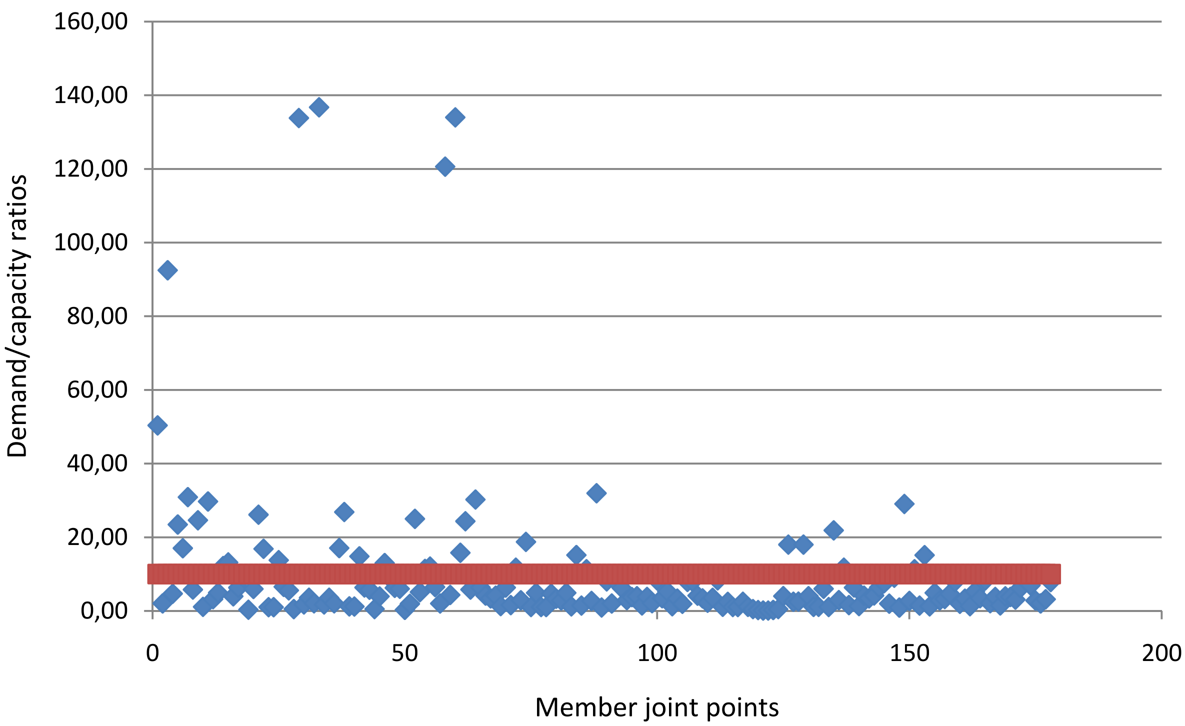

Figure A-20 -EY 2nd Floor Beam

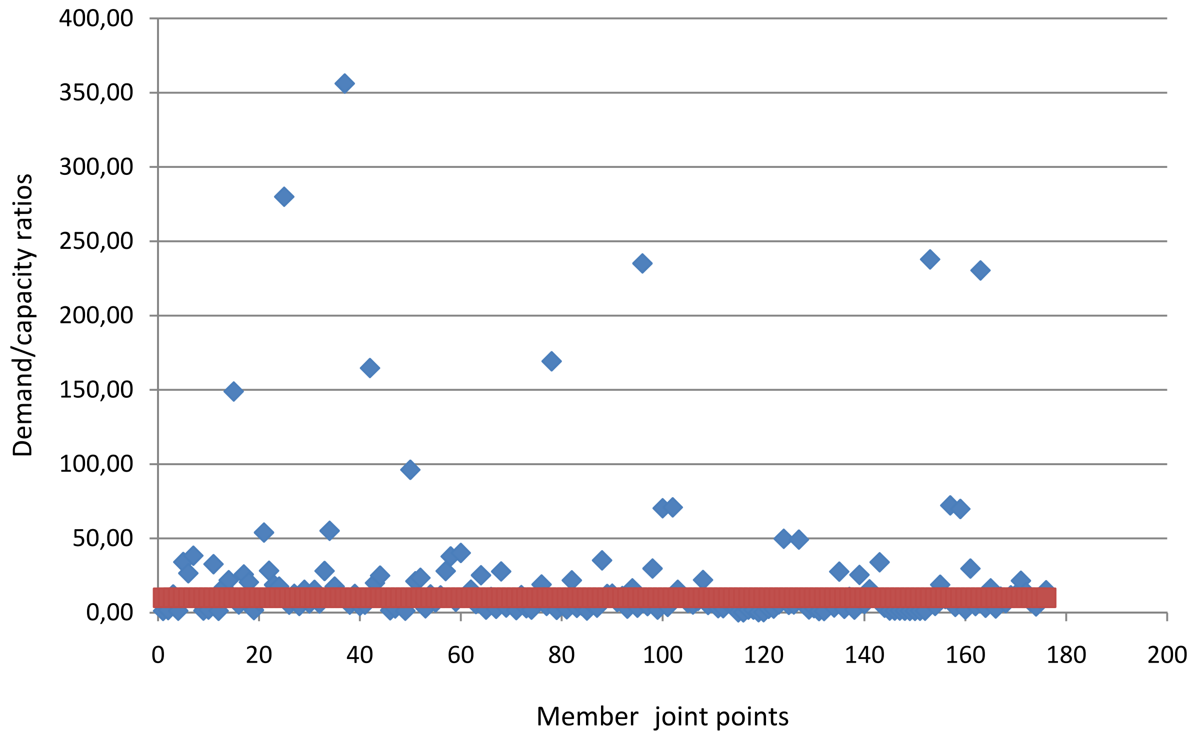

Figure A-21 -EY 1st Floor Beam

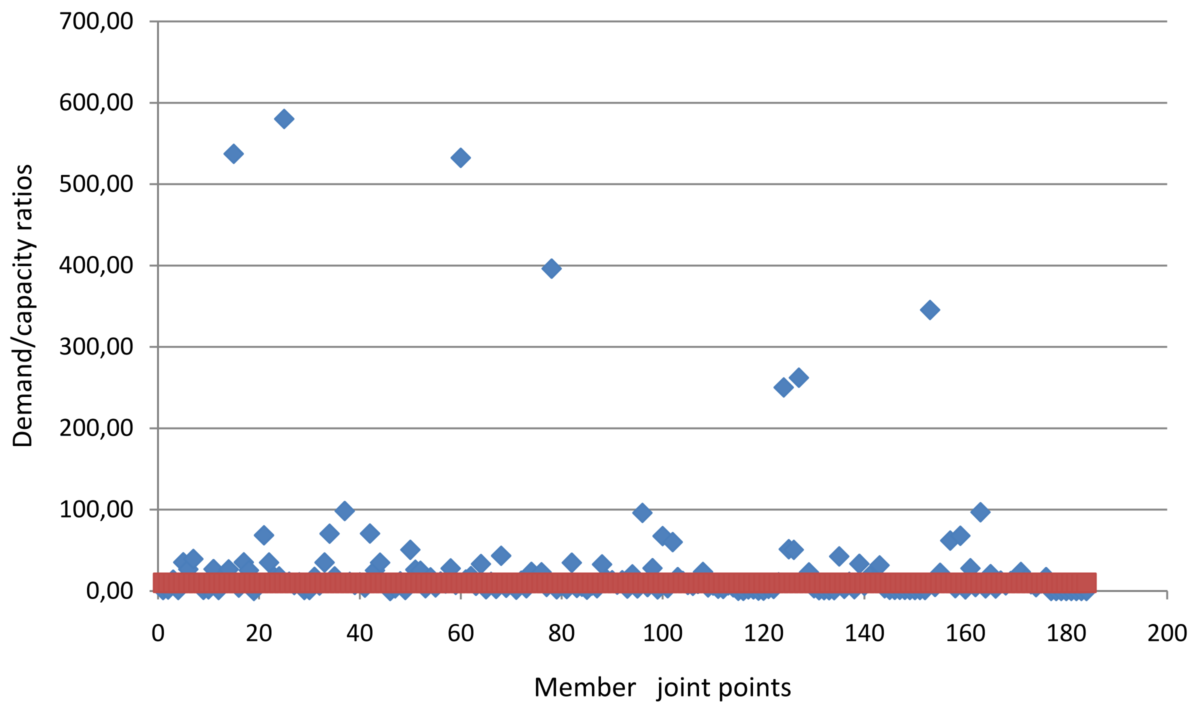

Figure A-22 -EY Ground Floor Beam

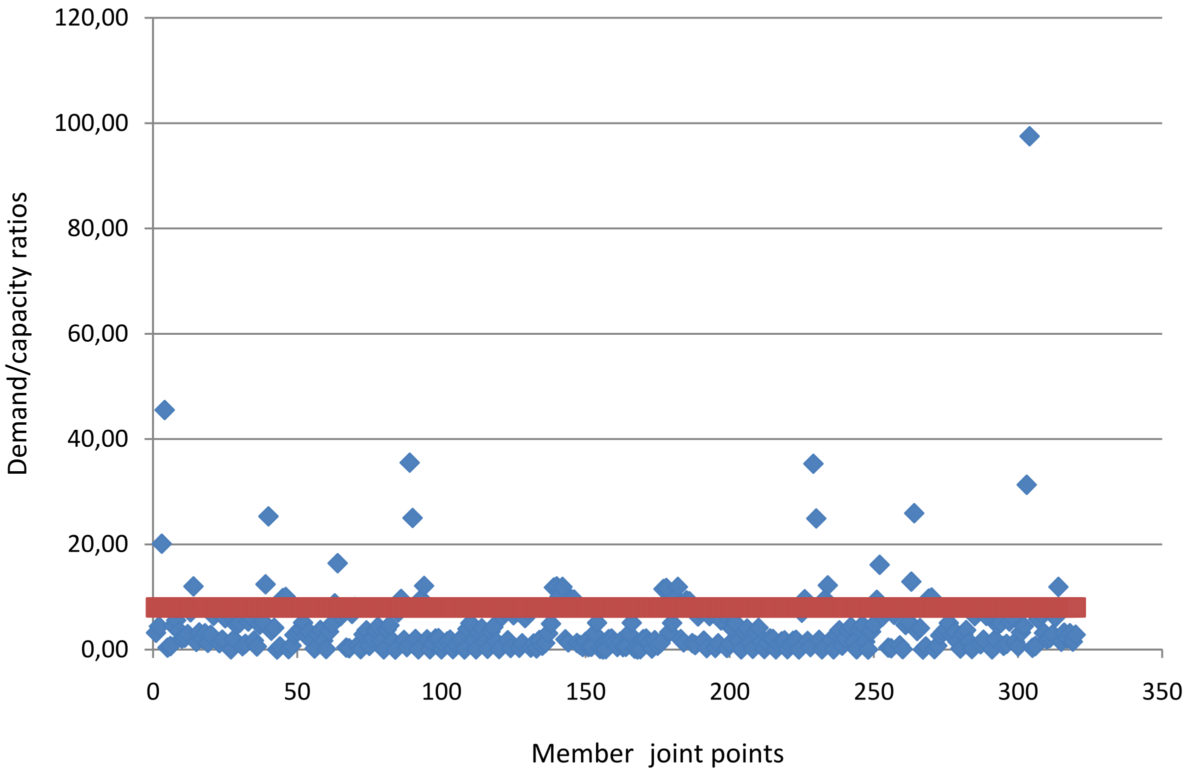

Figure A-23 -EY 2nd Floor Column

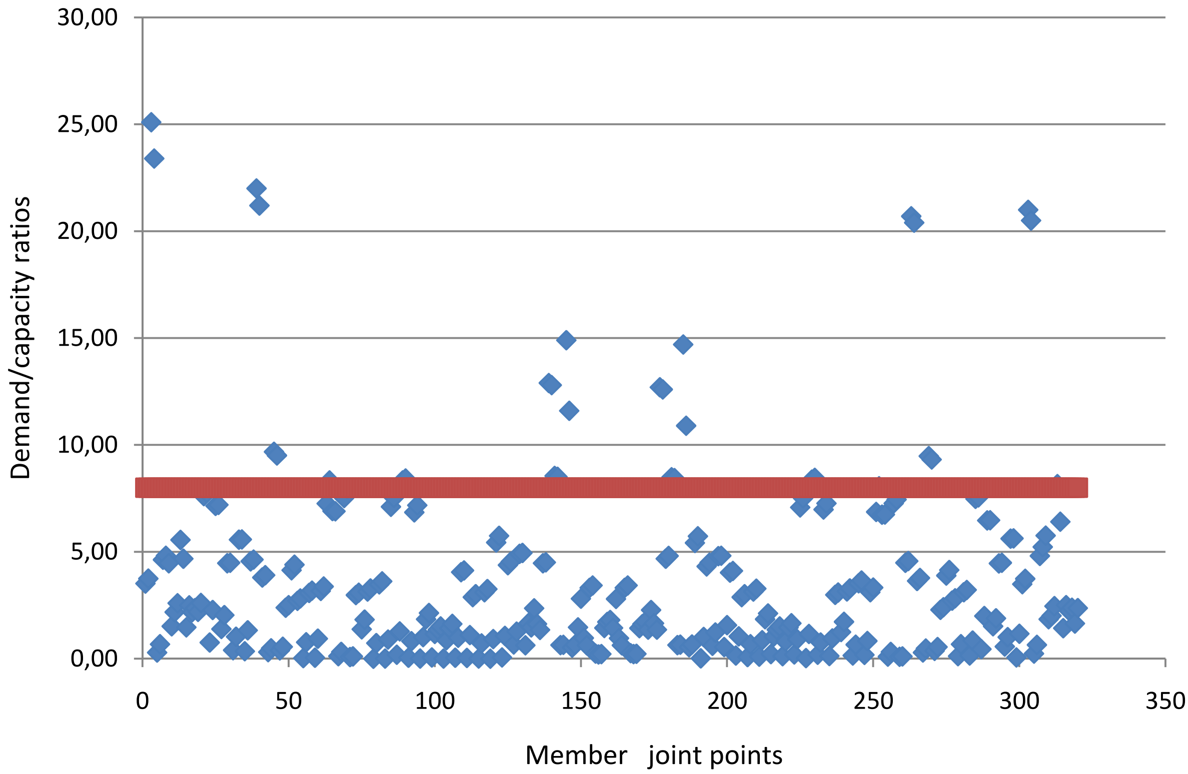

Figure A-24 -EY1st Floor Column

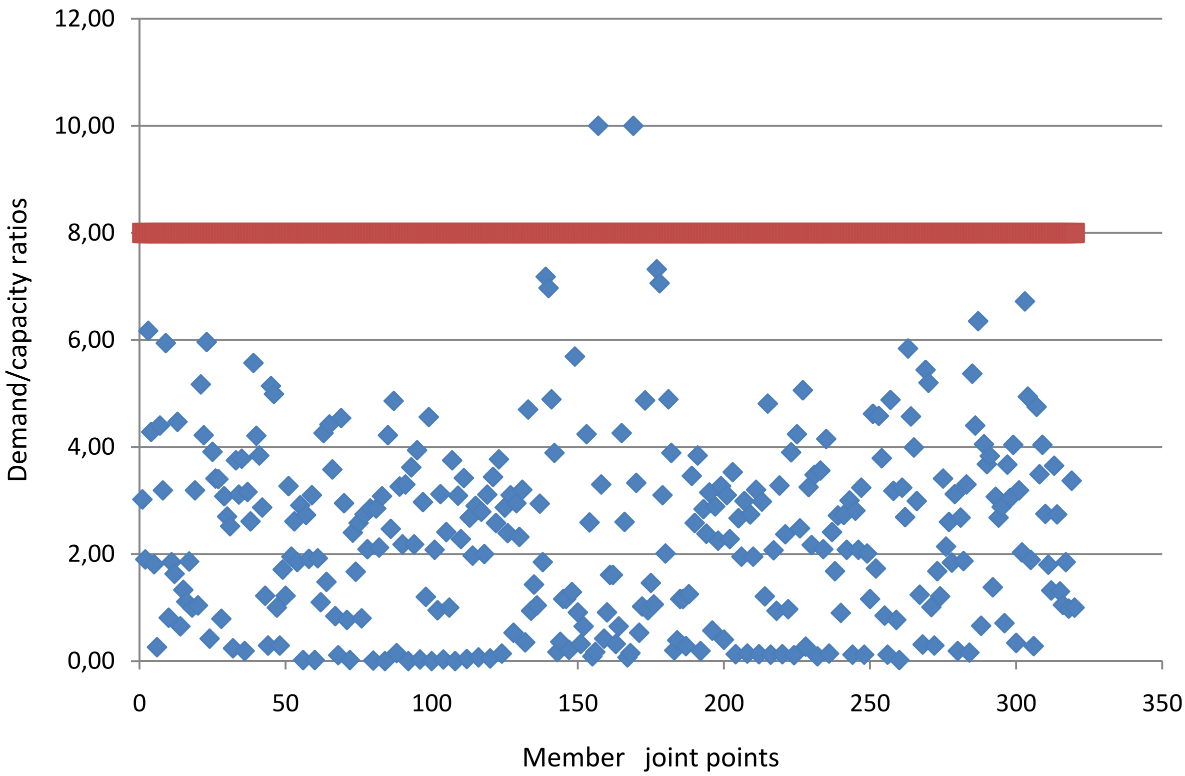

Figure A-25 -EY Ground Floor Column

A.2 Column Jacketing

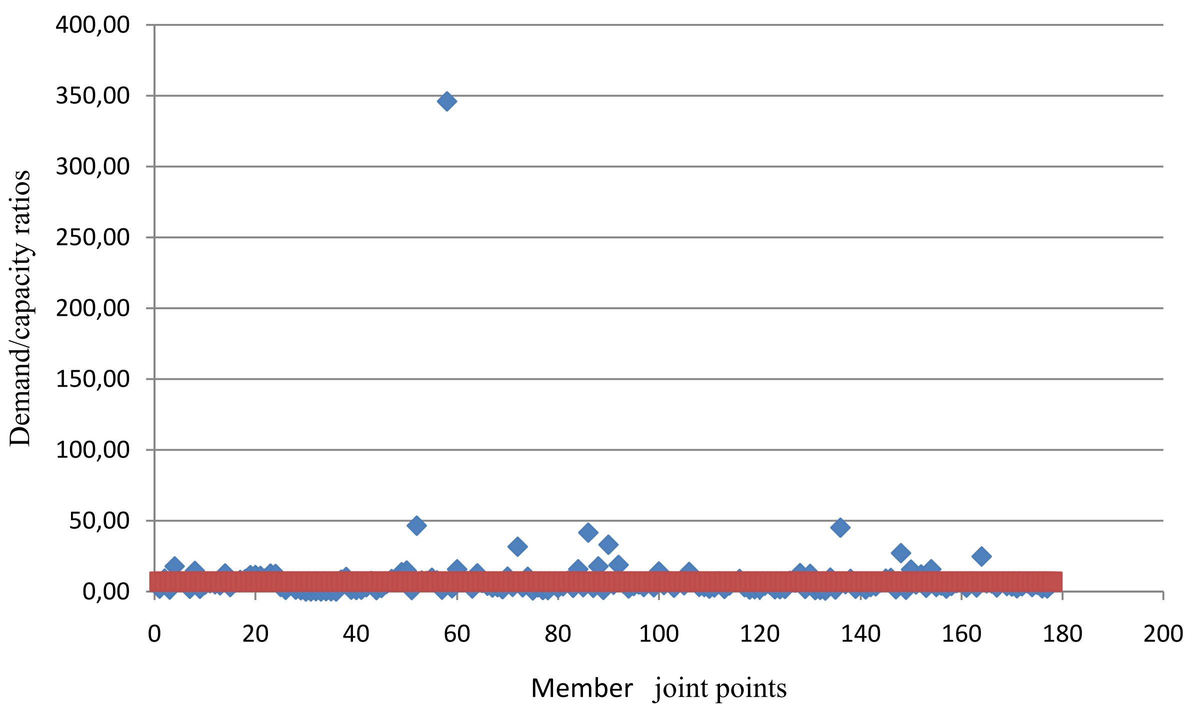

