

İSTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**COMPARATIVE DESIGN OF HIGH RISE RC BUILDING ACCORDING TO
EUROCODE AND ASCE 7-10/ACI 318-11/IBC 2012**

M.Sc. Thesis by

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Department: Civil engineering

Programme: Structural Engineering

JANUARY 2015

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**EUROCODE VE ASCE 7-10 / ACI 318-11 /IBC 2012
YÖNETMELİKLERİNE GÖRE ÇOK KATLI BETONARME BİNALARIN
KARŞILAŞTIRMALI TASARIMI**

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To God , For Your everlasting love and guidance.

FOREWARD

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ABBREVIATIONS

ACI	: The American Concrete Institute
ASCE	: American Society of Civil Engineers
DCH	: High Ductility Class
EC 1	: EN 1991: (Eurocode 1) Actions on structures
EC 2	: EN 1992: (Eurocode 2) Design of concrete structures
EC 8	: EN 1998: (Eurocode 8) Design of structures for earthquake resistance
ELSL	: Equivalent Lateral Static Load
IBC	: International Building Code
MWFRS	: Main Wind Force Resisting Systems
RC	: Reinforced concrete
RSA	: Response Spectrum Analysis
SLS	: Serviceability Limit State
ULS	: Ultimate Limit State

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COMPARATIVE DESIGN OF HIGH RISE RC BUILDING ACCORDING TO EUROCODE AND ASCE 7-10/ACI 318-11/IBC 2012

SUMMARY

In today's world the development of high tech technology, especially in concrete sciences, let engineers to create higher and higher buildings with the variety of architectonic shapes and forms. It is observed especially in tall buildings in the last 20 years. Modern projects surprised us by complicated forms alluding to history, tradition etc. The innovations in skyscraper design not only promote new ideas in construction solutions but as well in reduction of energy consumption, improving comfort of people who live in. Modern equipment of high buildings makes that they can be considered as an intelligent buildings.

Designing of high rise buildings with complex form is possible thanks to the development in the computer technology. Increased computational power of computers allowed to create more advanced engineering programs with better simulations and technics in preparing the structural models. However, the use of new computing solutions from other fields such as mechanics is still limited. It concerns dynamic solutions – the behavior of buildings during earthquakes and hurricanes. This limitation is due to not enough numbers of studies and researches.

The main idea of this thesis was to prepare a computational model of existing or tall building under construction in one of the structural analysis program. According to dimensions and details taken from building project the main structural model was created in ETABS and Autodesk Robot Structural Analysis Software.

It is important to mention that created model was simplification of real building. Curves, arcs in slabs were replaced by perpendicular and diagonal edges. These simplifications don't influence in generally on results obtained, because they concern only geometry. Model was defined as a core wall structure mixed with reinforced concrete frame building in order to maximize the floor – space by consolidating structural support.

Concrete core was used as a base or fundament for carrying a beams connected with outside placed columns. According to regulations used in America (ASCE 7-10, ACI 318-11, IBC 2012) and Europe (Eurocode 1, 2, 8) main working loads, namely dead, load, wind and seismic were defined. Dead load in both cases were assumed to have the same value. Materials and their configuration in structural elements like floor, don't depend on the chosen building regulation. However, live load have been changed according to separate category and prescribed conditions in buildings codes. Detailed calculations of wind load were necessary for the design and construction of more wind resistant building. The speed of the wind acts as a pressure when it meets with a building. The intensity of that pressure helps to describe how wind load acts on the structure. Unfortunately there is no precise code which would deal with its pressure distribution in general. In American Codes wind load does not change over a height, just some coefficients are reduced in direction perpendicular to the direction of wind. Although in Eurocode model is a bit more complex. Windward side is divided into rectangular areas where pressure is defined over reference height of each of the segment. If we just look at the total volume of wind forces and building or simplified segments dimensions of wind pressure coefficient, we can easily compare approximate results of wind actions. Moreover, it should be mentioned that the unpredictability character of loading makes difficulties to obtain accurate calculations. To design a structure against earthquake, method called response spectrum analysis, where maximum responses for a system are estimated, was chosen. The method involves the calculation of only the maximum values of the displacements and member forces in each mode of vibration using smooth design spectra that are the average of several earthquake motions. Because of long computation process all of seismic calculation were done with the help of ETABS. Seismic parameters according to localization, type of the ground and their classification varies from code to code which slightly influence the results. However, the diagram of shear base force after all gave some ideas about range of seismic actions. Based on the results obtained in computational part in ETABS, according to Eurocode and ASCE, structural elements like beam, column and wall were checked and reinforcement was design. The usage of steel reinforcement, placement details were compared and some discrepancies between two ways of analysis were explained as a conclusion. The results of the project were some simple construction drawings included after computational part.

**EUROCODE VE ASCE 7-10 / ACI 318-11 / IBC 2012
YÖNETMELİKLERİNE GÖRE YÜKSEK KATLI BETONARME
BİNALARIN KARŞILAŞTIRMALI TASARIMI**

ÖZET

Bugünün dünyasındaki yüksek teknik teknolojinin gelişimi özellikle somut bilim , mühedislere yüksek binalar inşaa etmesine ,mimari çeşitlilikleri geliştirmesine ve yeni binalarda çeşitli şekiller ve formlar inşaa etmesine imkan tanımaktadır. Bu özellikle son 20 yılda bütün yüksek bina yapımında uygulanmaktadır. Modern projeler eski yapılara nazaran bizi daha fazla şaşırmaktadır. Yüksek binaların tasarımındaki yenilikler sadece yeni fikirlerin oluşumuna yer açmamakta aynı zamanda yaşamak için insanların konforu ve rahatını da geliştirmektedir.

Kompleks formlara sahip yüksek yapı tasarımı bilgisayar teknolojilerindeki gelişmeler sayesinde mümkündür. Yapısal modellerin hazırlanmasında bilgisayarların ileri hesaplama gücü daha iyi simülasyon ve tekniklerle daha ileri mühendislik hizmeti yaratılmasına izin veriyor. Bu durum yapının deprem ve rüzgar anındaki davranışı gibi dinamik çözümleri de kapsıyor.

Bu tez, var olan veya yapım aşamasındaki yüksek binanın farklı yönetmelikler kullanılarak yapısal analiz programlarından biriyle hesaplanması bulunması amacıyla hazırlanmıştır. Bina projesinden alınan boyutlar ve detaylar ETABS ve AUTODESK ROBOT yapısal analiz programlarında işlenerek ana yapının modeli oluşturulmuştur.Oluşturulan modelin, gerçek yapının basitleştirilmiş hali olduğunu belirtmek gerekir.Eğriler ve döşeme yayları, dik ve çapraz kenarlar ile değiştirildi.Bu sadeleştirmeler genel olarak sonucu etkilemez çünkü bu sadeleştirmeler yalnızca geometriyle alakalı değişikliklerdir.

Model, kat planında net alanı arttırmak için yapı taşıyıcıları merkezde çekirdek ile birlikte betonarme çerçeve olarak tanımlandı. Betonarme çekirdek dışarıda konumlanan kolonlara bağlanan kirişleri taşıyan bir taşıyıcı olarak kullanıldı..

Ana yükler sırasıyla ölü yük, hareketli yük,rüzgar yükü ve sismik yük, Amerika'da (ASCE 7-10, ACI 318-11, IBC 2012) ve Avrupa'da (Eurocode 1, 2, 8) kullanılan standartlara göre tanımlandı.

Her iki durumda da ölü yükün aynı değere sahip olduğu varsayılmıştır. Malzemeler ve zemin özellikleri her iki yönetmeliğe göre tasarında aynı kabul edilmiştir. Ancak hareketli yük ayrı bir kategoriye ve bina kodlarındaki öngörülen şartlara bağlı olarak değişmiştir.

Rüzgar yükünün detaylı hesapları binanın rüzgar direncinin hesabında ve dizaynında gereklidir. Rüzgar hızı bina yüzeyiyle karşılaştığında basınç etkisi gösterir. Bu basınç etkisinin yoğunluğu yapıda rüzgar yükünün nasıl davranacağını tanımlamaya yardımcı olur. Fakat, genel anlamda basınç dağılımını kesin olarak gösteren bir standart bulunmamaktadır. Amerikan standartlarında rüzgar yükü, rüzgar yönüne paralel olarak yükseklikle değişmemekte, sadece rüzgar yönüne dik doğrultuda bazı azaltma katsayıları bulunmaktadır. Buna rağmen Eurocode ise bir miktar daha komplekstir. Rüzgar yönü doğrultusundaki bölüm her bir segmentin referans yüksekliğinde tanımlanan dikdörtgen alanlara bölündü. Eğer sadece rüzgar kuvvetinin toplam hacmi ve binanın ya da basitleştirilmiş segmentlerin ölçülerininin rüzgar basıncı katsayısına bakarsak rüzgar hareketinin yaklaşık sonuçlarını kolayca karşılaştırabiliriz. Bunun da ötesinde yüklemenin tahmin edilemeyen özelliklerinin doğru sonuçlar elde etmeyi zorlaştırdığından söz edilebilir.

Depreme dayanıklı yapı tasarımı için, davranış spektrum analizi yöntemi kullanılmıştır. Bu yöntem, sadece yerdeğistirmelerin maksimum değerlerinin hesaplanmasını ve birkaç deprem hareketinin ortalamasını içeren pürüzsüz tasarım spektrumu kullanılarak hazırlanan her bir titreşim modundaki eleman kuvvetleri içerir. Sismik hesaplamalar işlemi uzun zaman aldığı için Etabs yardımıyla tamamlanmıştır. Yerleşime, zemin türüne ve yönetmeliklerde farklılık gösteren sınıflandırmalara göre seçilen sismik parametreler sonuçları etkilemektedir.

Buna rağmen, taban kesme kuvveti diyagramı deprem etkileri aralığı ile ilgili bilgi verir. Eurocode ve ASCE'ye dayanarak yapılan ETABS hesaplama sonuçlarına göre, giriş, kolon ve perde gibi yapı elemanları kontrol edildi ve gerekli donatıların hesabı yapıldı. Donatıların kullanımı, yerleşim detayları karşılaştırıldı ve analizin iki yoluna göre olan farklılıklar sonuca göre açıklandı. Bazı basit çizimlerin sonuçları ek olarak hesaplama kısmından sonra bulunmaktadır.

1. INTRODUCTION

The history of high-rise buildings began over 130 years ago. Place of birth is Chicago, where in the late nineteenth century, the first school of high rise buildings was opened. Undoubted impulse to the development of the construction industry was the invention of elevators in 1853 by Elisha Otis, and then drive outlet. From the very beginning, the skyscrapers were a symbol of prestige investor. This led to start at the beginning of the twentieth century, the pursuit of height records. One of the most interesting origins of this rivalry is built in 1931 in NewYork's Empire State Building.

The development of technology (especially concrete) allowed not only to climb higher and higher, but also to diversify their forms and shapes. This is particularly evident in high-rise buildings of the last twenty years. Contemporary designs stereotypes storm in the form of high-rise glass cuboid and surprise complex forms referring to the history, traditions and even religion. Skyscrapers promote the development of innovative solutions not only in terms of design, but also in the fields of vertical communication, reduce energy consumption, improve comfort in the building of human life or safety of the people living in it.

The ability to design more complex and seemingly impossible to exist structural forms are required nowadays from civil engineers. Almost all of the newest trends in building technology are smart implemented right there. Requirements towards these kind of structures don't include just resisting wind and earthquakes or other types of loads, protect occupants from fire and support their weight but also providing utilities, comfortable climate and conveniently access. The problem in tall building design are considered among the most complex encountered given the balances required between capacity design, economy and utility. The two load source, namely: wind and earthquake load fulfil the most important function during analysis process.

Wind flow is quite complex and turbulent in nature. In designing for wind, building can not be considered independently from its surroundings. Variations of wind speed called turbulence have impact in determining building oscillations.