Figure A-26 +EX 2nd Floor Beam

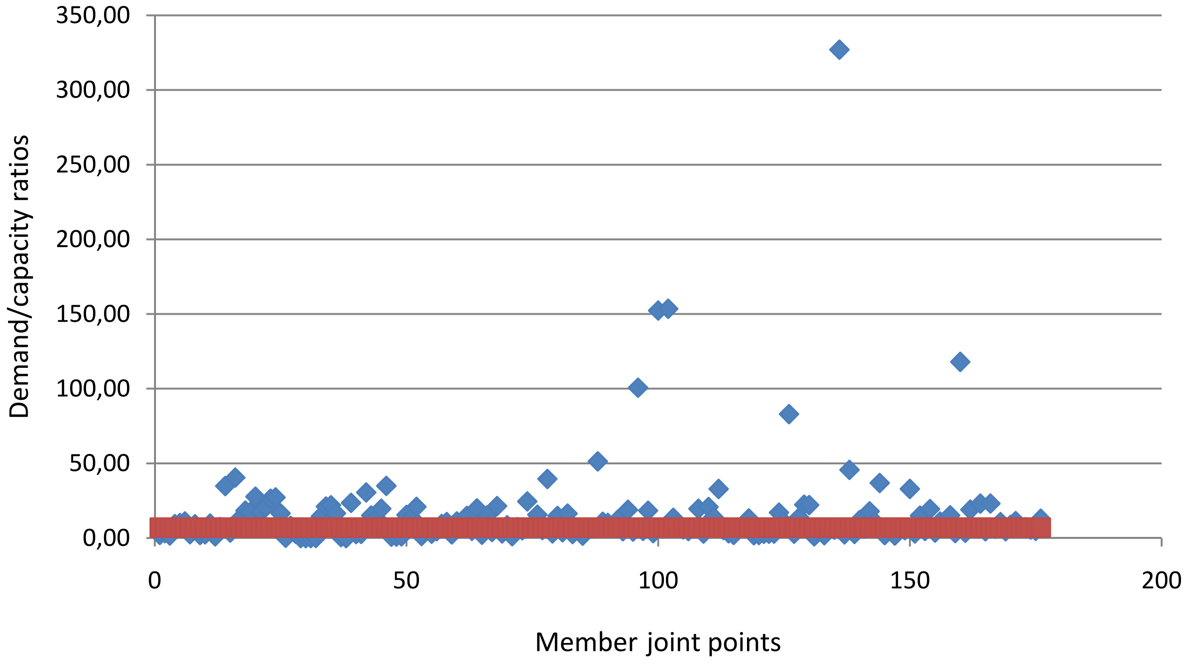

Figure A-27 +EX 1st Floor Beam

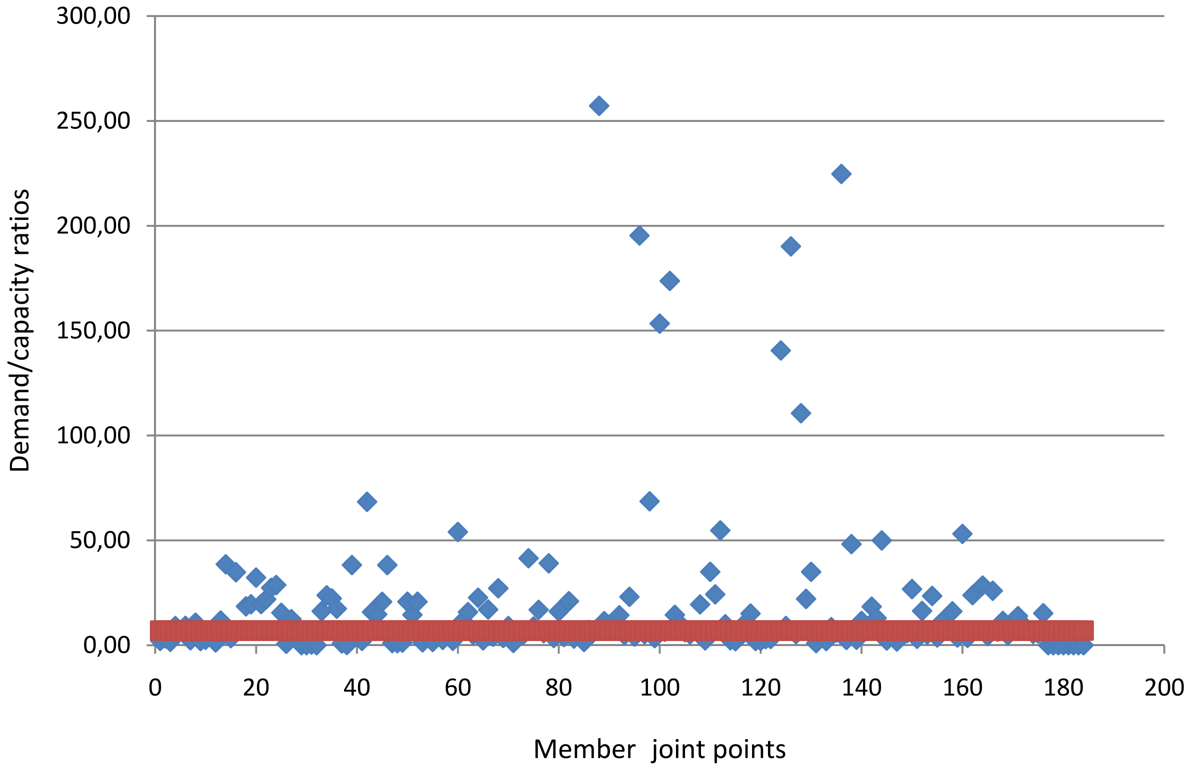

Figure A-28 +EX Ground Floor Beam

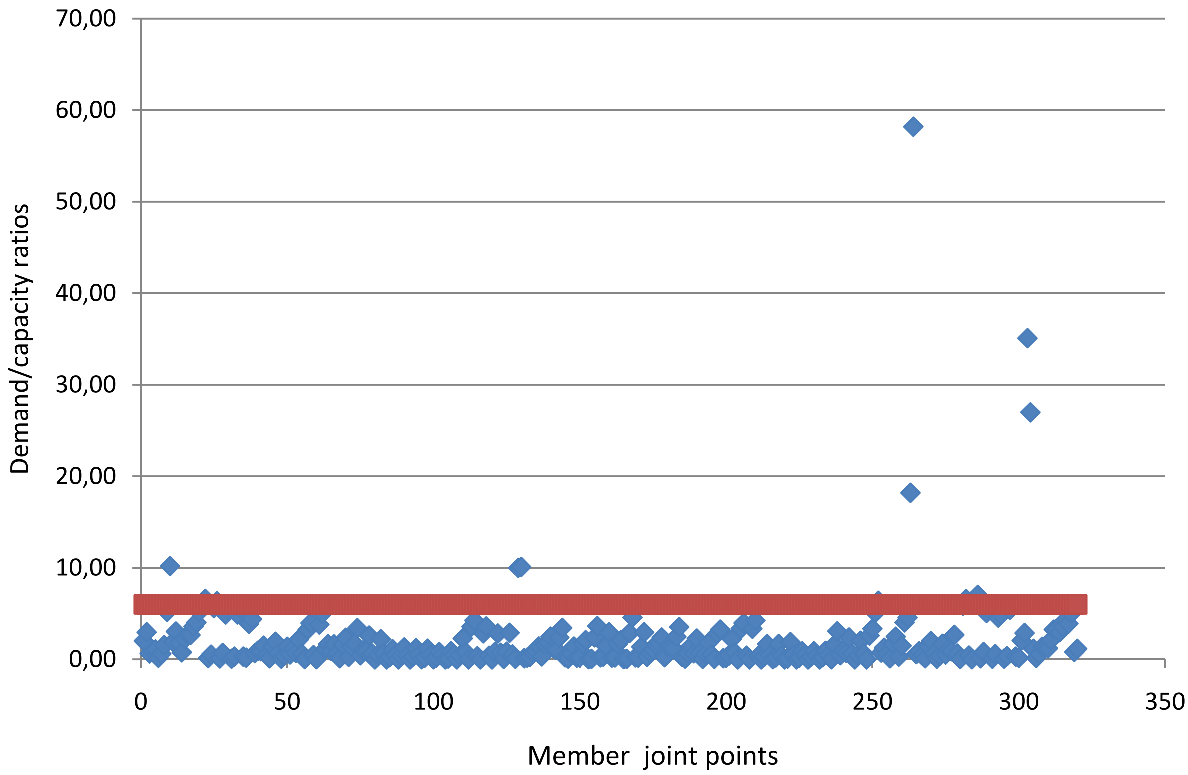

Figure A-29 +EX 2nd Floor Column

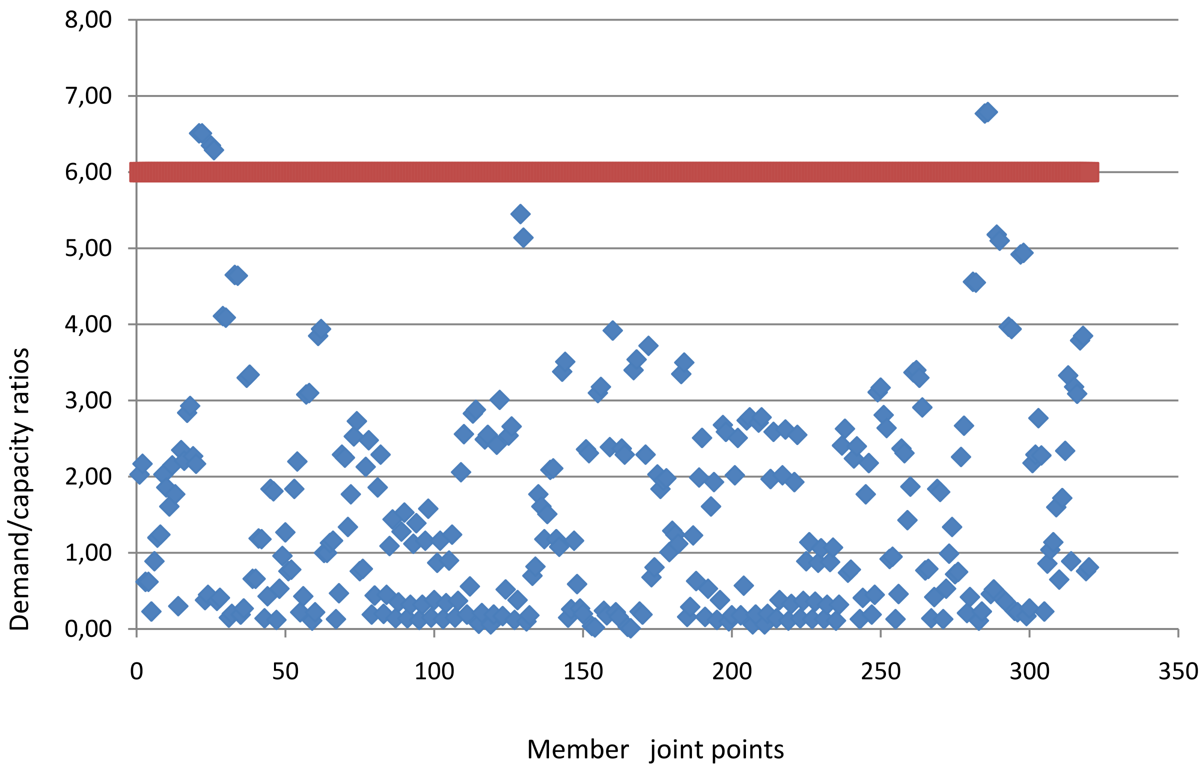

Figure A-30 +EX 1st Floor Column

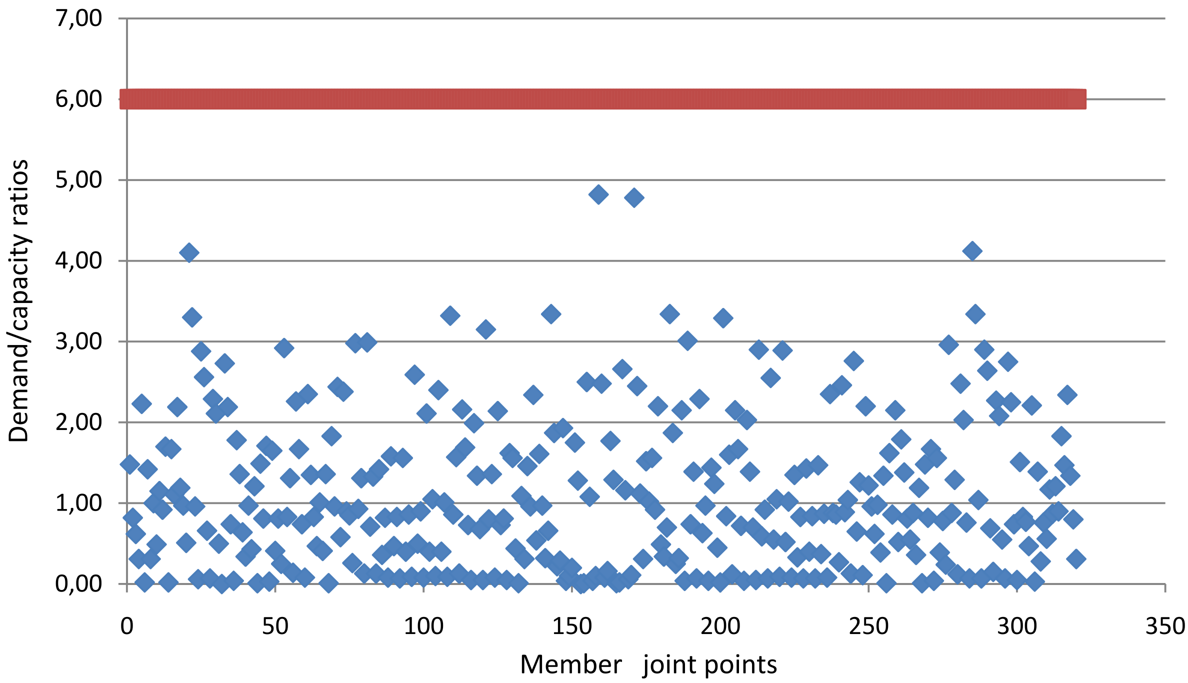

Figure A-31 +EX Ground Floor Column

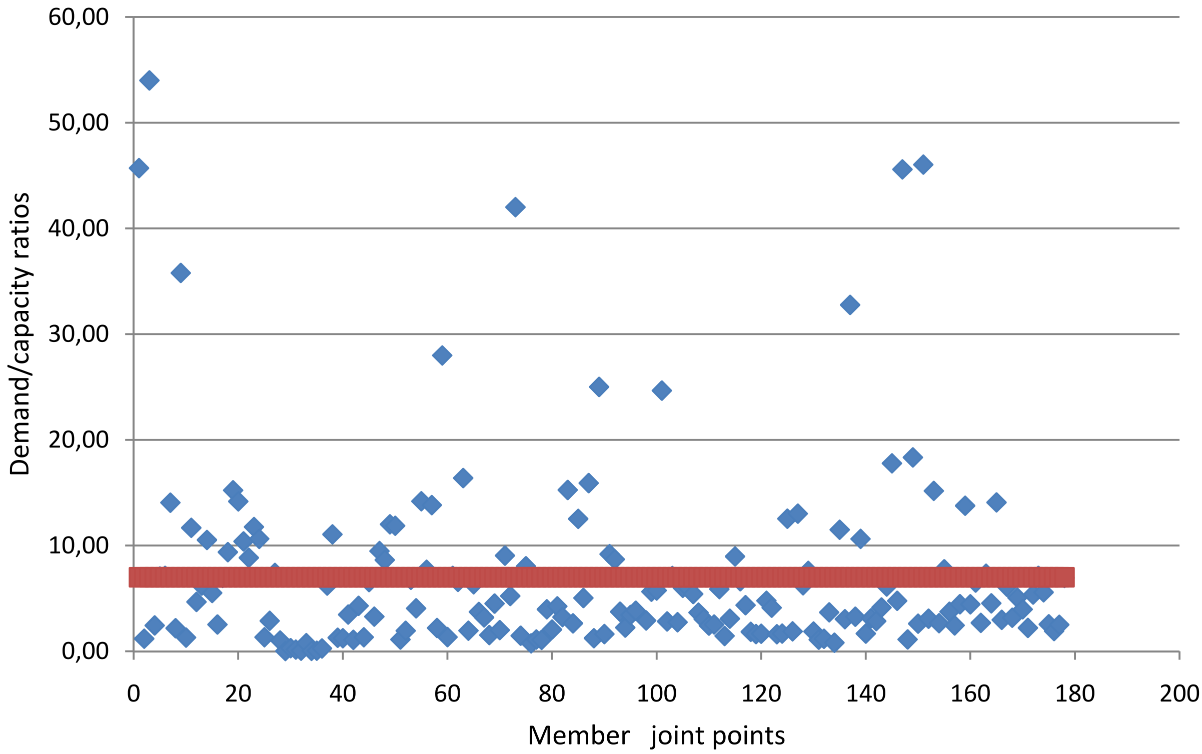

Figure A-32 -EY 2nd Floor Beam

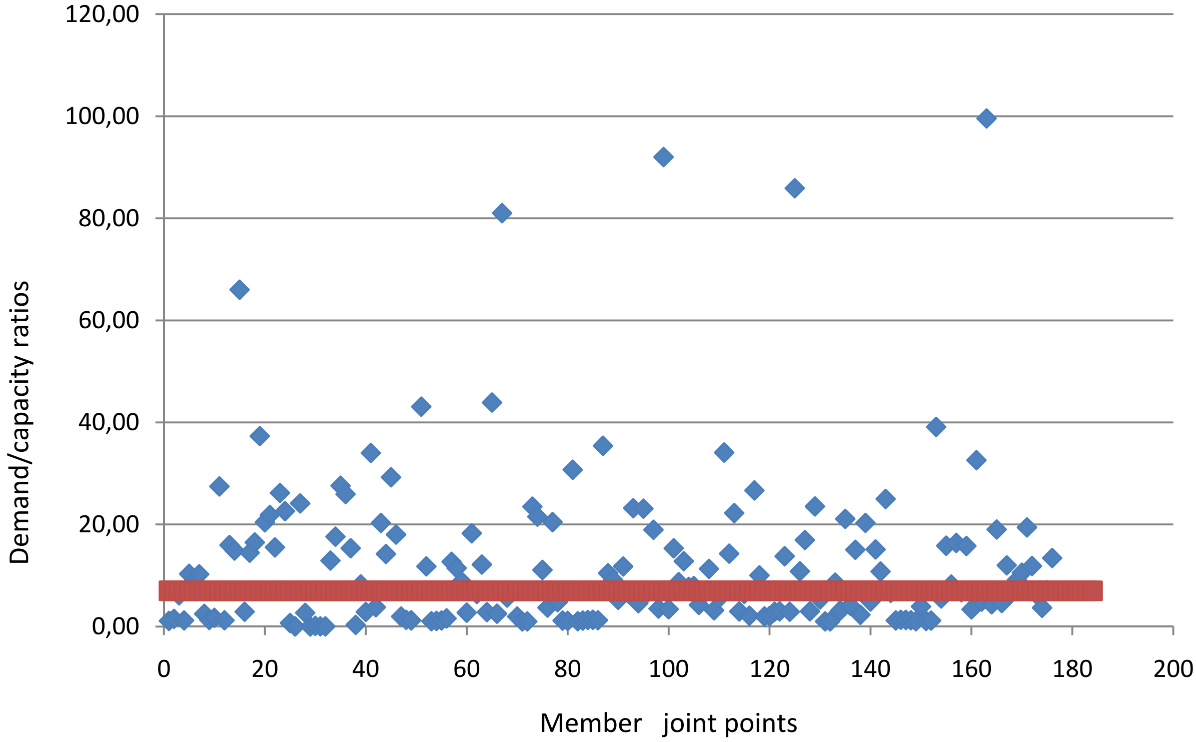