Wind loads therefore, need to be studied as if they were dynamic in nature. The intensity of loading depends on how fast it varies and response characteristic of structure. A second big role in design plays earthquake analysis which is not easy to conduct in a simple way. The behavior of a building during an earthquake is a vibration problem. Tall buildings are invariably more flexible than low-rise buildings. The magnitude of inertia forces induced during an earthquake depends on the building mass, ground acceleration and the dynamic characteristic of the structure which will be proved later on. However, the methodology of creating model of tall buildings and later the attempt to make a proper analysis represent a big challenge for engineers as a most advanced computational process. The idea of usability and universality of the structure served as the inspiration to write this thesis, which primary objective was to disseminate information on the latest concepts, techniques and software design data to structural engineers.

Preparing preliminary design in this thesis, which focused on making general framework and basis to build the project on, had a huge impact on results obtained. Not all of the conditions had to be satisfied, however results obtained during the analysis should show the general tendency of behavior of the structure and should be on the safe side design. Future studies should put attention on detailed design and optimization, parameters of the part being created can be modified and changed then.

1.1 Scope and Objective of Work

The aim of this work was to built a digital model in one of the structure analysis software and according to international building regulations make static and dynamic analysis of high rise building. The dead, live, wind, snow and earthquake load was taken into consideration. Calculated values of the impact on the structure were put into computer program (ETABS, Robot Structural Analysis 2015). Then, according to results obtained in computational part, the design of some basic structural elements like: beam, column and shear wall (core) were made. Every step of analysis and design was calculated twice and separately. At the beginning according to Eurocode and after all according to American regulations (ASCE 7-11, ACI 318-11, IBC 2012). The usage of steel reinforcement, placement details were compared and some discrepancies between two ways of analysis were explained as an conclusion. The results of the member design are given and some simple construction drawings are included after computational part in chapter 5.

Design high rise building is a part of buildings complex .The total area of land where complex will be placed is 320000 m² with total project area of 1688898 m². It will not be just sleeping area with luxury apartments but also surface to active life and entertainment for the XXI century.



Figure 1.3: Visualisation of high rise building environment.

Representative avenue in the complex will connect stores, special boutiques, dinning facilities and art galleries. Two shopping malls, one at each end of the street, restaurants and cafes located in outdoor area of 5000 m² will provide inhabitatnts great facility to not move outside the complex in need of doing shopping ect. Moreover, residential and office transportation for the entire project was designed in a manner that would not intersect with the shopping and visitor traffic. Objects are going to be completed and ready to move in in september 2015.Two giant squares, one 12000 m² and the other 28000 m² are located in the middle of complex to provide an open space among buildings. There will be also an amphitheater for outdoor shows and concerts, as well as a special stage for kids.

1.2.2 Building

The total height of the construction including roof covering is 154,63 m. The reinforced concrete skeleton of the building is in-filled with cladding panels to form exterior walls. Central concrete cores consist of combination of shear wall, namely legs, which are large enough to contain elevator shafts and other mechanical components. Around the perimeter of the building an array of support columns and shear walls are located and they are connected by horizontal beams to one another and to the core increasing stiffness in both directions. The dimensions of the columns is trimmed down along the height of building. Width and height of the cross section of the column is changed every 15 storey to minimize the disturbance on construction.

- Height of the building : $h = 145\text{ m}$
- Number of floors : 45
- Typical height of one floor : 3,25 m
- Basic seismic-force-resisting system: Shear wall –frame interactive system together with core –supported system
- Structural elements used in model : Rigid diaphragms, RC frames and columns, RC shear walls
- Foundations: Piled raft foundation systems

Dimensions of the typical floor plan:

Width : $b = 43,22\text{ m}$ (the longest dimension)