Figure A-33 -EY 1st Floor Beam

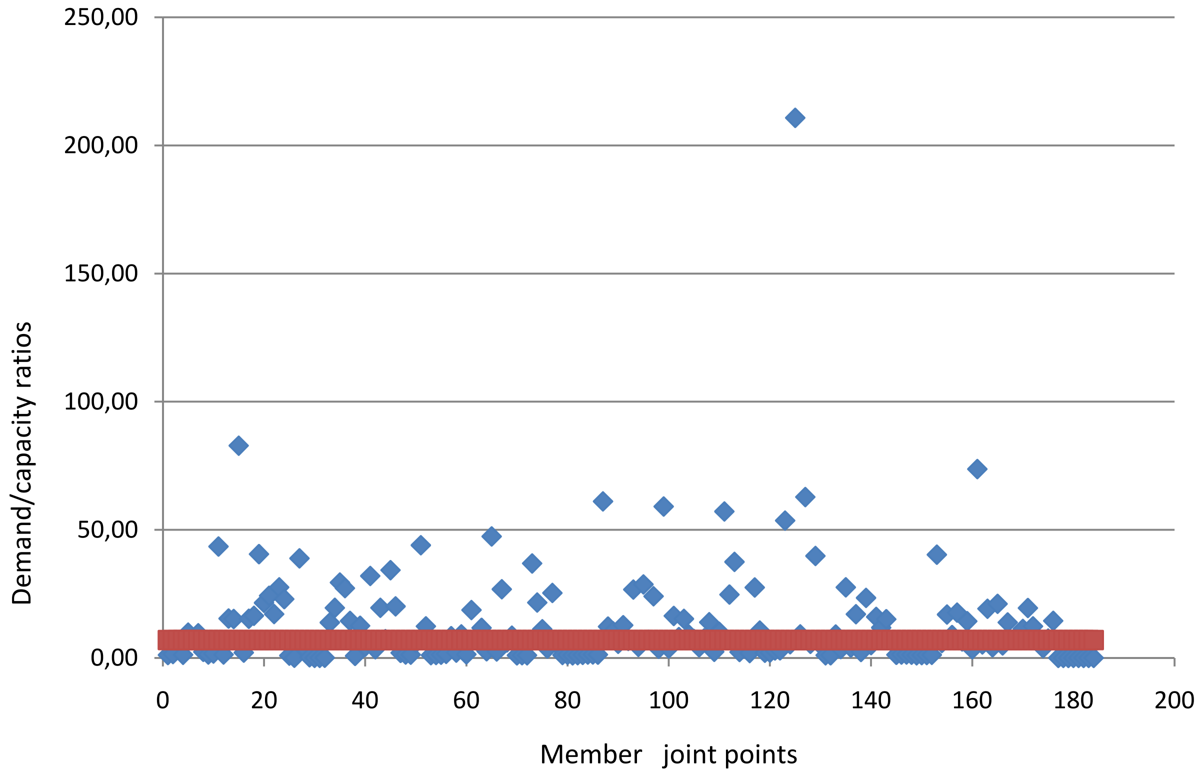

Figure A-34 –EY Ground Floor Beam

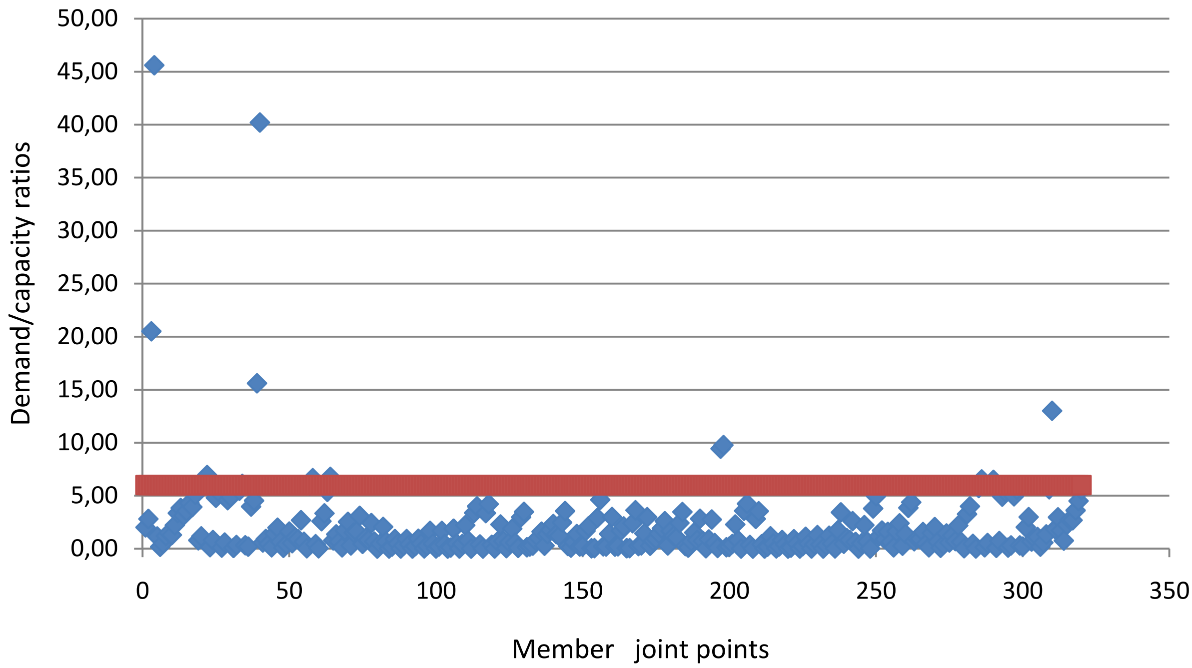

Figure A-35 –EY 2nd Floor Column

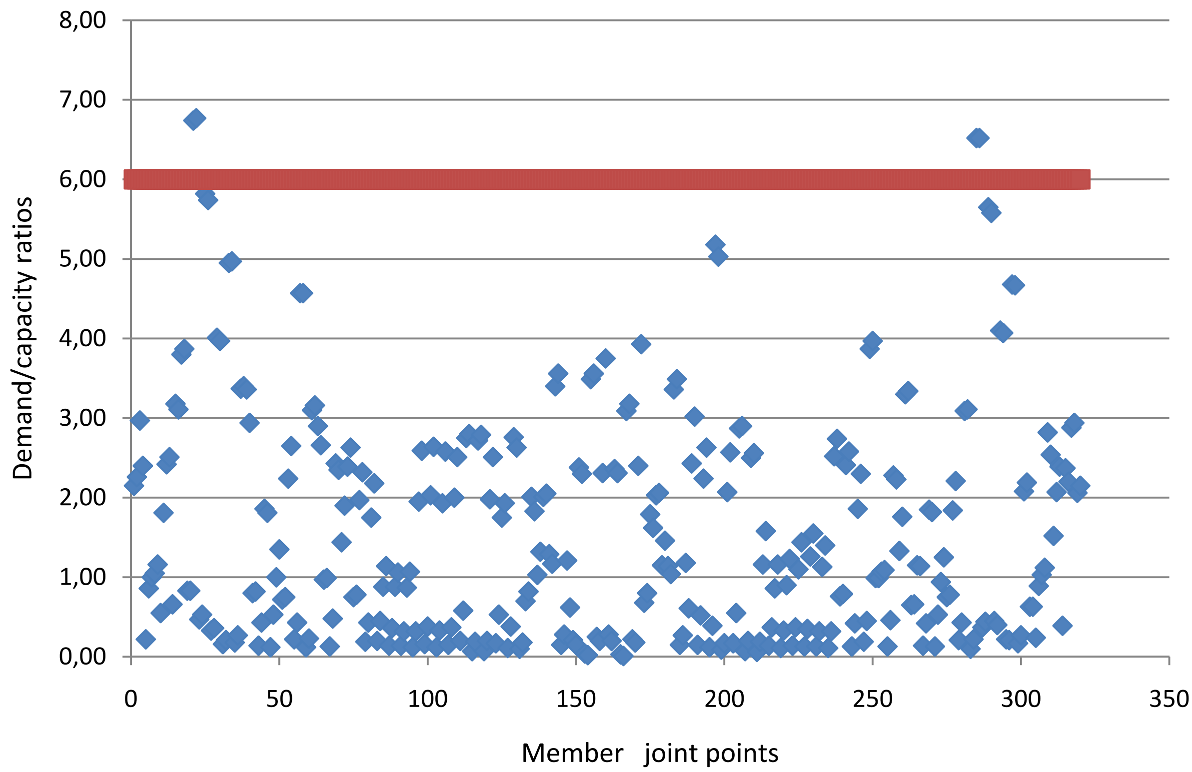

Figure A-36 –EY 1st Floor Column

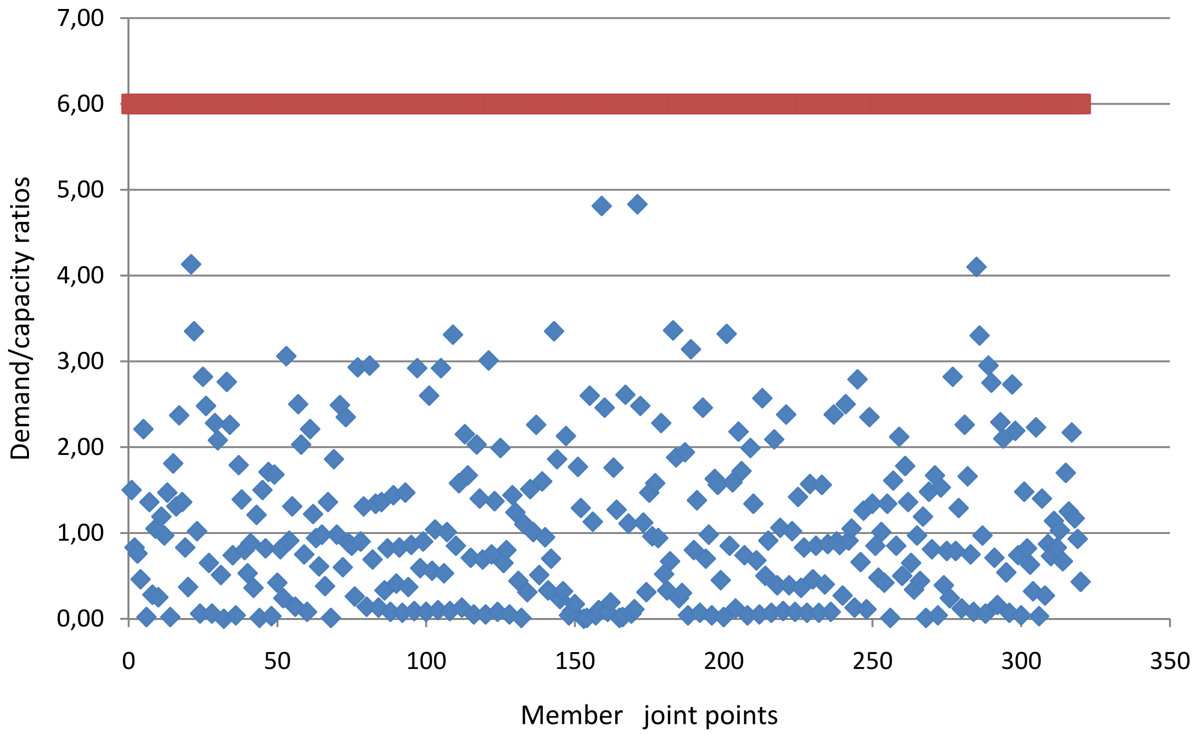

Figure A-37 –EY Ground Floor Column

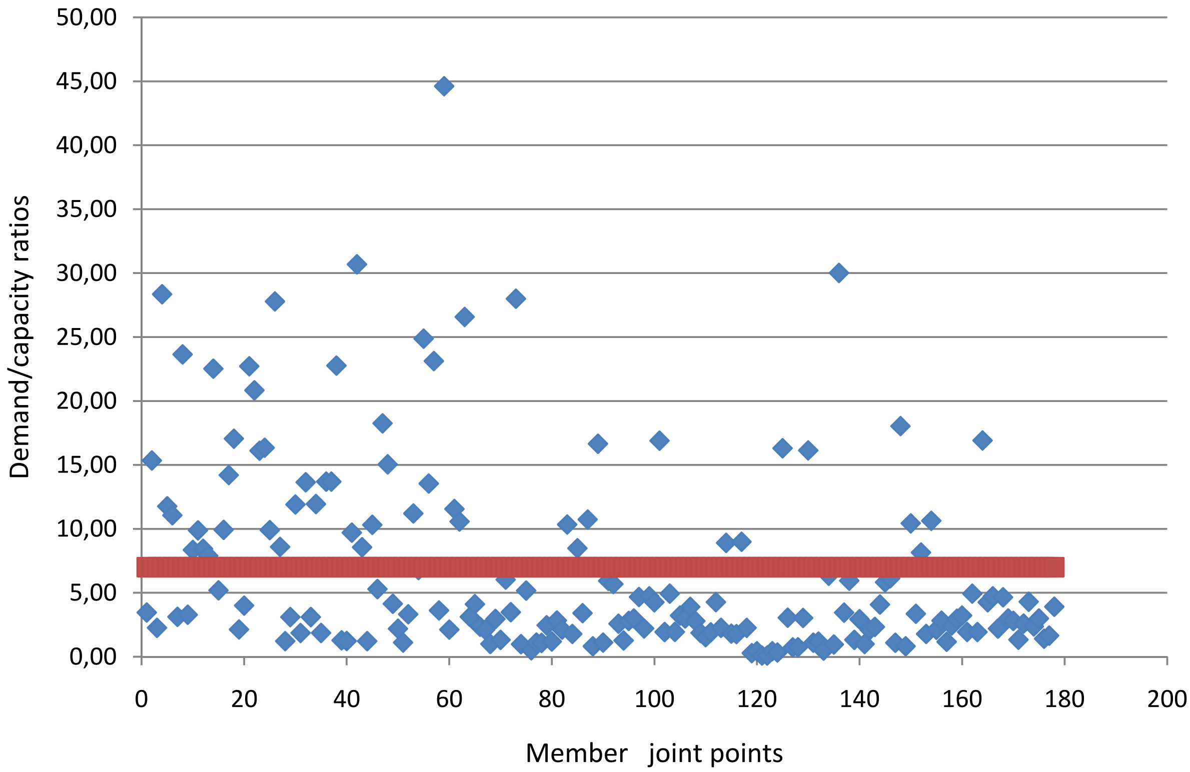

Figure A-38 +EY 2nd Floor Beam

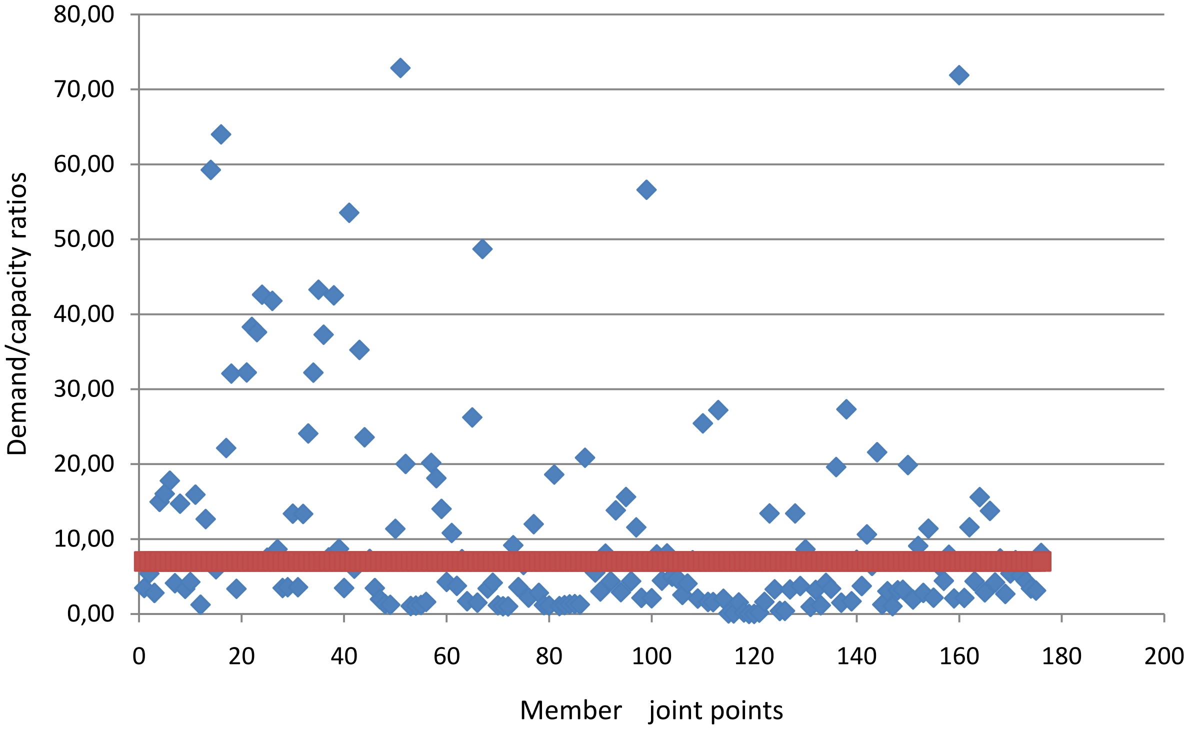

Figure A-39 +EY 1st Floor Beam