Height : $l = 24,10\text{ m}$

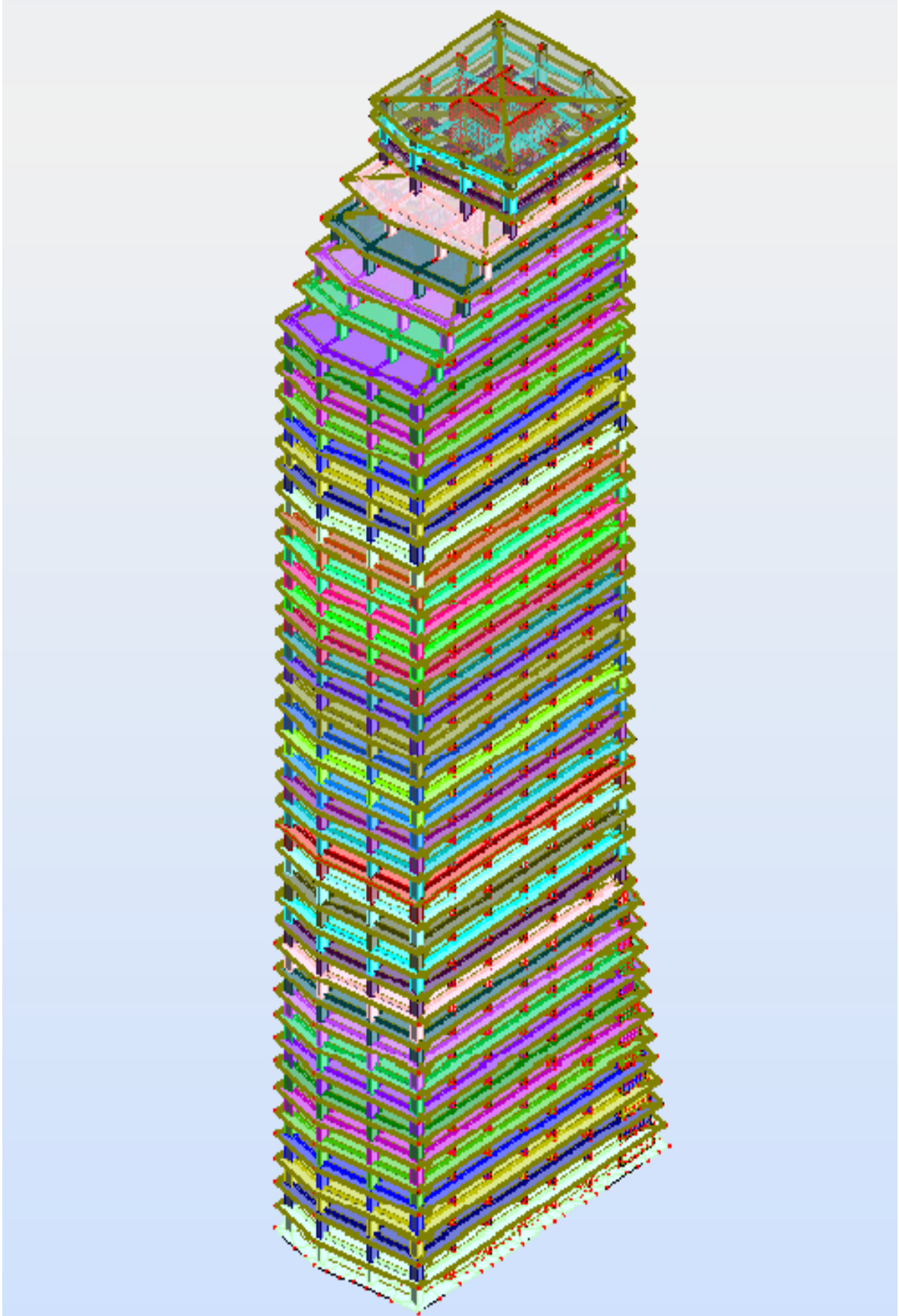


Figure 1.4: Model of a building in RSA 2015.

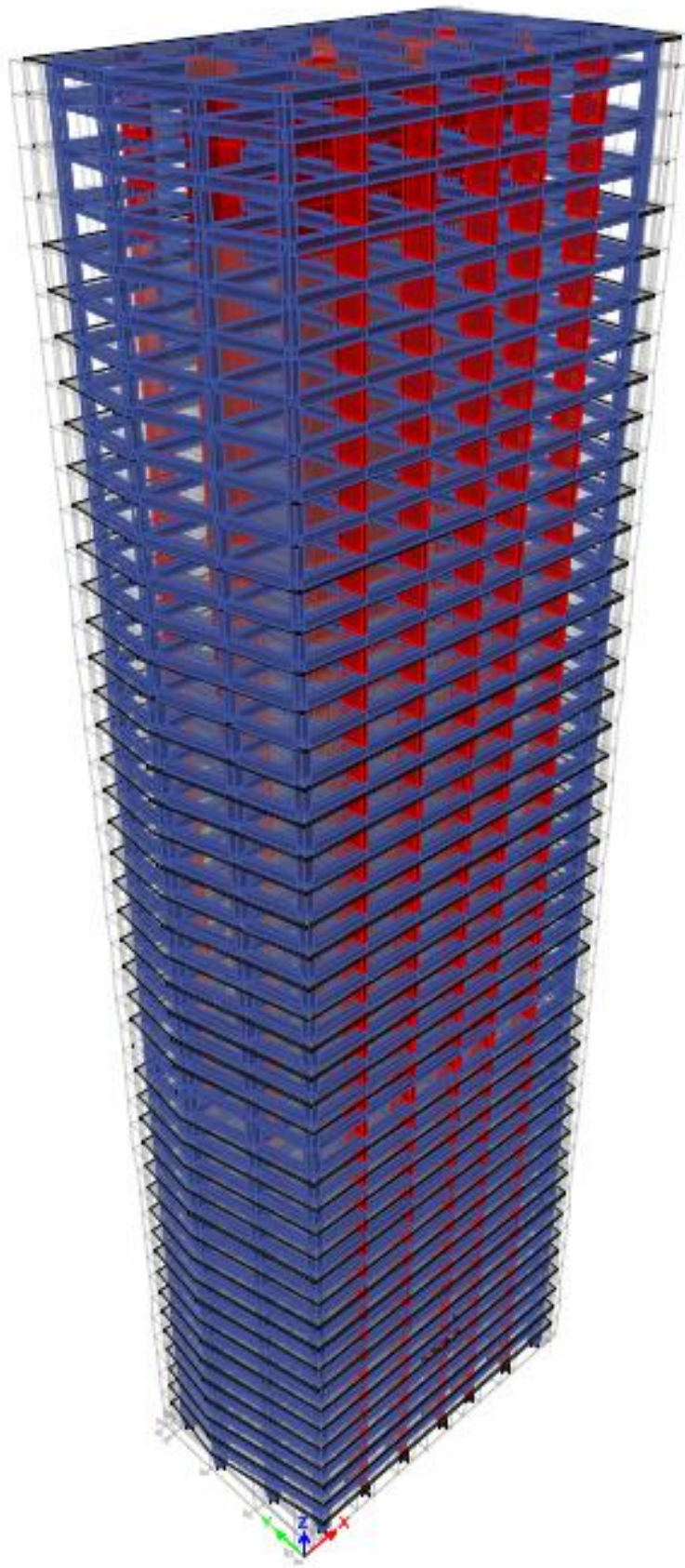


Figure 1.5: Model of a building in ETABS 2015.