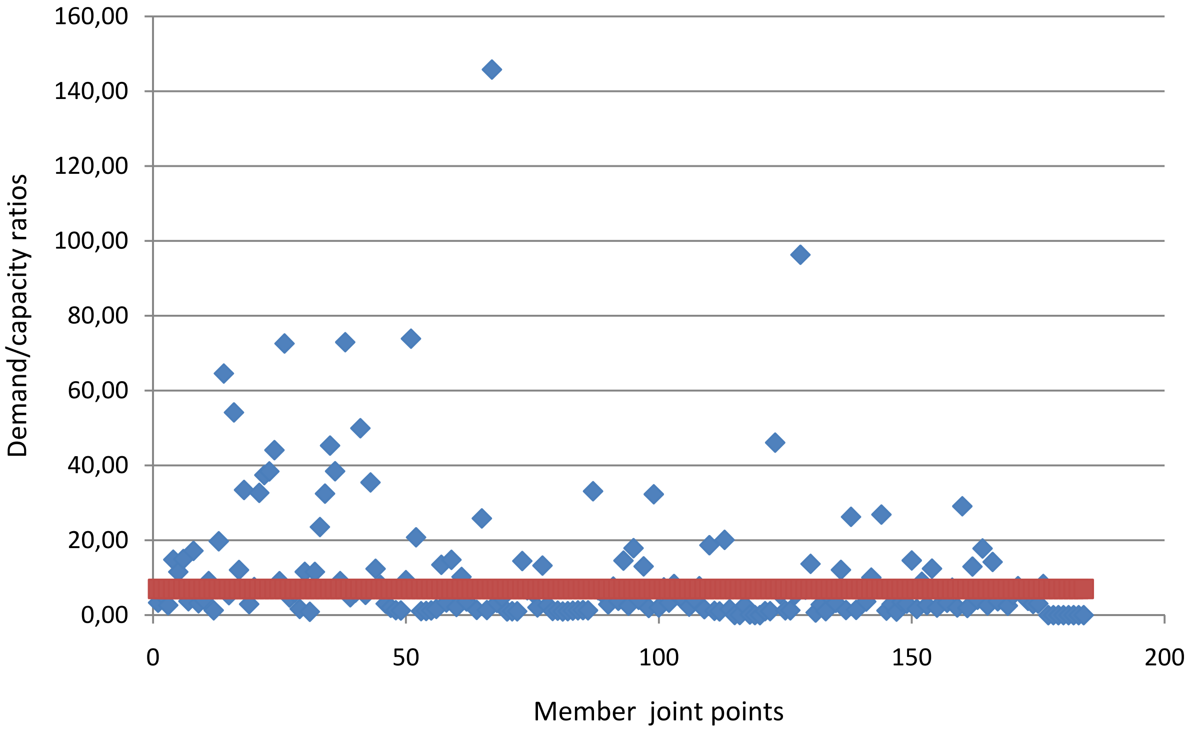

Figure A-40 +EY Ground Floor Beam

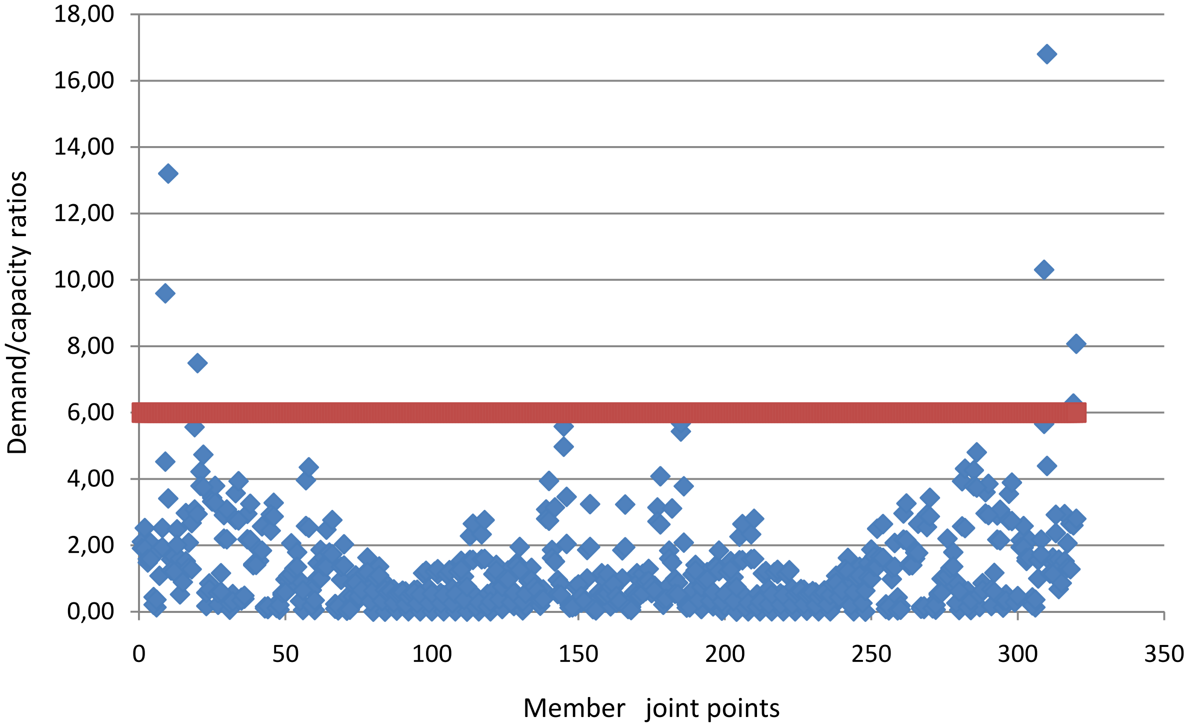

Figure A-41 +EY 2nd Floor Column

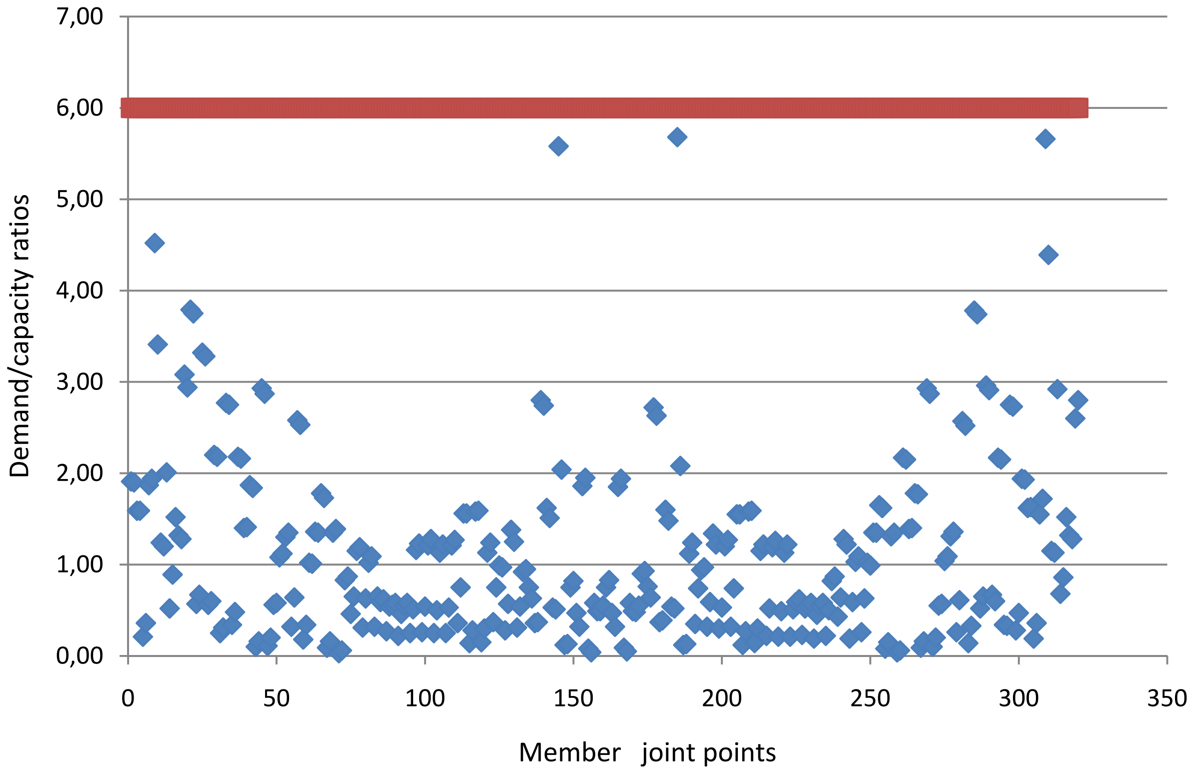

Figure A-42 +EY 1st Floor Column

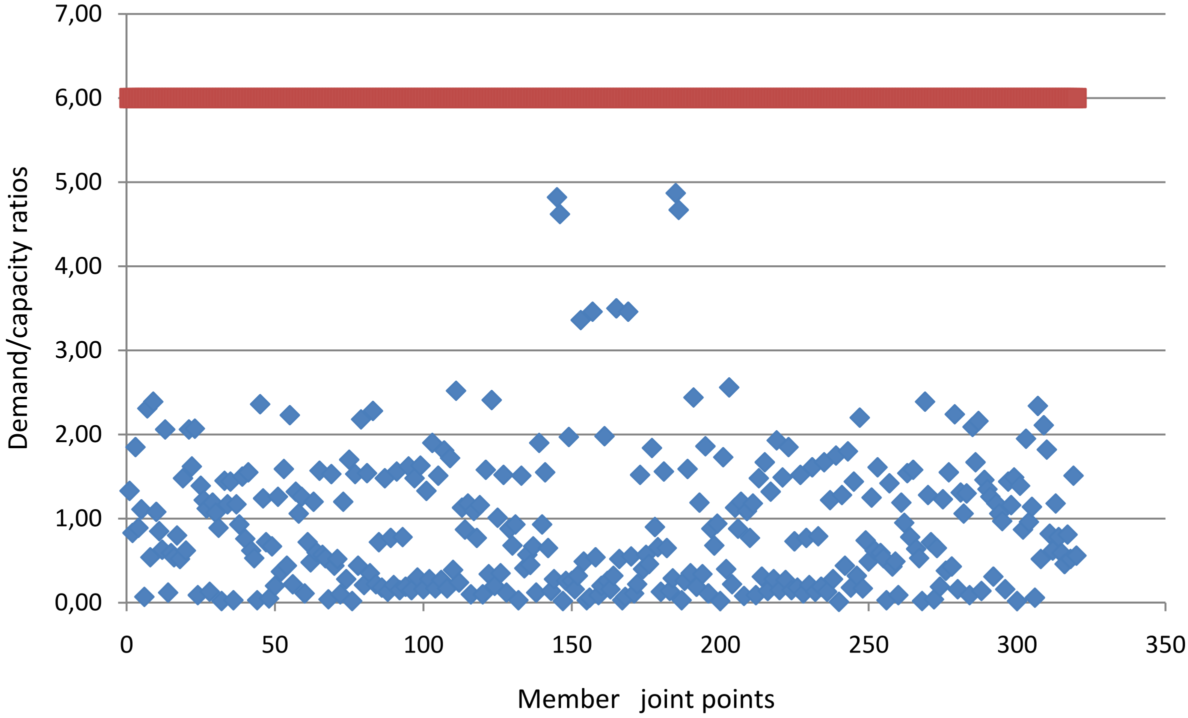

Figure A-43 +EY Ground Floor Column

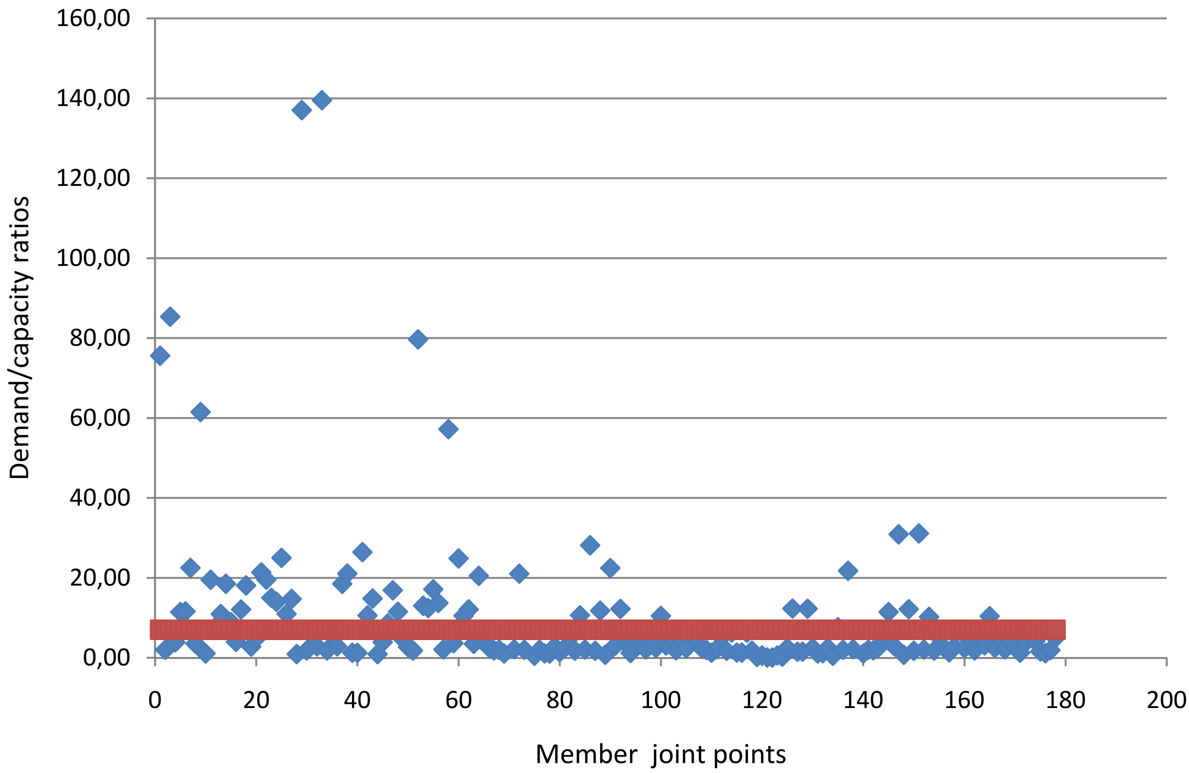

Figure A-44 -EY 2nd Floor Beam

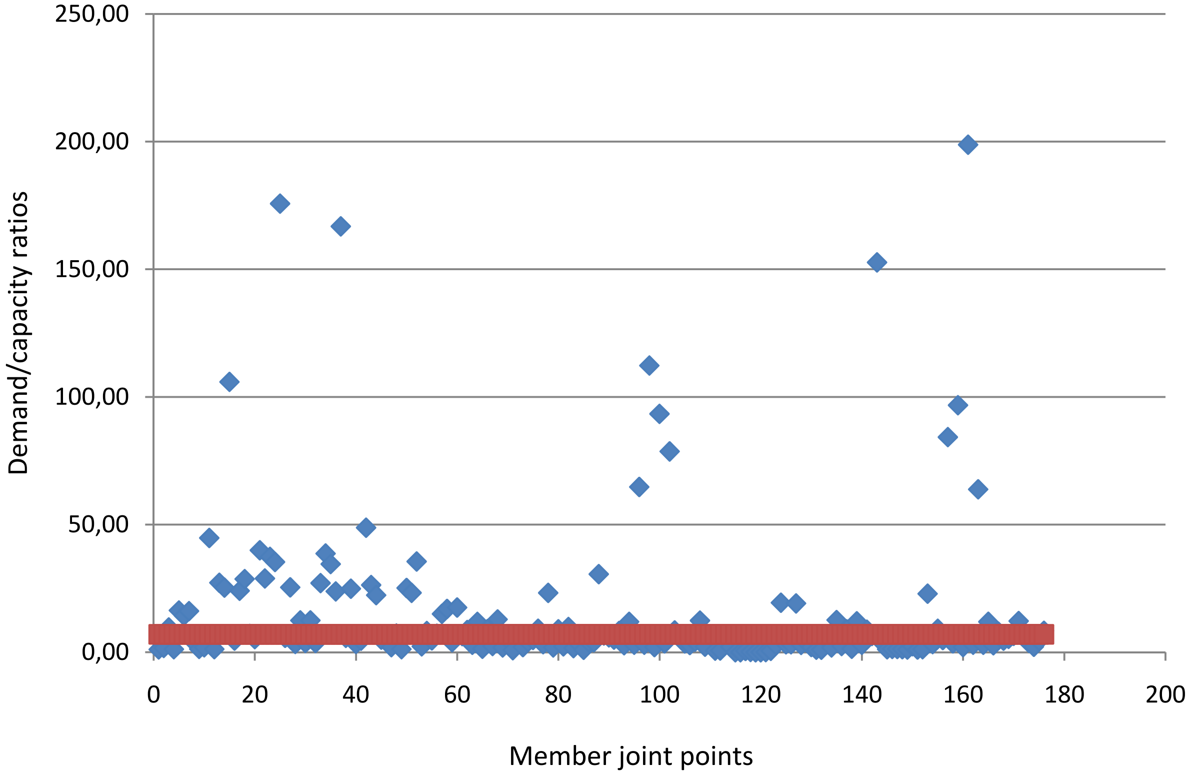

Figure A-45 -EY 1st Floor Beam

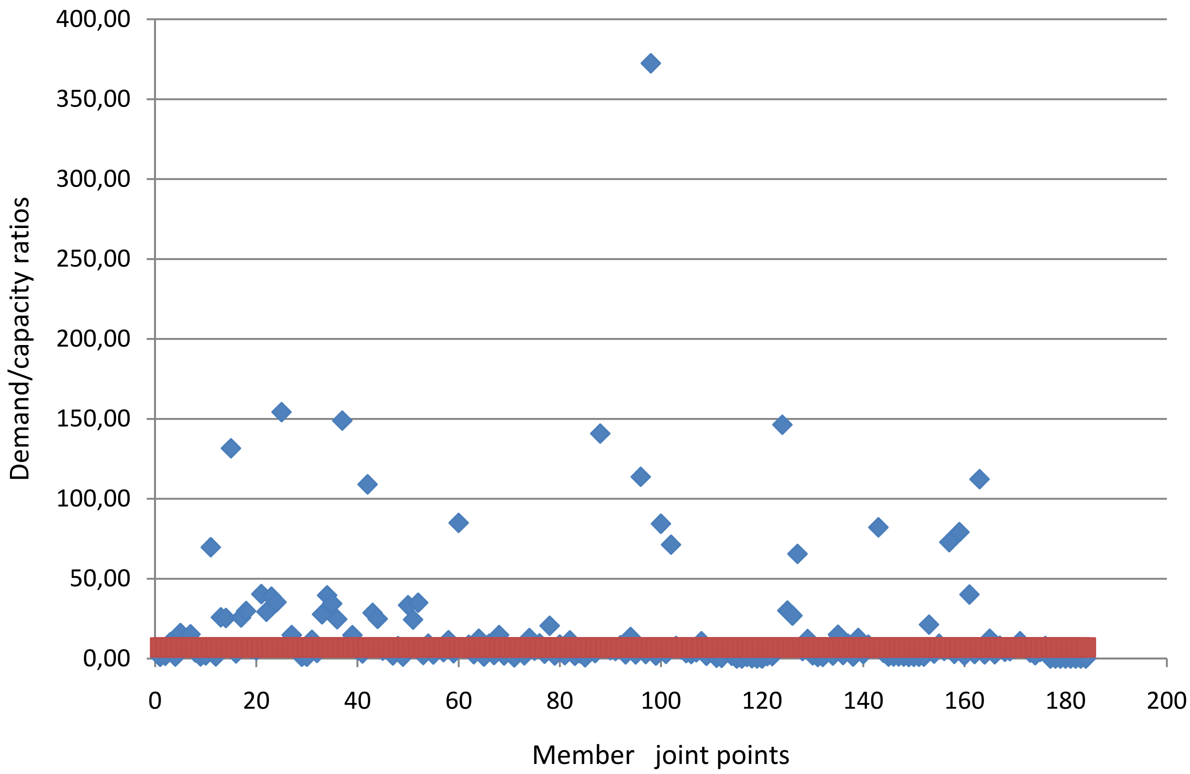

Figure A-46 -EY Ground Floor Beam