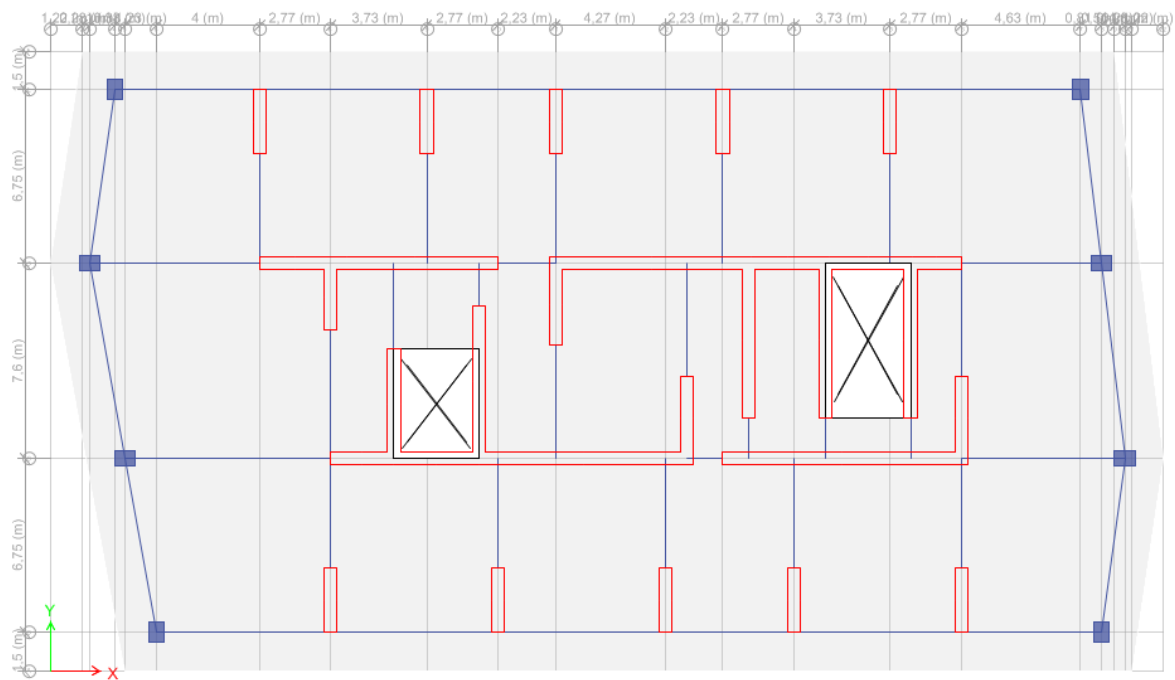
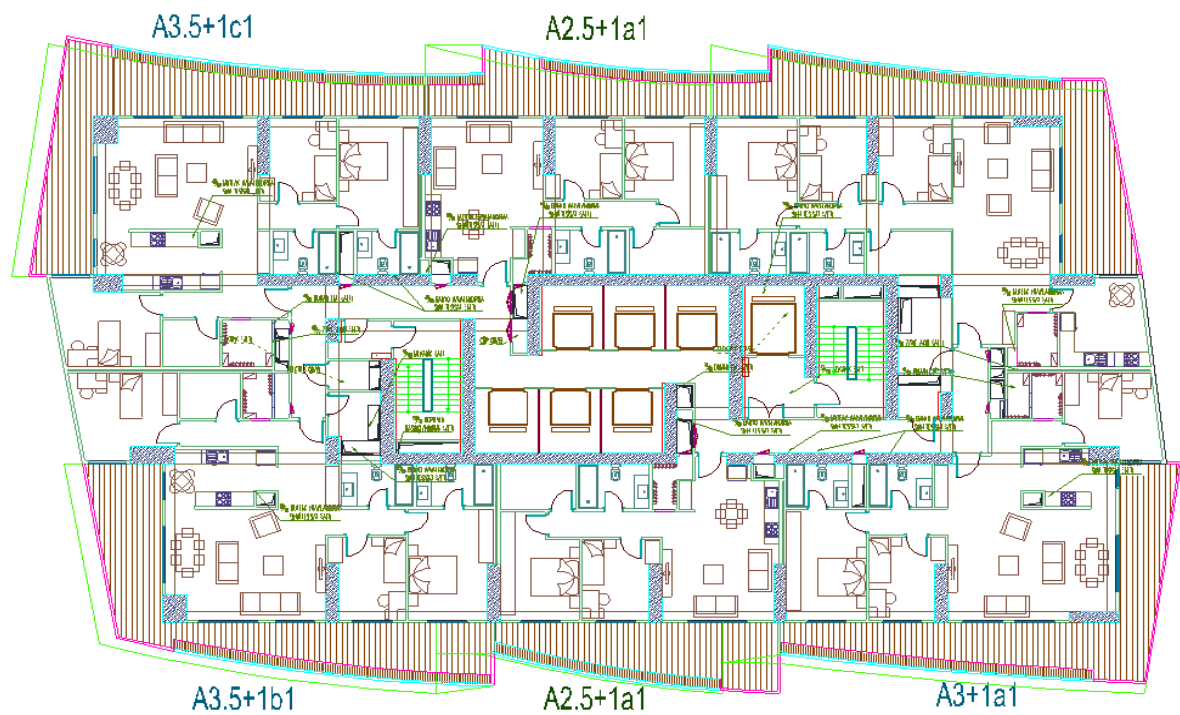


Figure 1.7: Model of typical floor in ETABS.

Basic seismic-force-resisting system definition:

Structure is designed in such a way that its lateral force resistance is provided by frames and shear walls. According to International Code Council, shear wall-frame interactive system in this thesis uses combinations of shear walls and frames designed to resist seismic lateral forces in proportion to their rigidities, considering interaction between shear walls and frames on all levels. This combined system can be said as a dual system. Ductile frames provide a significant amount of energy dissipation when required particularly in upper stories of the building. According to Table 12.2-1 of ASCE 7-10, system can be categorized as dual system with special moment frames capable of resisting at least 25% of prescribed seismic forces, special reinforced concrete shear walls.

Structural elements used in model:

1. Rigid diaphragms

Slabs have been modeled as rigid diaphragms which have infinite in-plane stiffness properties, and therefore they neither exhibit membrane deformation nor report the associated forces. The infinite in-plane stiffness components of a rigid diaphragm allows the stiffness matrix to condense, decreasing computational time. For rigid diaphragms, the accidental eccentricity associated with seismic loading is concentrated and applied at the center of mass. For wind cases and rigid diaphragm, load is applied at geometric centroid. A diaphragm is rigid for the purpose of distribution of story shear and torsional moment when the lateral deformation of the diaphragm is less than or equal to two times the average story drift.

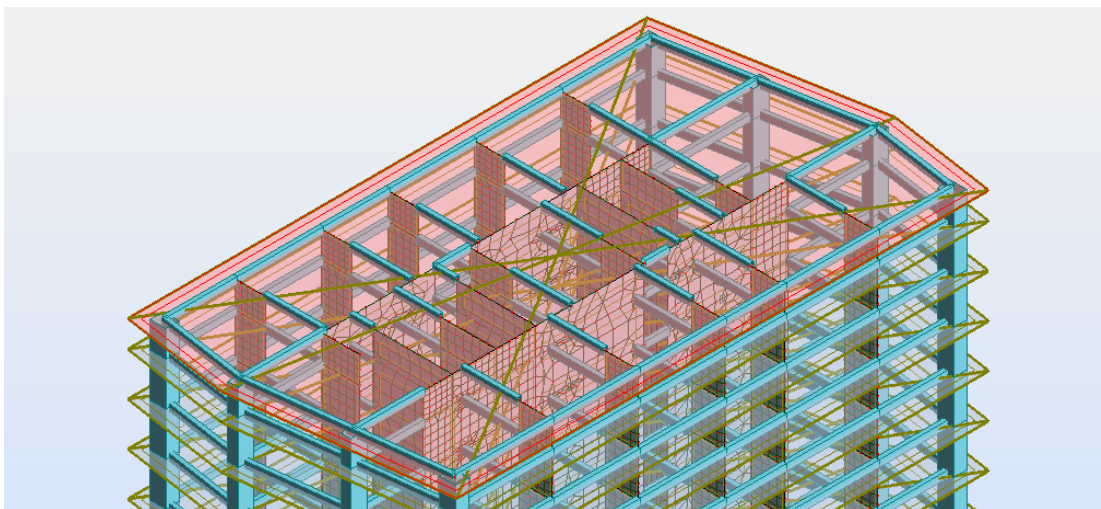


Figure 1.8: Rigid diaphragms.

2. RC moment frame: beams and columns

Moment frames consists of RC beams and columns which resist lateral forces by flexure as well as along the axis of the members.

Columns transmit through compression, the weight of the structure above to other structural elements below. They are used to support beams on which the upper parts of walls or ceilings rest. Width and height of the cross section of the column is changed every 15 storey to minimize the disturbance on construction.

- Storey: 1-15: 1200 x 1000 mm
- Storey 16-30: 1000 x 900 mm
- Storey 31-45: 800 x 600 mm

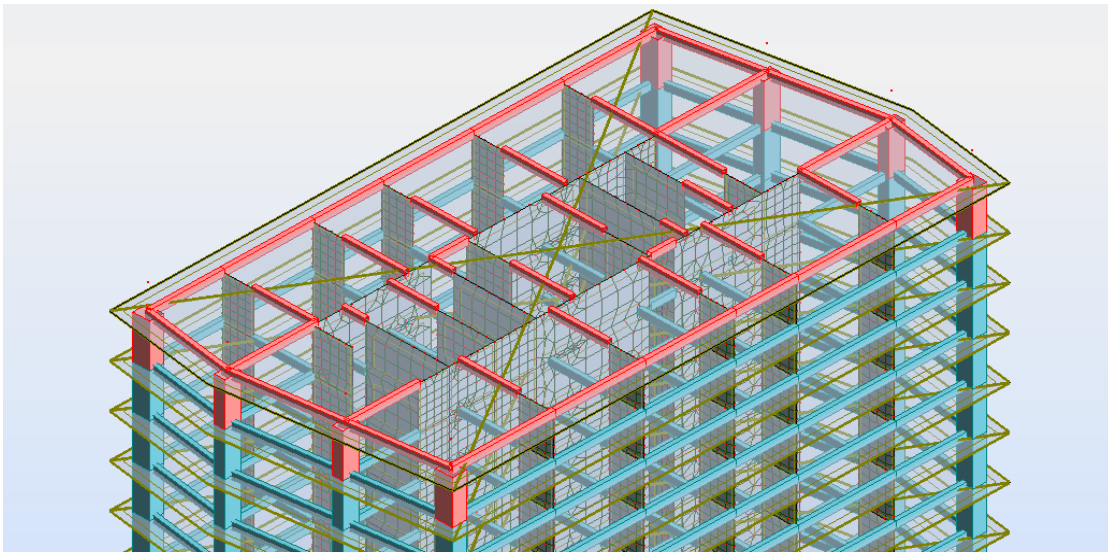


Figure 1.9: Scheme of RC moment frame.

3. Shear walls

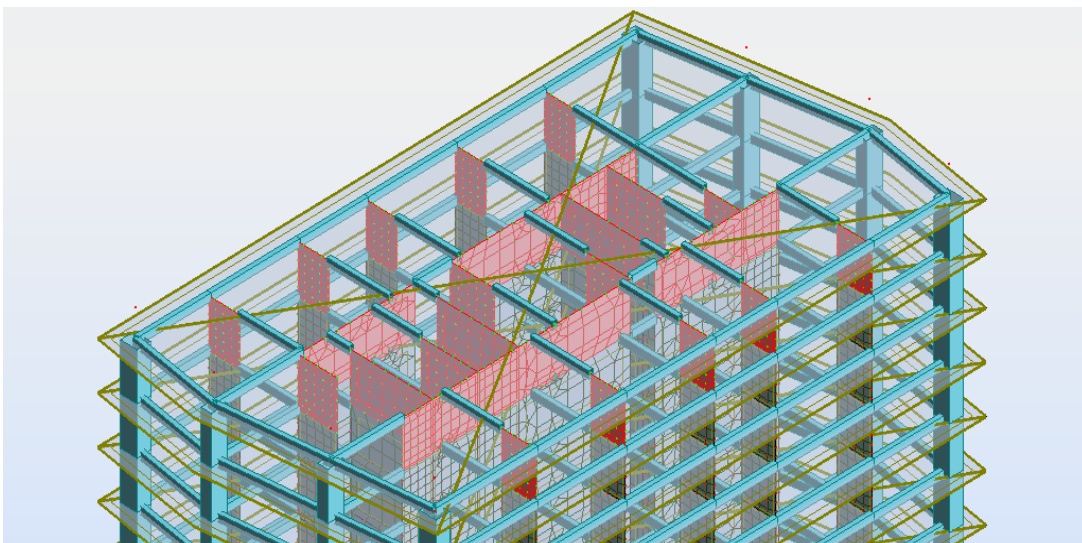


Figure 1.10: Scheme of shear walls .

Shear walls counter the effects of lateral load acting on a structure. Shear walls provide large strength and stiffness to building in the direction of their orientation (reduce lateral sway). Shear walls are more effective when located along exterior perimeter of the building, they should be symmetrical in plan: reduce twist. A structure of shear walls in the center of a building — encasing an elevator shaft form a shear core.