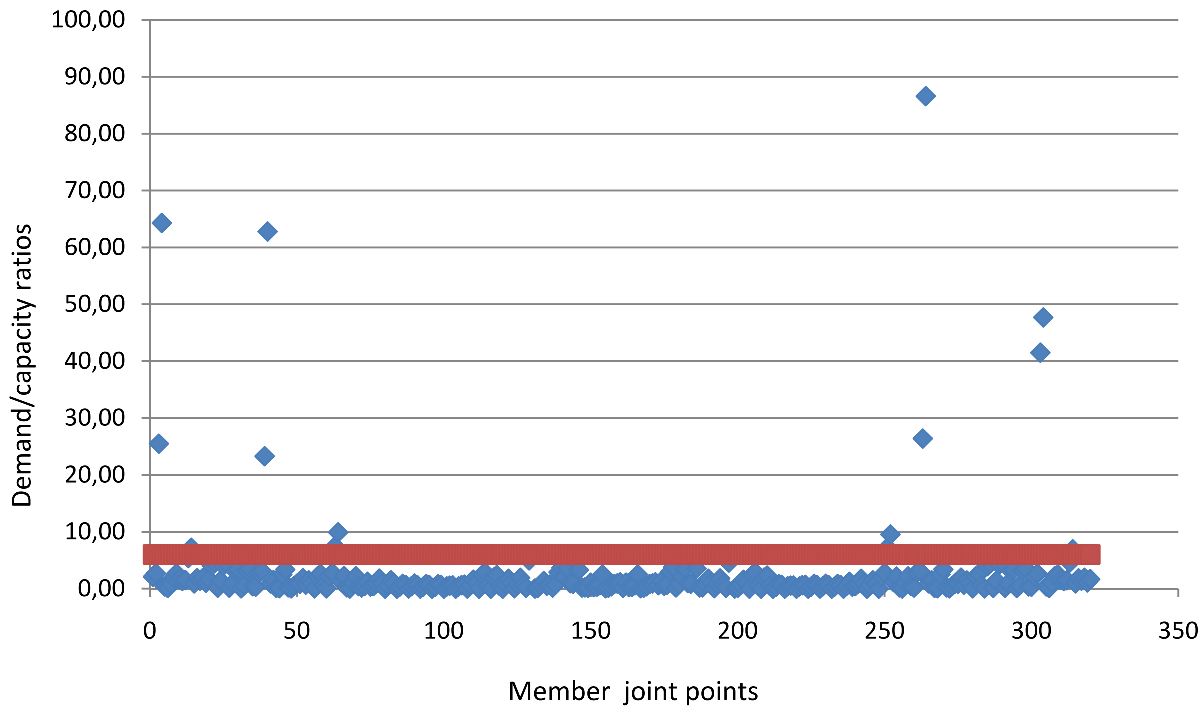

Figure A-47 -EY 2ndFloor Column

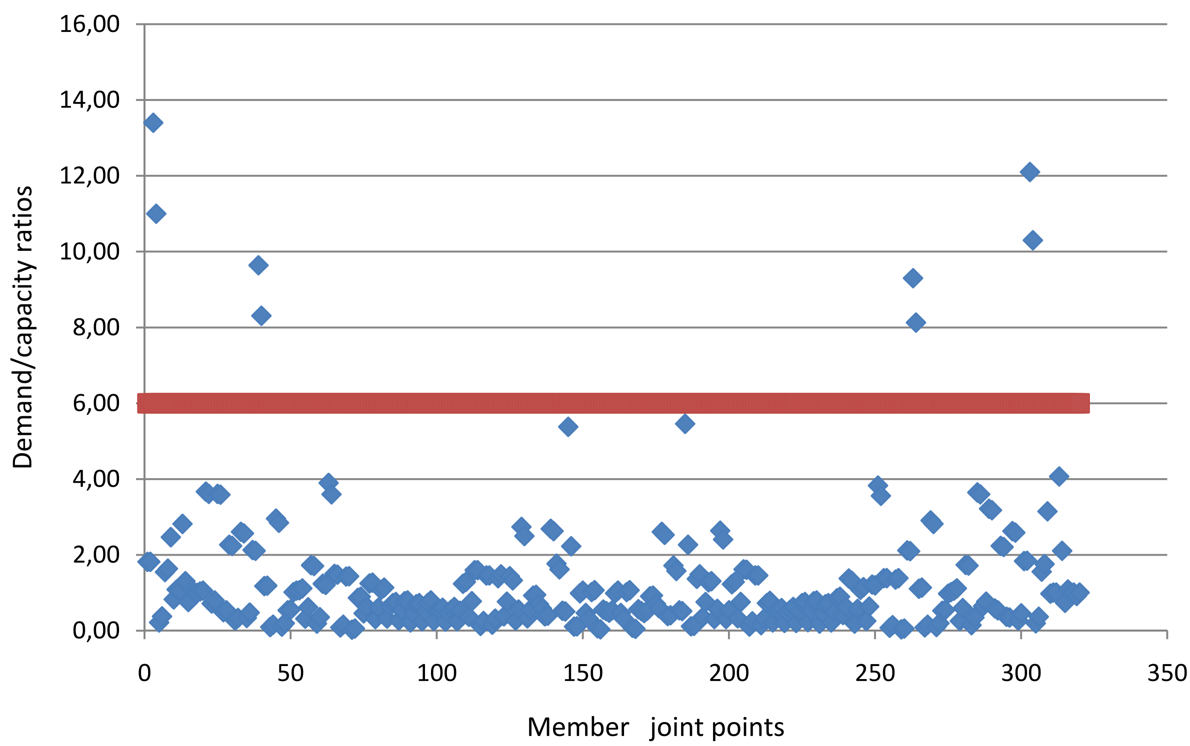

Figure A-48 -EY 1st Floor Column

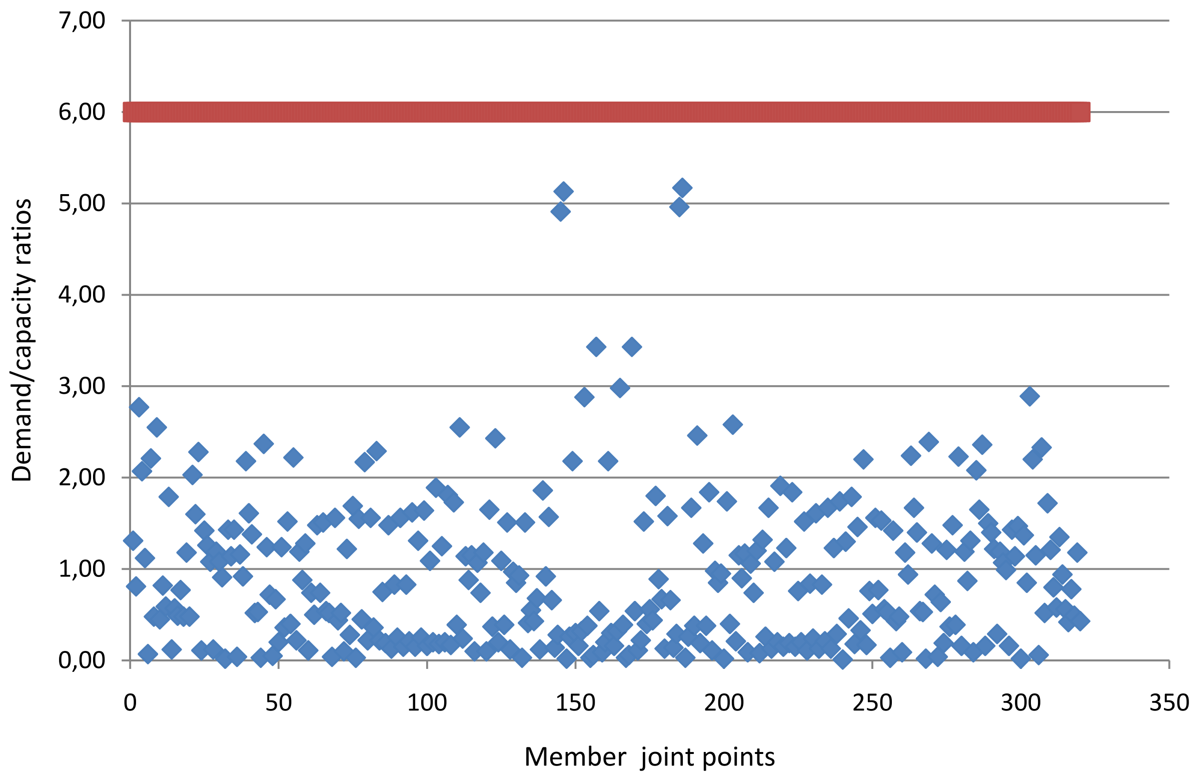

Figure A-49 -EY Ground Floor Column

A.3 Shear Wall

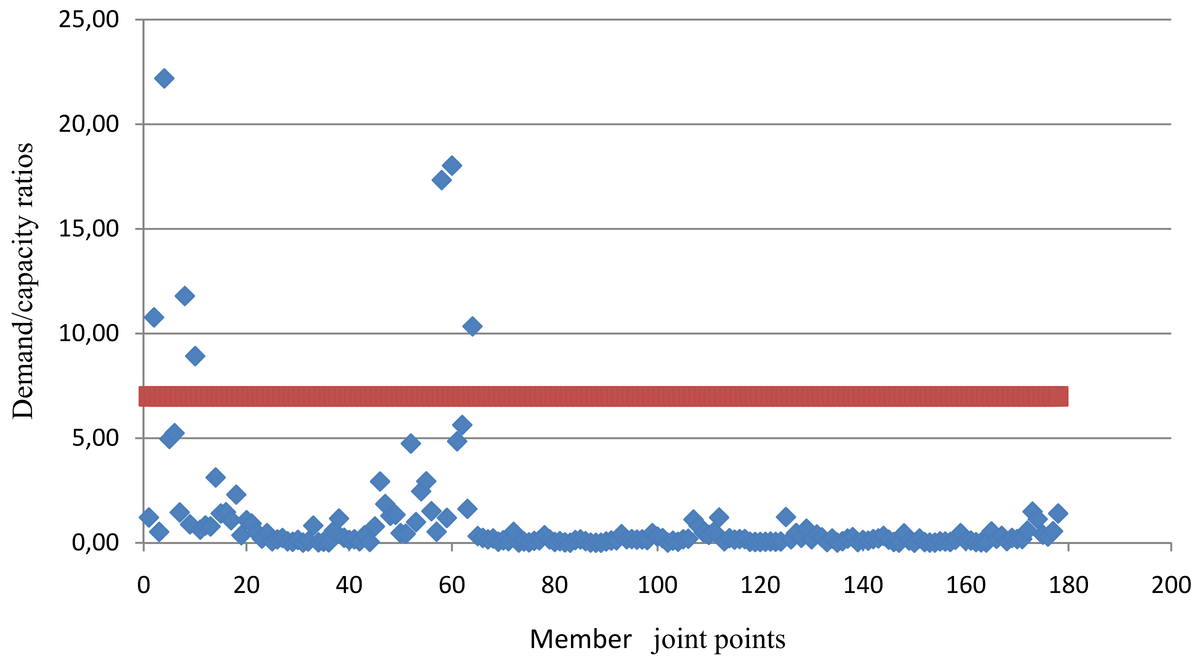

Figure A-50 +EX 2nd Floor Beam

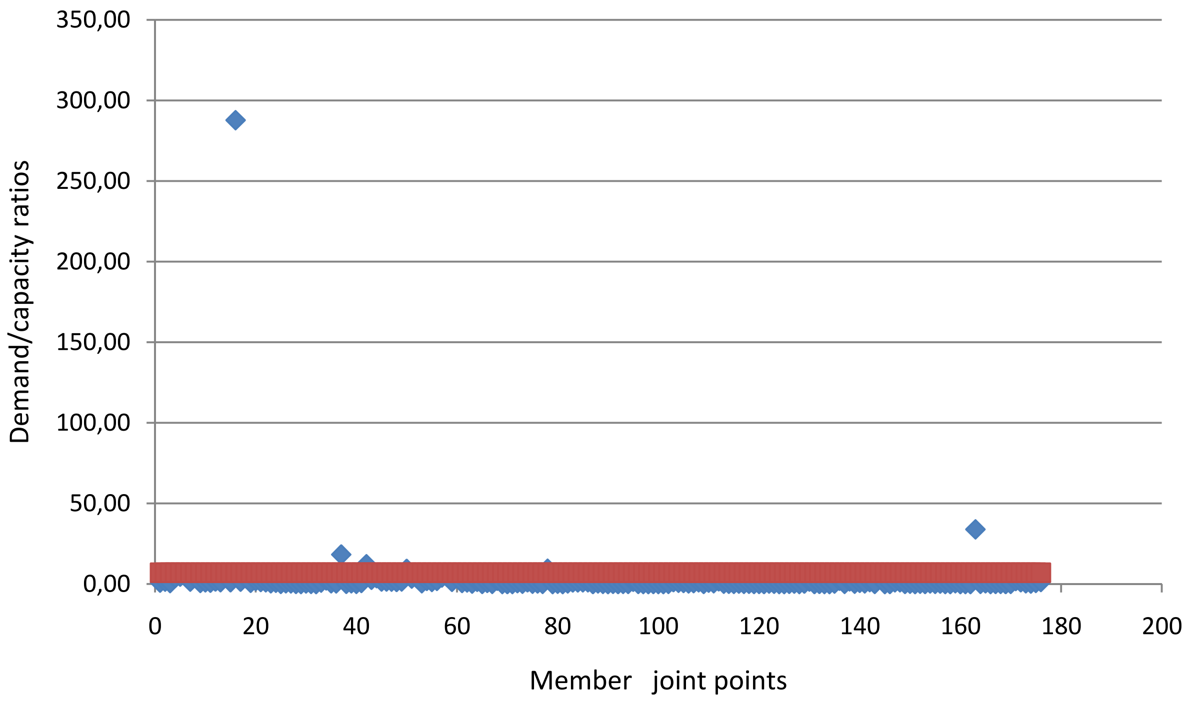

Figure A-51 +EX 1st Floor Beam

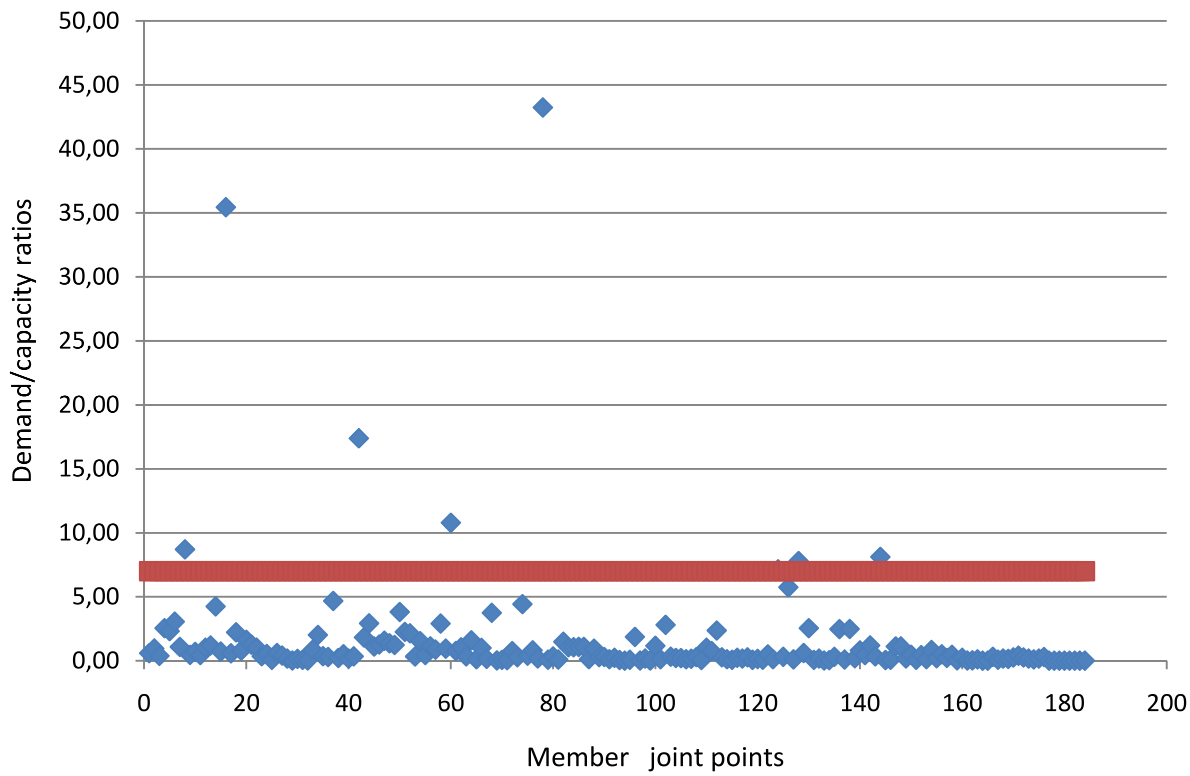

Figure A-52 +EX Ground Floor Beam

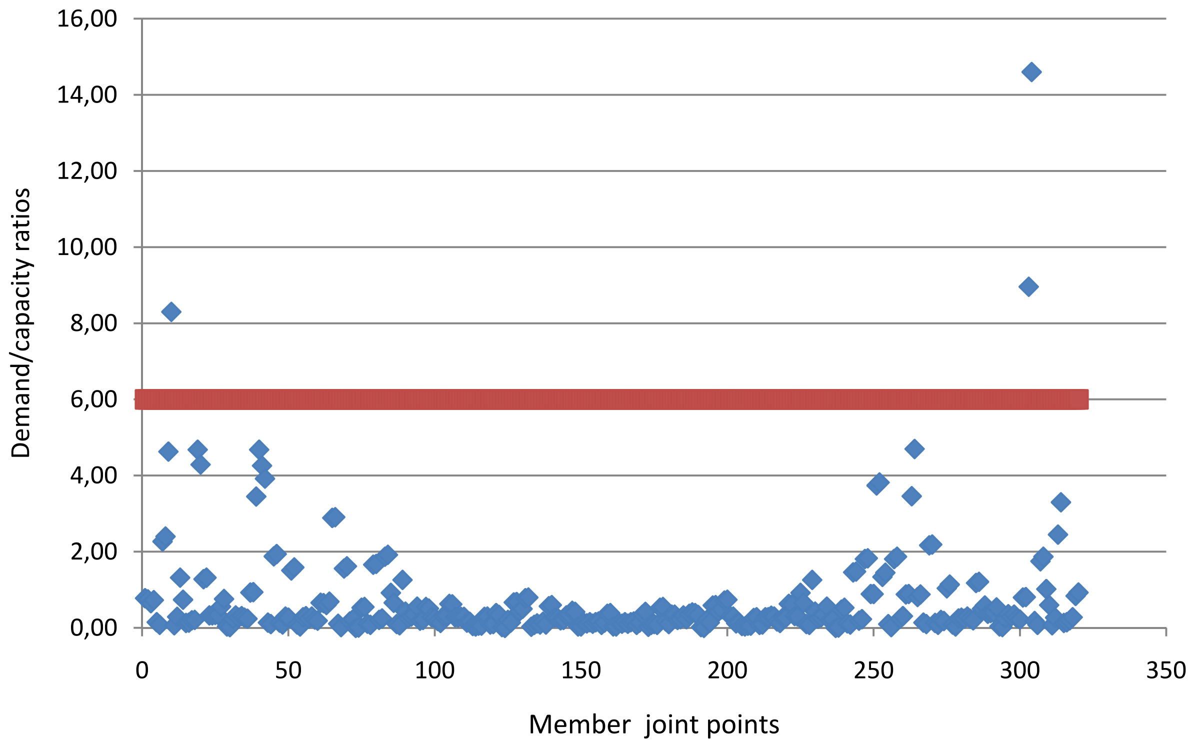