1.3 Structural Materials

- Concrete C40/50

Specified concrete compression strength: $f'_c = 40 \text{ MPa}$

Weight per unit volume: $25 \frac{\text{kN}}{\text{m}^3}$

Modulus of elasticity: $E = 35000 \text{ MPa}$

Poisson's ratio: $\nu = 0,2$

Shear modulus: $G = 14580 \text{ MPa}$

- Steel A615Gr60

Minimum yield strength: $f_y = 413,7 \text{ MPa}$

Minimum tensile strength: $f_u = 620,5 \text{ MPa}$

Expected yield strength: $f_{ye} = 455,1 \text{ MPa}$

Expected tensile strength: $f_{ue} = 682,6 \text{ MPa}$

Weight per unit volume: $76,97 \frac{\text{kN}}{\text{m}^3}$

Modulus of elasticity: $E = 199948 \text{ MPa}$

Materials used should meet the design requirements set out by American and European construction standards.

Products must also have the certificate of quality or results of laboratory tests confirming the required quality.

1.4 Theoretical basis of calculations

The whole structure is divided into finite elements. Elements are connected only at the nodes. The deformation within each element is defined at the based on nodal displacements (element functions shape). In this way, the internal energy structure depends only on independent nodal parameters. Nodal displacements recorded for the whole structure create a global vector of unknown structure displacements q . The general pattern of the system of equations equilibrium is:

$$Mq'' + Cq' + Kq = f(t) - f(t, q) \quad (1.1)$$

K - stiffness matrix which is the sum of the following components of the matrix:

$$K = K_0 + K_\sigma + K_{NL} \quad (1.2)$$

K_0 - initial stiffness matrix (independent of the vector q)

K_σ - stress matrix (linearly dependent compressive stress)

K_{NL} - matrix dependent on other components of the vector q

C - damping matrix

q - the displacement vector (or total displacement increments)

q' - vector velocity (first derivative of the displacement vector q time)

q'' - vector acceleration (second derivative of displacement vector q time)

$f(t)$ - vector of external forces

$f(t, q)$ - unbalanced force vector.

These equations are written for the whole structure using the global displacements q .

This means that the displacements are defined in the global coordinate system.

The transformation of the local coordinate system to global coordinate system (and vice versa) is a standard operation on matrices.

Static analysis:

The basic equations of equilibrium (1.1) can be simplified when additional assumption will be made that the load applied is quasi static.

This means that loads are applied to the structure are so slow that speed and acceleration of the masses of the construction are equal to zero, and the forces of inertia and damping and kinetic energy and damping can be omitted. Simplified in this way, the system of equations shows a static system of equations with many degrees of freedom of the structure. There are two types of static structural analysis: linear analysis and nonlinear analysis. In this thesis linear analysis will be conducted.

Linear analysis:

Linear statics analysis is the basic type of construction in the program. During static structural analysis the following assumptions are made : small displacements and design rotations and perfectly elastic material. This implies that the principle of superposition can be applied. Elements of the stiffness matrix for this case are constants. Equilibrium equation takes the form:

$$K_0 q = f \quad (1.3)$$

To solve the problems of static linear displacement method is used. Results of static calculations include:

- Nodal displacement
- Internal forces in elements
- The reactions at the nodes of the support
- Residual forces at the nodes.

Dynamic analysis:

Computer programs can perform various types of dynamic calculations. For dynamic analysis majority of engineering software programs have adopted the same assumptions as for the linear analysis.

Weights used in the calculation of the dynamic structure may be defined on the basis on:

- self-weight design
- self-weight construction and gathered masses added
- weights derived from the forces

The floors of the building are treated as infinitely rigid in its plane, and completely flaccid on their plane. Adoption of the discrete model is justified by the fact that more than half of the mass of the building is focused on the level of floors.

For multi-storey buildings with stiffening walls , it is more convenient and easier to determine the susceptibility matrix D , and then the stiffness matrix K , then the equation (1.1) takes the form:

$$DMq'' + DCq' + q = DF \quad (1.4)$$

$$D = K^{-1} \quad (1.5)$$

During the modal analysis of the structures all the basic form of vibrations are determined.

Vibration equation of motion is derived from the equation (1.4) excluding external loads and damping matrix.

$$DMq'' + q = 0 \quad (1.6)$$

Solutions of the equation are written in the form of harmonic functions.

$$q(t) = asin(\omega t + \phi) \quad (1.7)$$

a- Vector of free vibration amplitude

After substituting (1.7) to (1.6), it is obtained a system of linear algebraic equations.

$$(DM - \omega^{-2}I)a = 0 \quad (1.8)$$

Which is non-zero solution to the condition that:

$$\det(DM - \omega^{-2}I)a = 0 \quad (1.9)$$

Equation (1.8) is called the characteristic equation with eigenvalue problem.

The roots of this equation are the frequencies of free vibrations.

The ordered set of natural frequencies in ascending order, creates a frequency vector of free vibrations. After substituting the frequency values to the equation (1.10) it is obtained mode shapes vectors. Vectors of free vibrations satisfy orthogonality conditions.

Seismic analysis

A seismic analysis must take adequate account of dynamic amplification of ground motions due to resonance. The normal way of doing this is by using a response spectrum. Methods based on linear analysis are widely used. Non linearity is considered by using a ductility –modified response spectrum.

It should be underlined that this method leads just to the maximum response, instead of fully describing the response.

Response spectrum analysis is often considered to be the most attractive method for the seismic design of a given structural system.

The equation of motion system with many degrees of freedom subjected to kinematic ground forces takes the form:

$$Mq'' + Cq' + Kq = p_{ef}(t) \quad (1.10)$$

$p_{ef}(t)$ – vector of the effective seismic force

$$p_{ef}(t) = -Mr\ddot{u}(t) \quad (1.11)$$

r –influence vector

Assuming that the eigenvalue problem is already solved, to solve equation 1.11 the method of self-transformation of matrix is used.