Figure A-53 +EX 2nd Floor Column

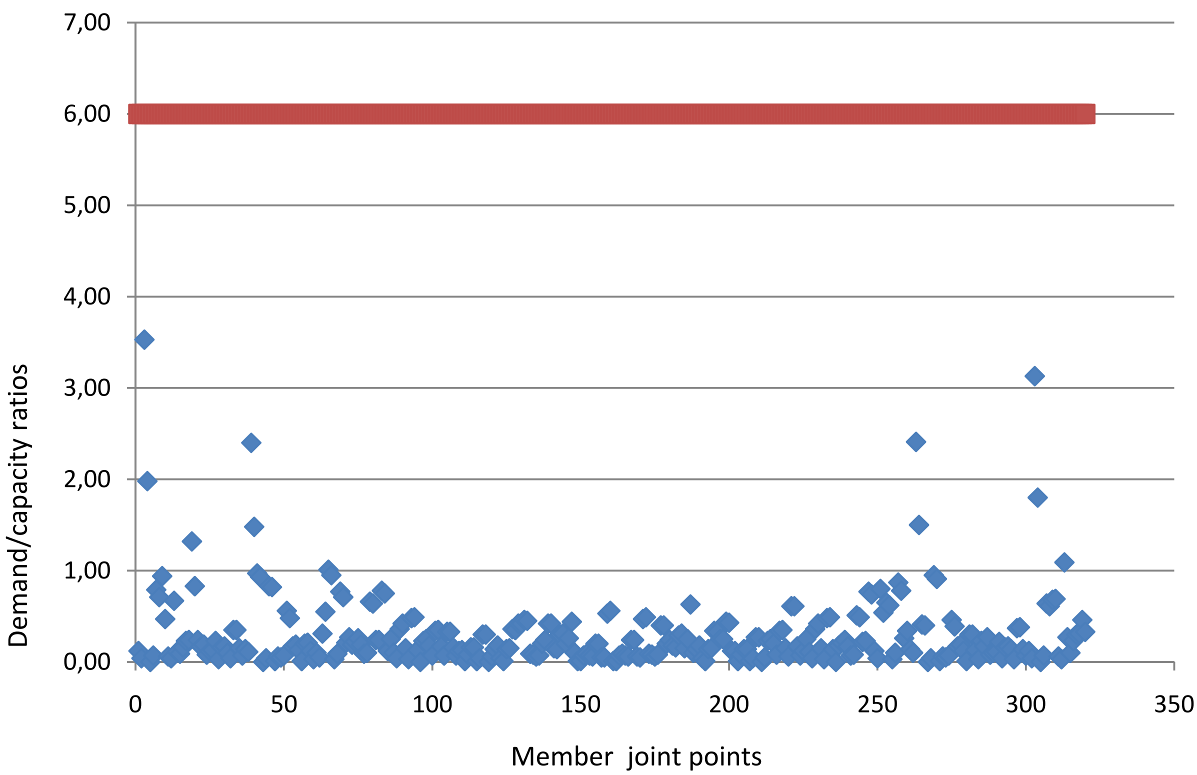

Figure A-54 +EX 1st Floor Column

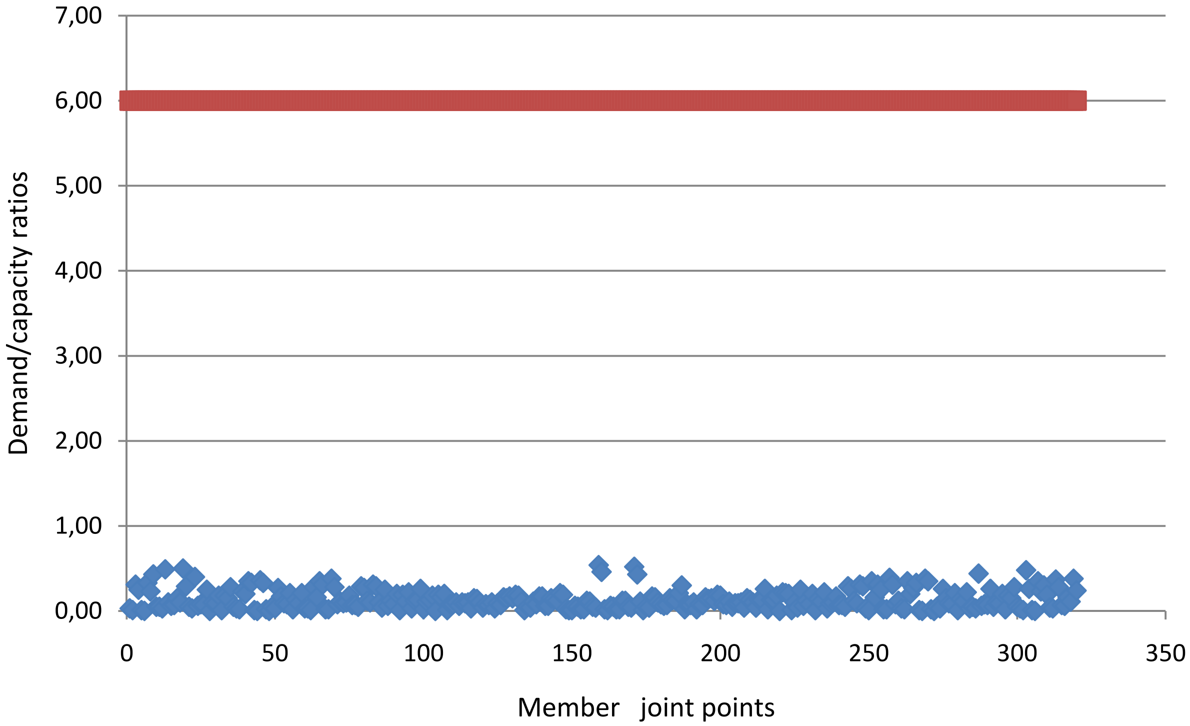

Figure A-55 +EX Ground Floor Column

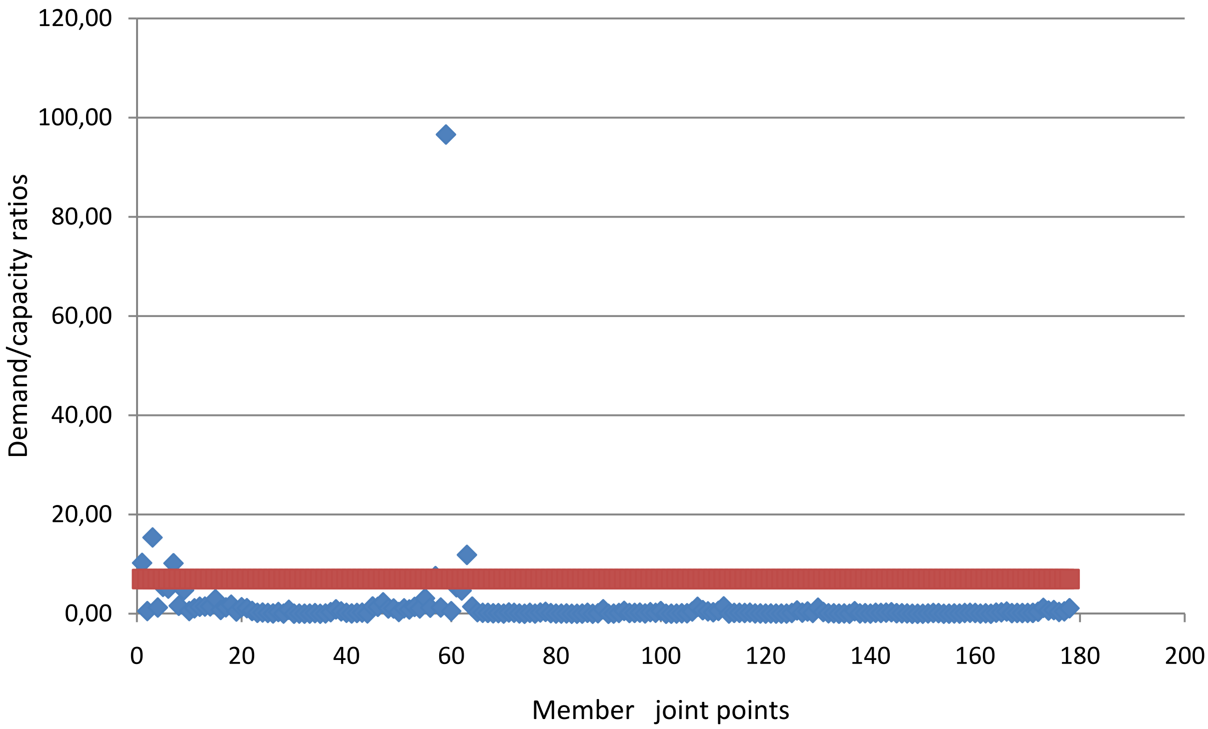

Figure A-56 -EX 2nd Floor Beam

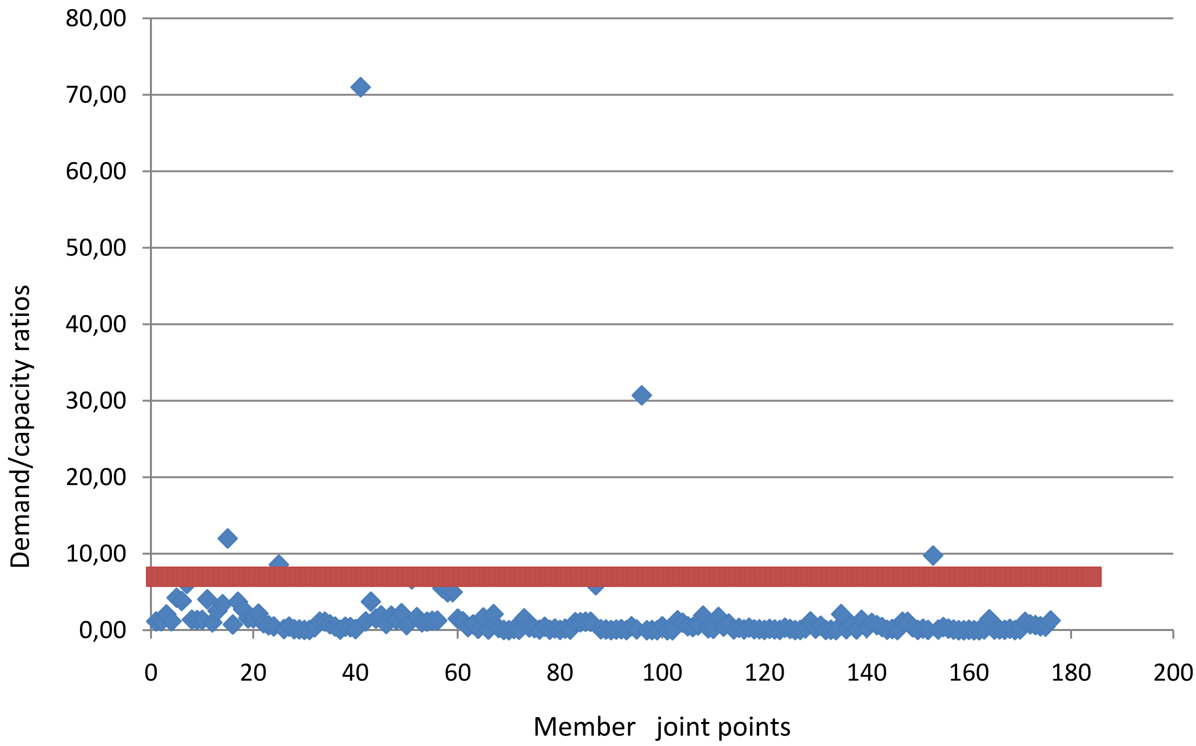

Figure A-57 -EX 1st Floor Beam

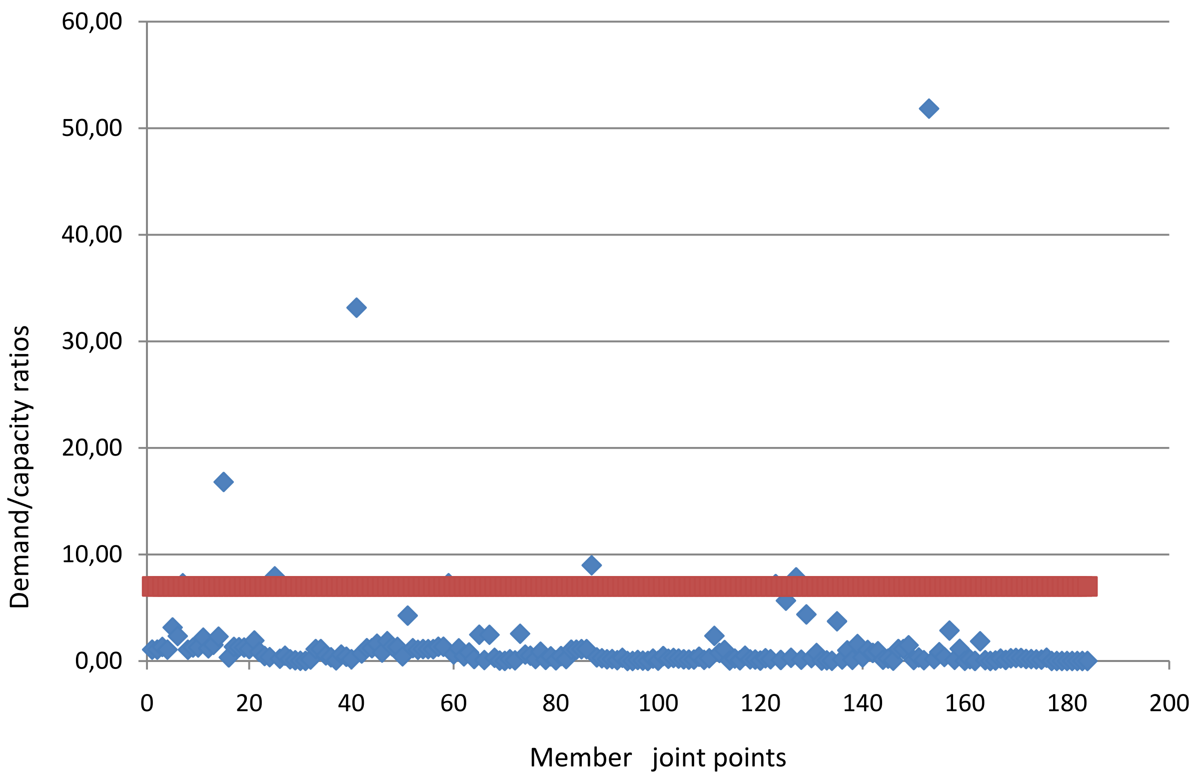

Figure A-58 -EX Ground Floor Beam

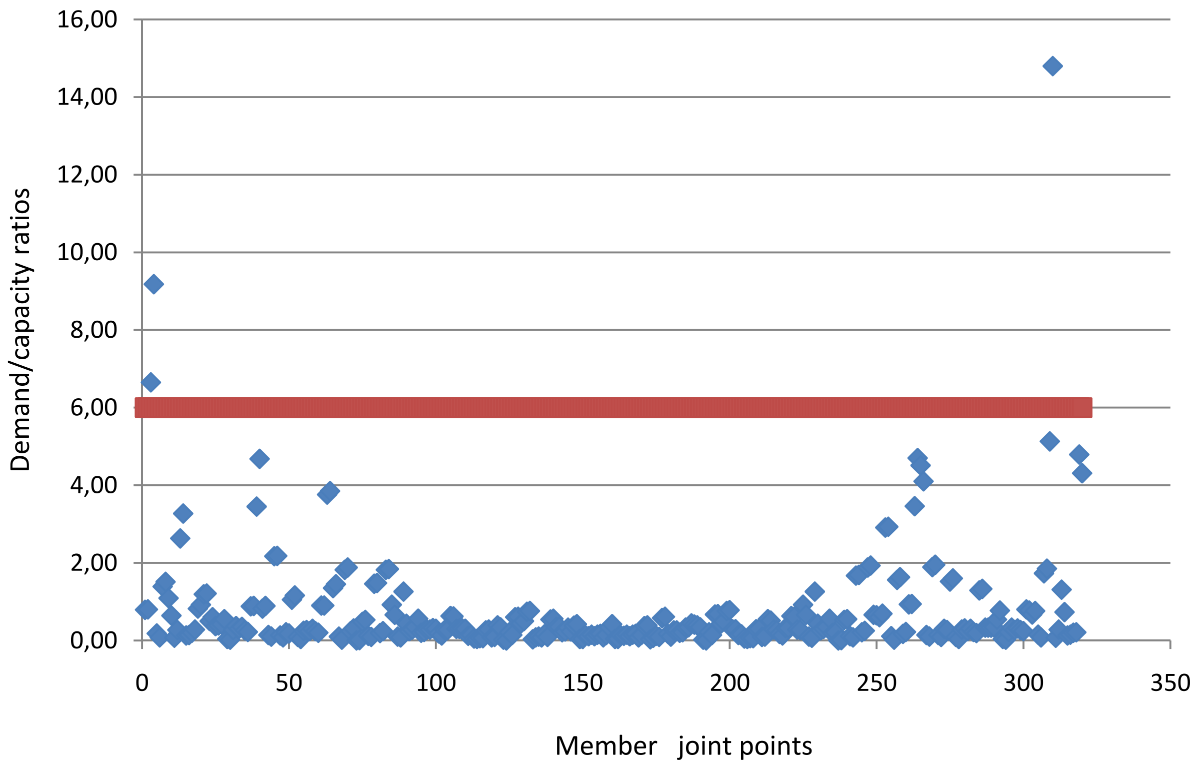

Figure A-59 -EX 2nd Floor Column

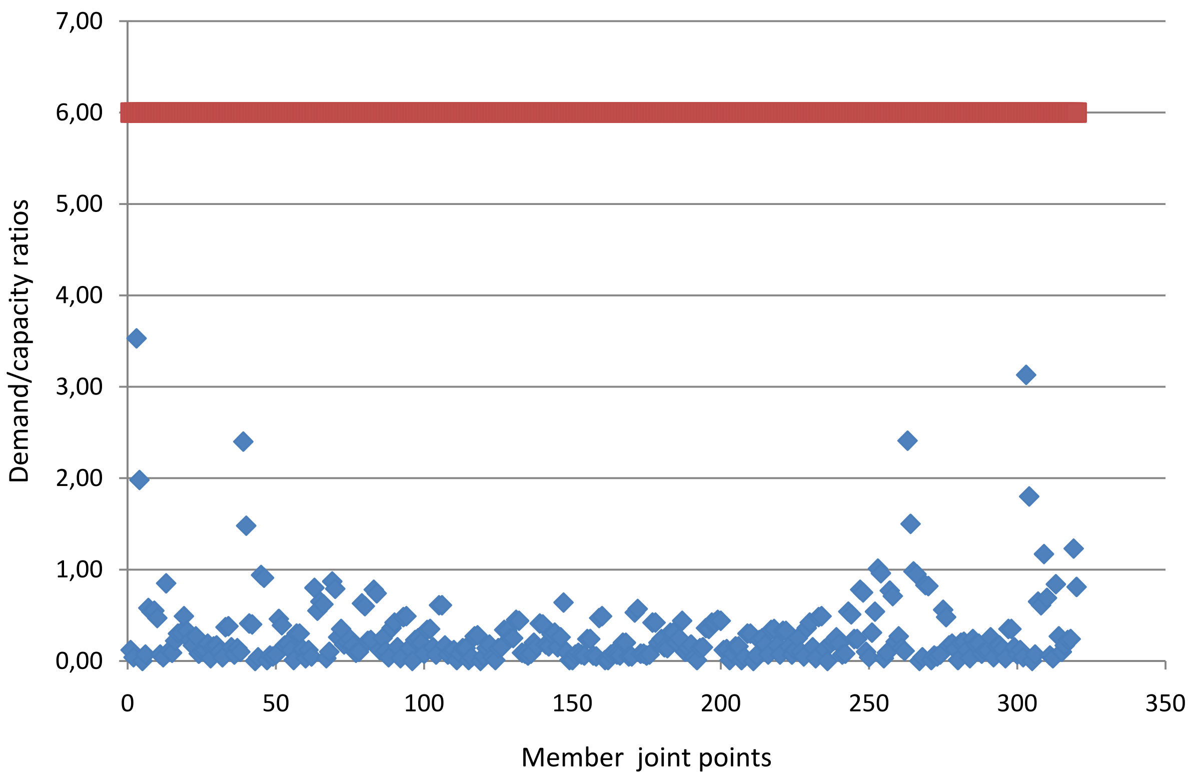

Figure A-60 -EX 1st Floor Column

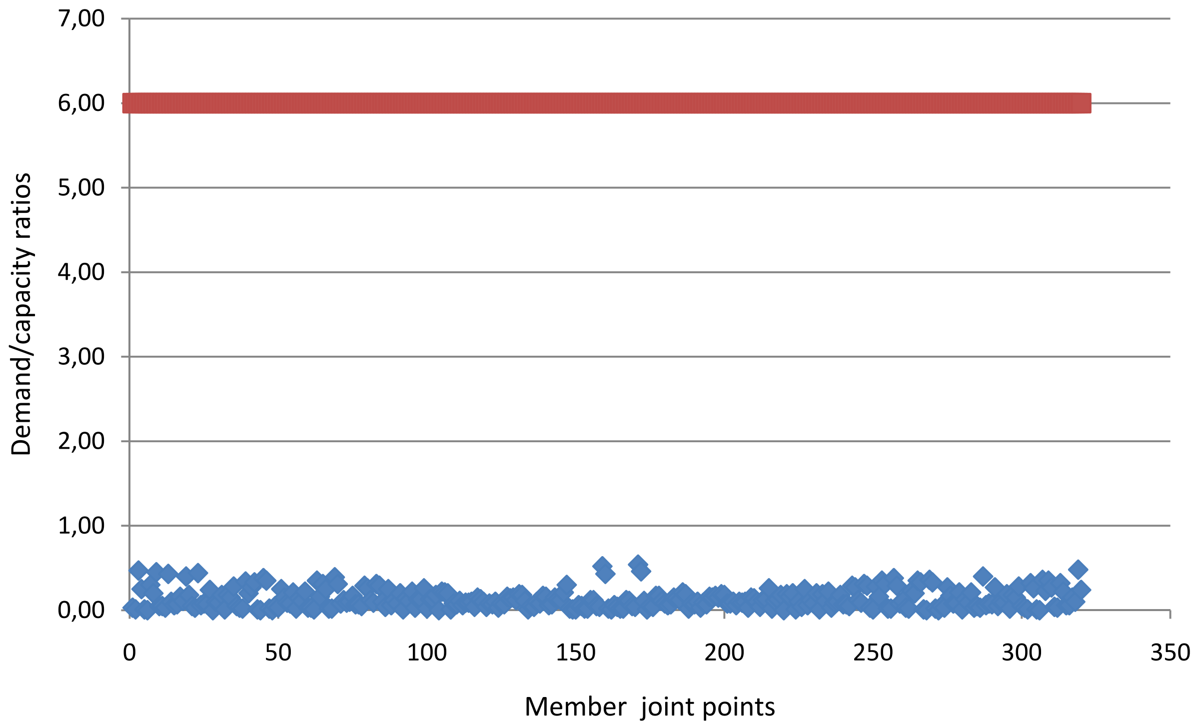

Figure A-61 -EX Ground Floor Column

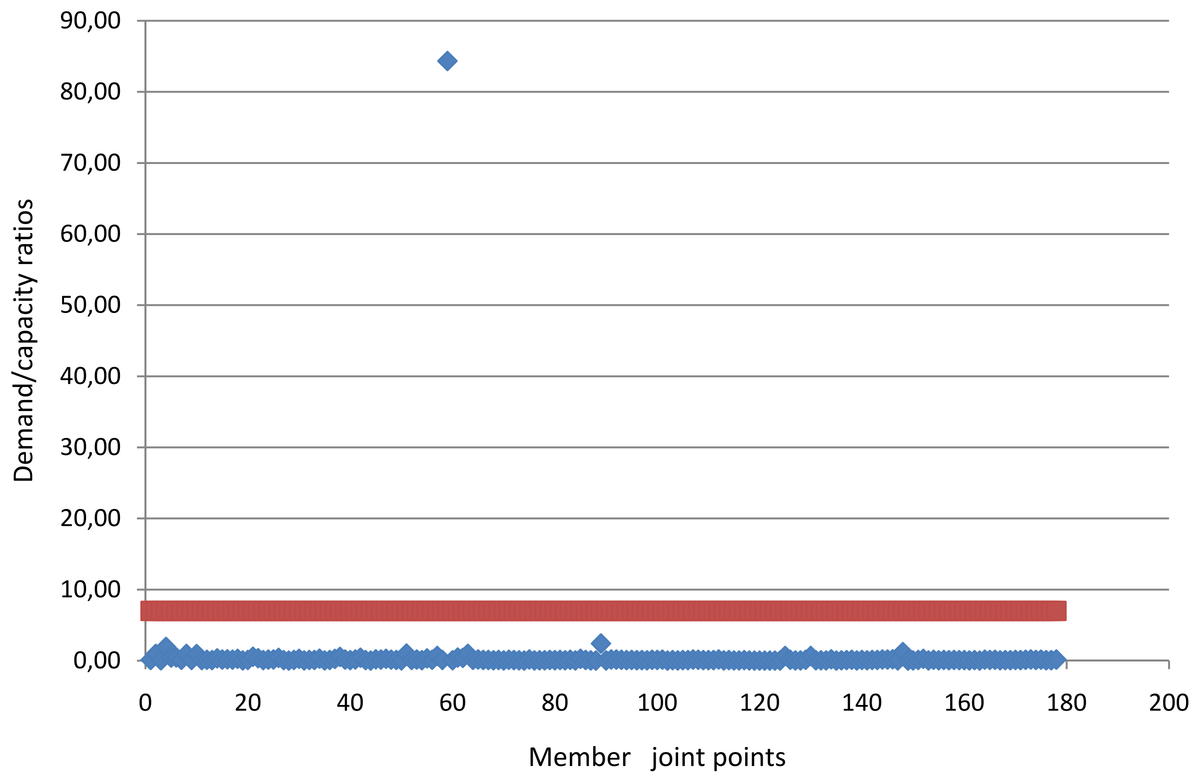

Figure A-62 +EY 2nd Floor Beam

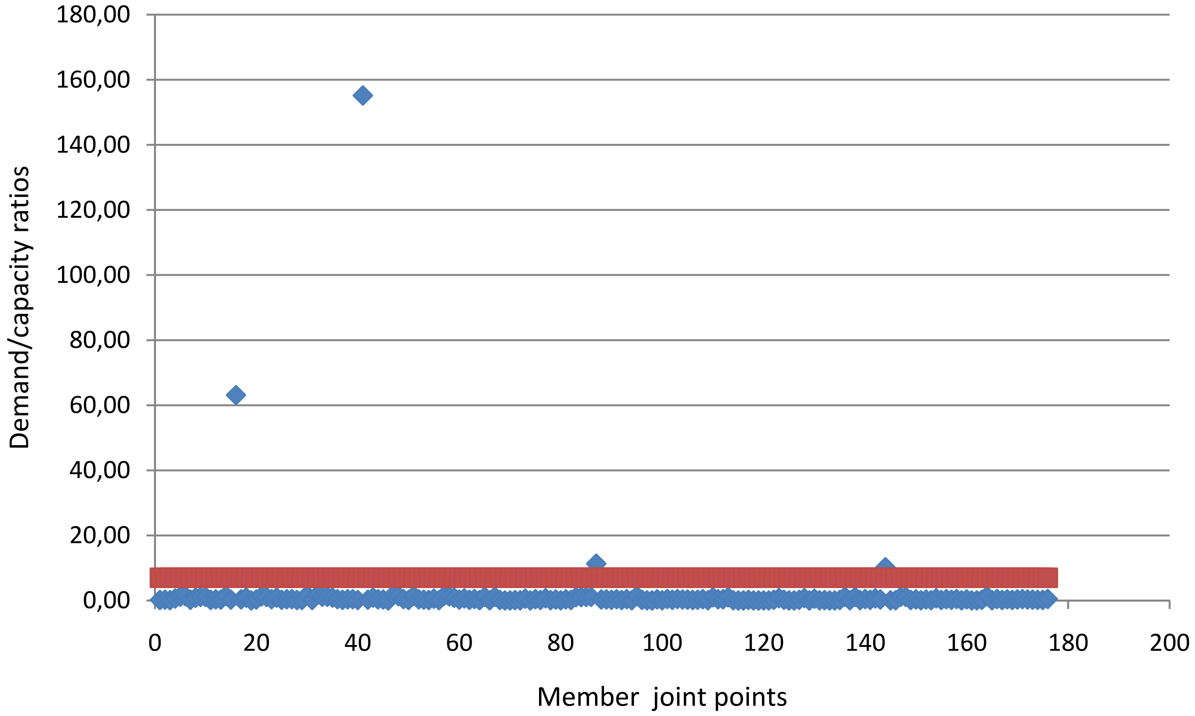

Figure A-63 +EY 1st Floor Beam

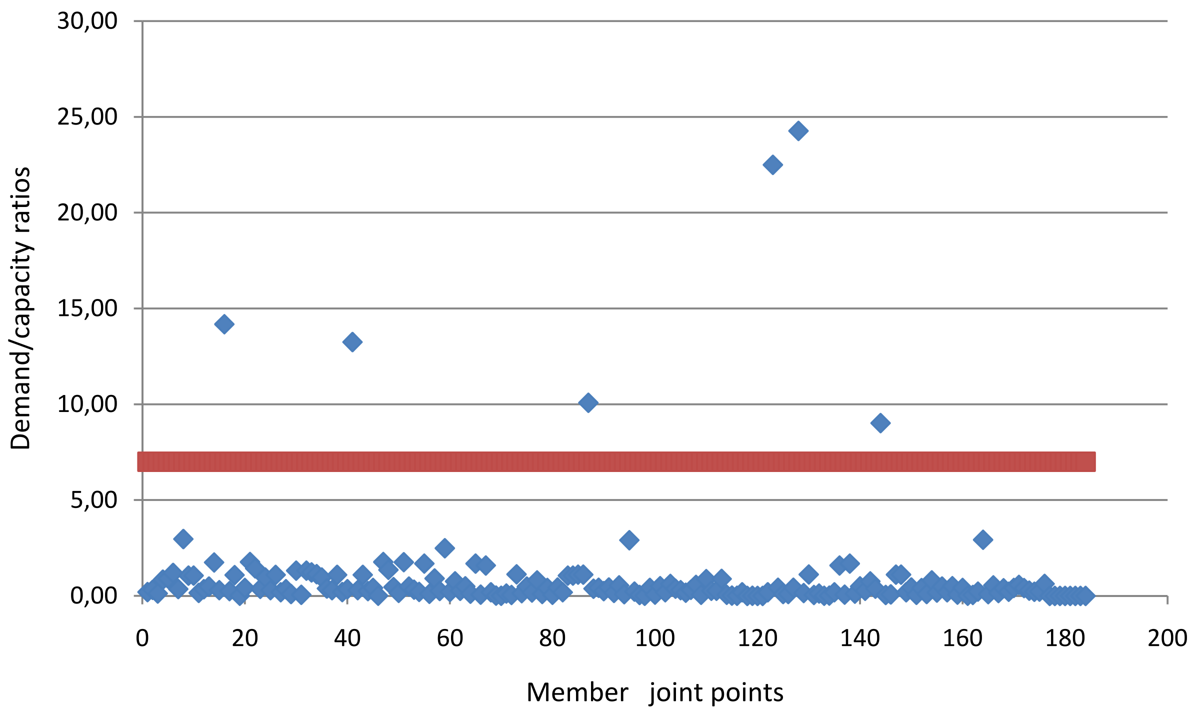

Figure A-64 +EY Ground Floor Beam

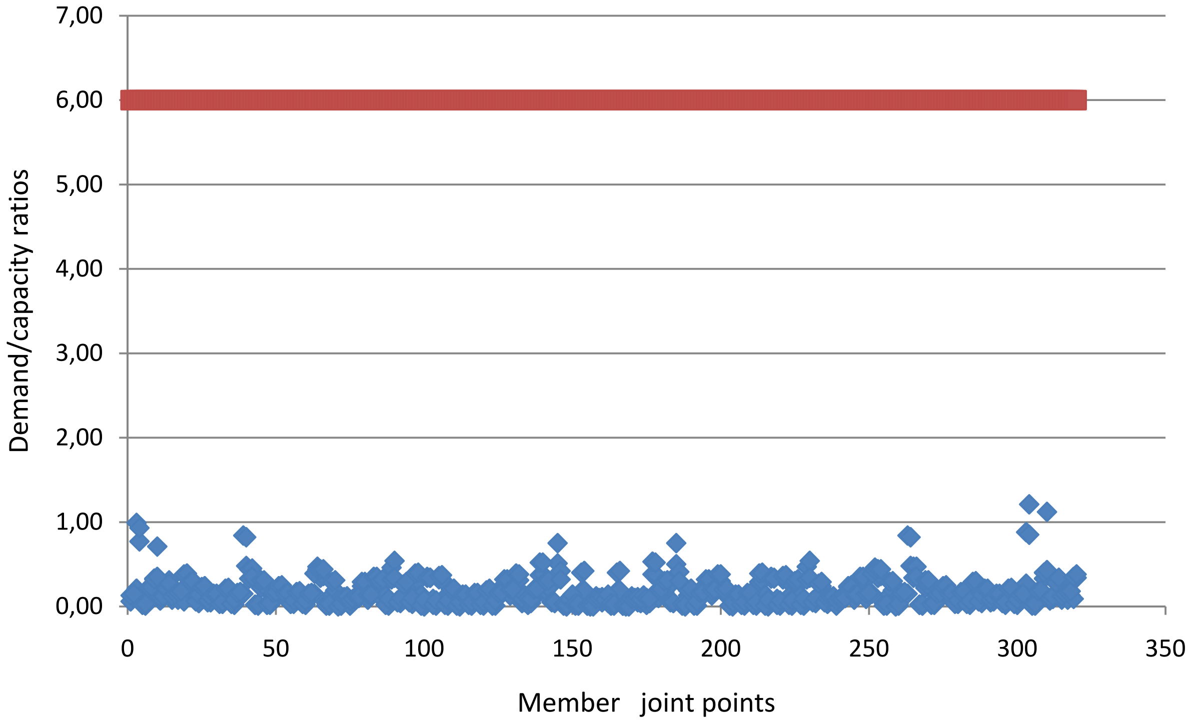

Figure A-65 +EY 2nd Floor Column

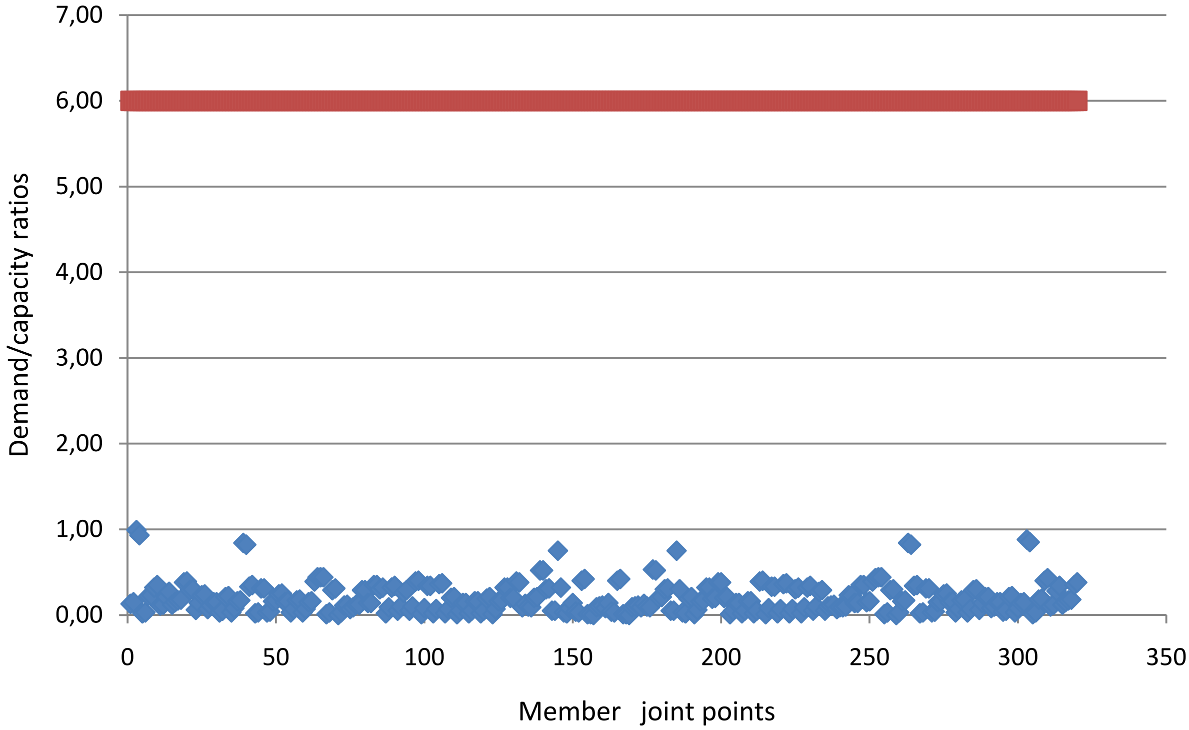

Figure A-66 +EY 1st Floor Column

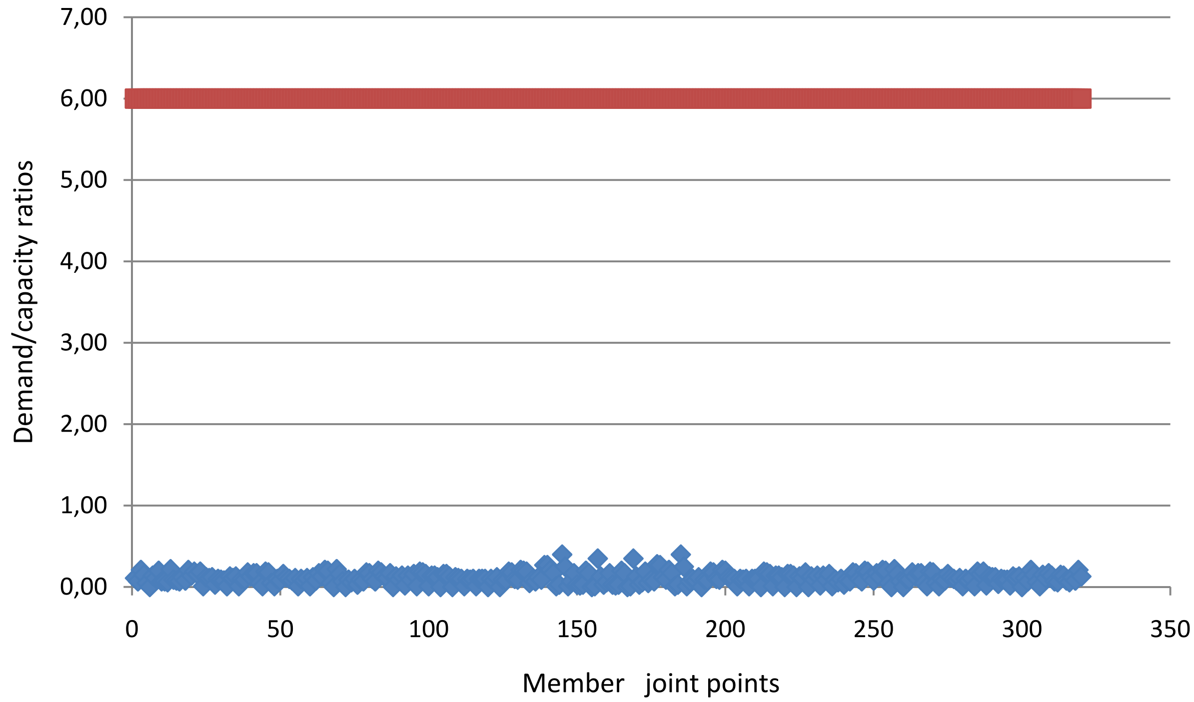

Figure A-67 +EY Ground Floor Column

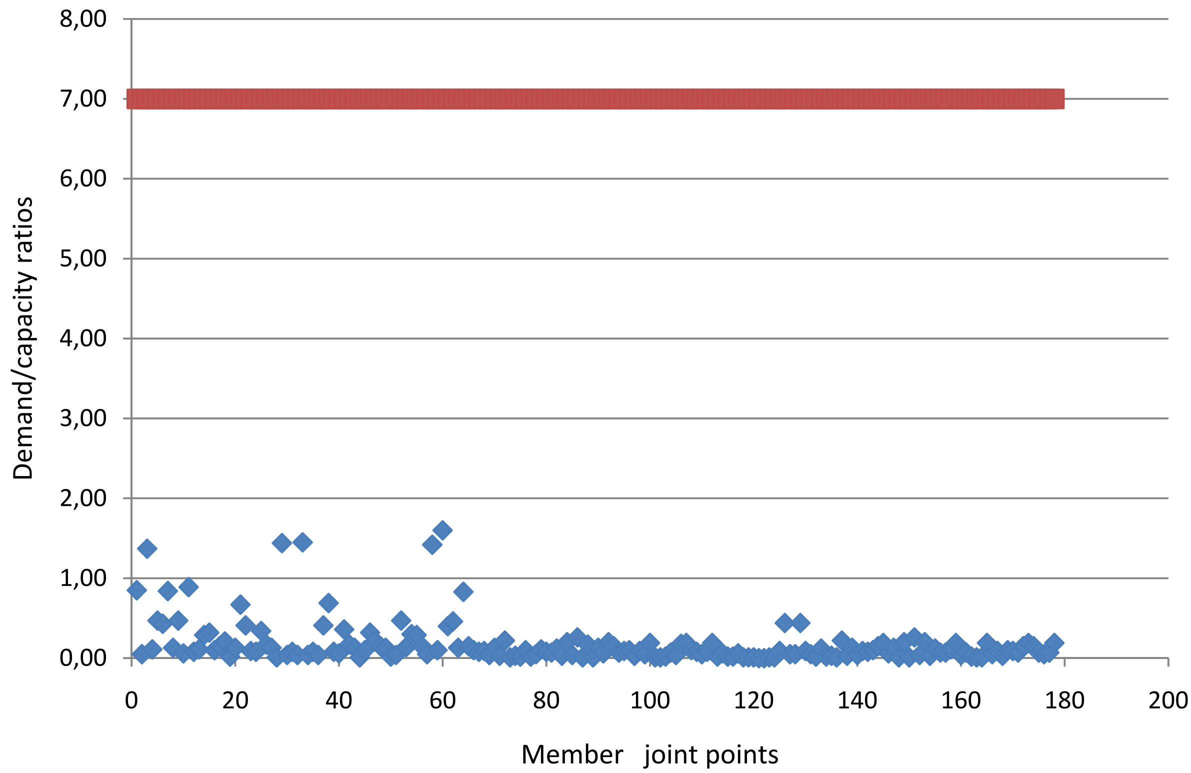

Figure A-68 -EY 1st Floor Beam

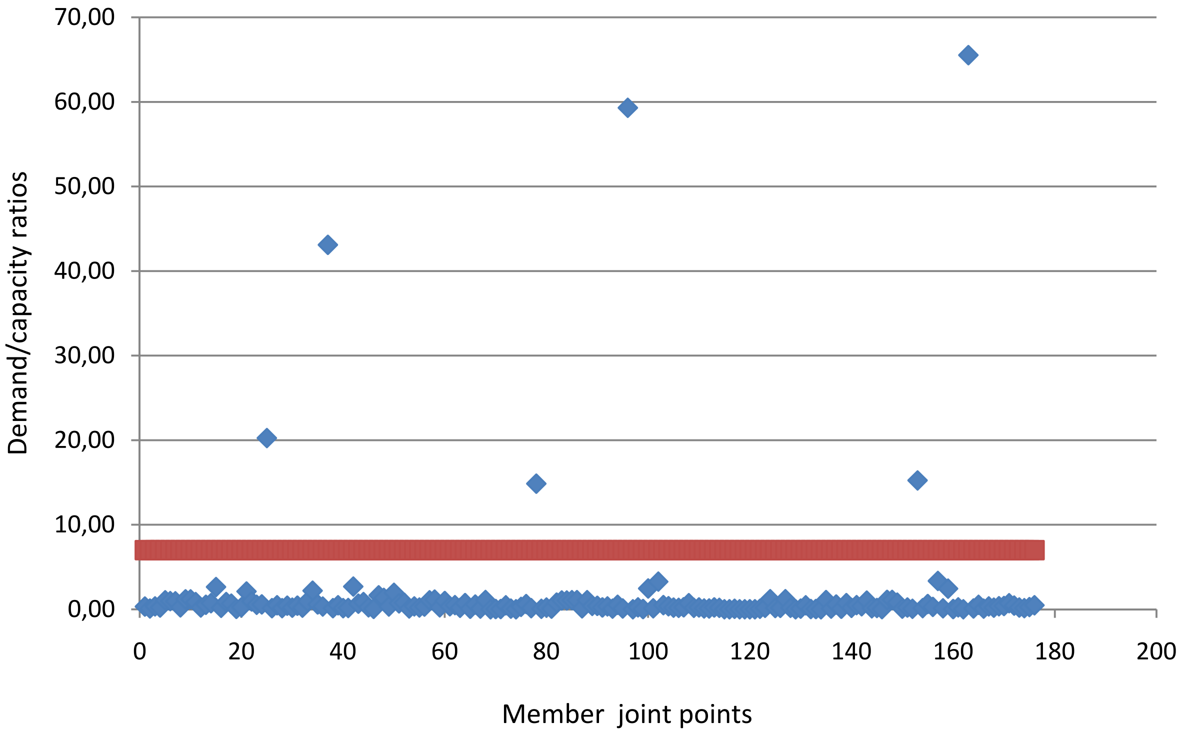

Figure A-69 -EY 1st Floor Beam

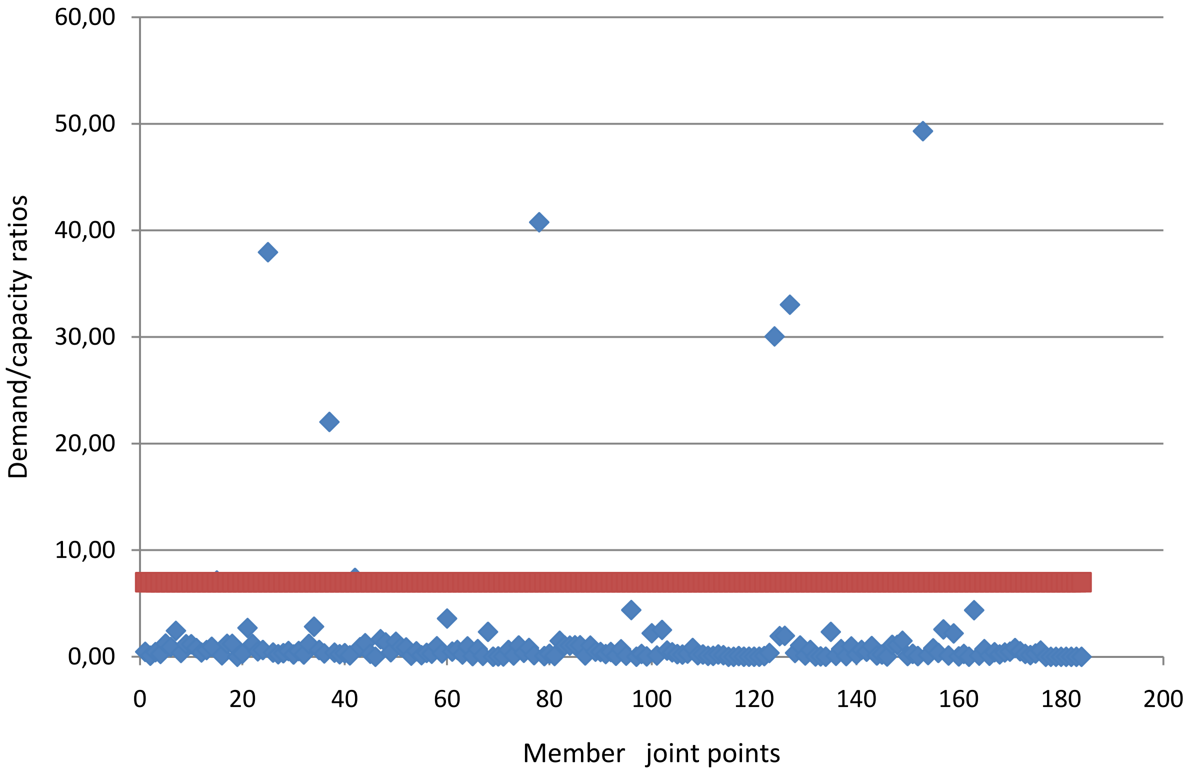

Figure A-70 –EY Ground Floor Beam

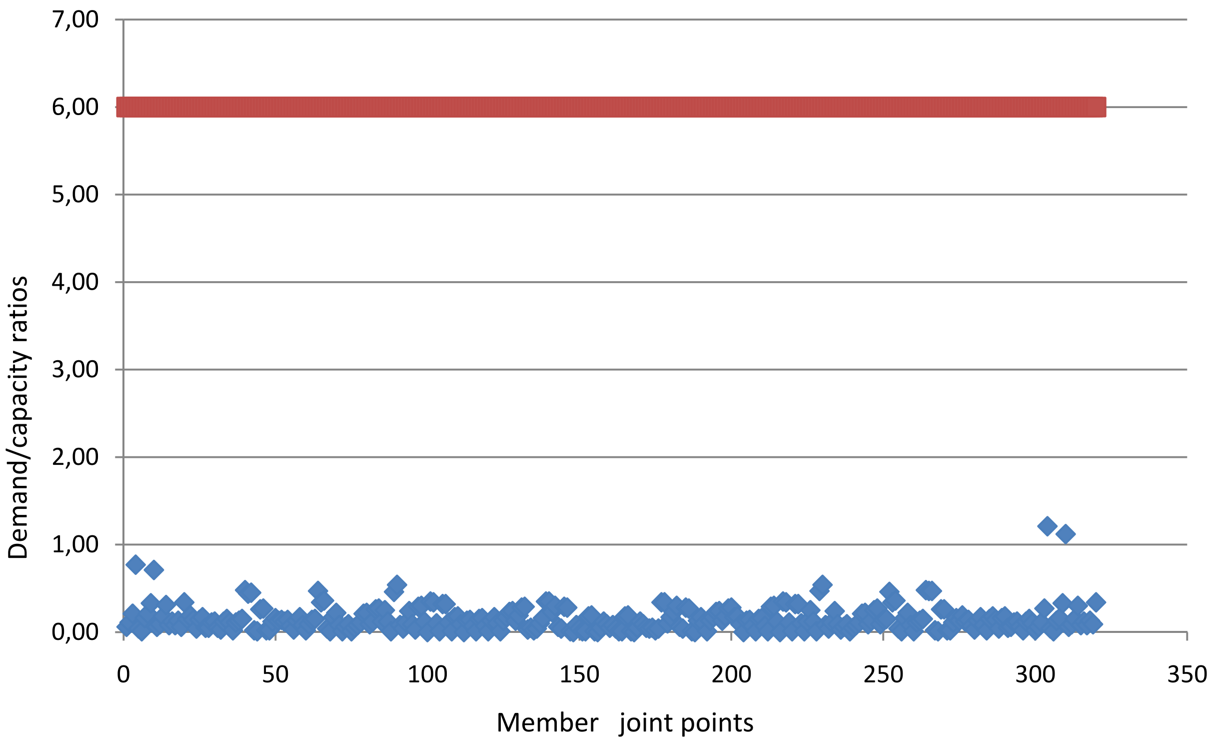

Figure A-71 –EY 2nd Floor Column

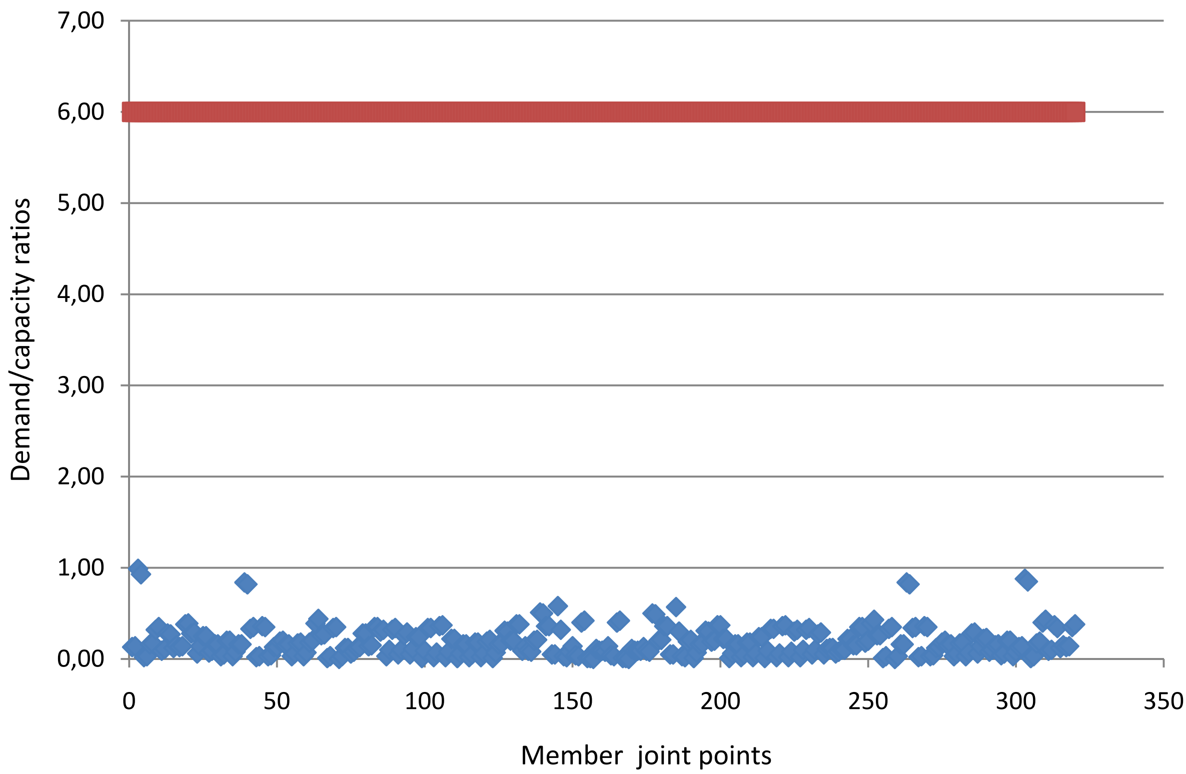

Figure A-72 –EY 1st Floor Column

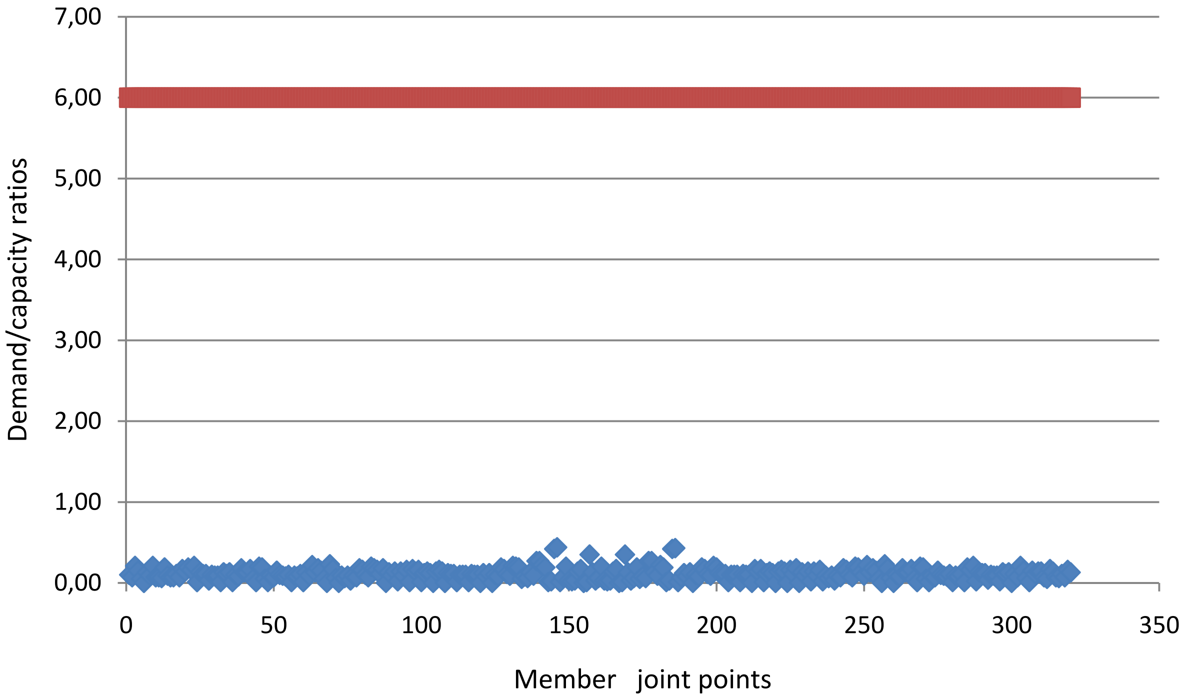

Figure A-73 –EY Ground Floor Column