Displacement vector $q(t)$ is presented then in form:

$$q(t) = \sum_{i=1}^n a_i y_i(t) = Ay \quad (1.12)$$

A – vector form of undamped free vibrations

y_i – normal coordinate

Substituting equation 1.12 to 1.11 and multiplying both sides by a_i^T following equation is obtained:

$$a_i^T M A \ddot{y} + a_i^T C A \dot{y} + a_i^T K A y = a_i^T p_{ef}(t) \quad (1.13)$$

$$a_i^T M A a_j = 0 \quad a_i^T K A a_j = 0 \quad (1.14)$$

Orthogonal conditions (1.14) make that only one component is different from zero in the matrix products containing matrices M and K when $i = j$. A similar reduction can be applied for component of the damping matrix C, on the assumption that the matrix is linear attenuation of a combination of mass and stiffness matrices.

$$C = \mu M + \kappa K \quad (1.15)$$

μ, κ - proportion coefficients

Then,

$$a_i^T C a_j = 0 \quad for \ i \neq j \quad (1.16)$$

With the help of 1.16 we are able to write equation 1.13 in the form :

$$m_i \ddot{y} + c_i \dot{y} + k_i y = p_i(t) \quad for \ i = 1, 2 \dots n \quad (1.17)$$

$$m_i = a_i^T M a_i \quad (1.18)$$

$$c_i = a_i^T C a_i \quad (1.19)$$

$$k_i = a_i^T K a_i = \omega_i^2 m_i \quad (1.20)$$

$$p_i(t) = a_i^T p_{ef}(t) \quad (1.21)$$

After dividing both sides of equation (1.17) by m_i , it is obtained:

$$\ddot{y} + \frac{c_i}{m_i} \dot{y} + \frac{k_i}{m_i} y = \frac{1}{m_i} p_i(t) \quad (1.22)$$

Using equation 1.20:

$$m_i = a_i^T M a_i \quad (1.23)$$

$$\xi = \frac{\mu}{2\omega_i} + \frac{\kappa\omega_i}{2} \quad (1.24)$$

Differential equations are equations systems with one degree of freedom with reference to the unit of mass, which the solution can be written using the Duhamel integral:

$$y_i(t) = \frac{a_i^T B r}{a_i^T B a_i} \cdot \frac{1}{\omega_{id}} \int_0^t u(\tau) e^{-i\xi\omega_i(t-\tau)} \sin|\omega_{id}(t-\tau)| d\tau \quad (1.25)$$

$$\omega_{id} = \omega_i \sqrt{1 - \xi_i^2} \quad (1.26)$$

The overall reaction is expressed by the formula:

$$q(t) = \sum_{i=1}^n a_i y_i = \sum_{i=1}^n a_i \eta_i \frac{1}{\omega_{id}} \int_0^t u(\tau) e^{-i\xi\omega_i(t-\tau)} \sin|\omega_{id}(t-\tau)| d\tau \quad (1.27)$$

$$\eta_i = \frac{a_i^T B r}{a_i^T B a_i} \quad (1.28)$$

η_i – participation rate of mode shapes.

It can be said that the share of the i-th form of vibration in response jth generalized coordinates q_j is:

$$q_{ij} = A_{ji}\eta_i \frac{1}{\omega_{id}} \int_0^t u(\tau) e^{-i\xi\omega_i(t-\tau)} \sin|\omega_{id}(t-\tau)| d\tau \quad (1.29)$$

A_{ji} is the j-th component of the i-th eigenvector.

The maximum value of the Duhamel integral in the above equations, is a value corresponding to i-th value of response spectrum frequency $S_d(\xi_i, \omega_i)$.

$$\max q_{ij} = |A_{ji}\eta_i S_d(\xi_i, \omega_i)| = \left| A_{ji}\eta_i \frac{1}{\omega_i} S_v(\xi_i, \omega_i) \right| \quad (1.30)$$

To estimate the total response of structure is not enough to apply the sum as in equation (1.29), because it can not be assumed that the maxima of the vibration response of individual characters occur at the same time. Therefore, for the determination, various methods like SRSS or CQC are used.

All things considered, for the dynamic model of the building with a diagonal mass matrix, the maximum seismic force acting on the j-th element of the structure of mass m_j , with vibration , can be estimated on their eigenvalue. Then, using the SRSS or CQC method it can be determined the overall strength of the seismic load on the j-th element.

In addition to the results of seismic analysis modal analysis , there are the following additional dynamic parameters for each form of vibration:

- contribution coefficients of seismic analysis
- the value of the spectrum of the seismic force
- modal coefficients
- displacements, internal forces, reactions and combinations of vibration.

2. DESIGN ACCORDING TO EUROCODE

Eurocodes are a set of harmonized technical rules developed by the European Committee for Standardisation for the structural design of construction works in the European Union.

Eurocodes are mandatory for the specification of European public works and are intended to become the standard for the private sector. Additionally, each country is expected to issue a National Annex to the Eurocodes which will need referencing for a particular country.

2.1 Dead Loads

Table 2.1: Floor loading.

Type of loading	Characteristic value [kN/m ²]	Loading coefficient γ	Design value [kN/m ²]
PERMANENT LOADS			
Carpeted Floor 0,07 kN/m ²	0,07	1,35	0,0945
Alignment layer of cement mortar 3 cm - 23 kN/m ³ · 0,03	0,69	1,35	0,9315
Insulating film 0,05 kN/m ²	0,05	1,35	0,0675
Mineral wool - Stoprock 5 cm 1,61 kN/m ³ · 0,05	0,08	1,35	0,108
Reinforced concrete slab 18 cm 25 kN/m ³ · 0,018 m	4,5	1,35	6,075
Suspended ceiling 0,01 kN/m ²	0,01	1,35	0,0135
	5,4	1,35	7,29

Table 2.2: Edge loading.

Type of loading	Characteristic value [kN/m ²]	Loading coefficient γ	Design value [kN/m ²]
DEAD LOAD OF CURTAIN WALL			
Glazing 19 mm 24 kN/m ³ · 0,019	0,456	1,35	0,62
Supporting structure 0,3 kN/m ²	0,3	1,35	0,41
	0,756	1,35	1,02

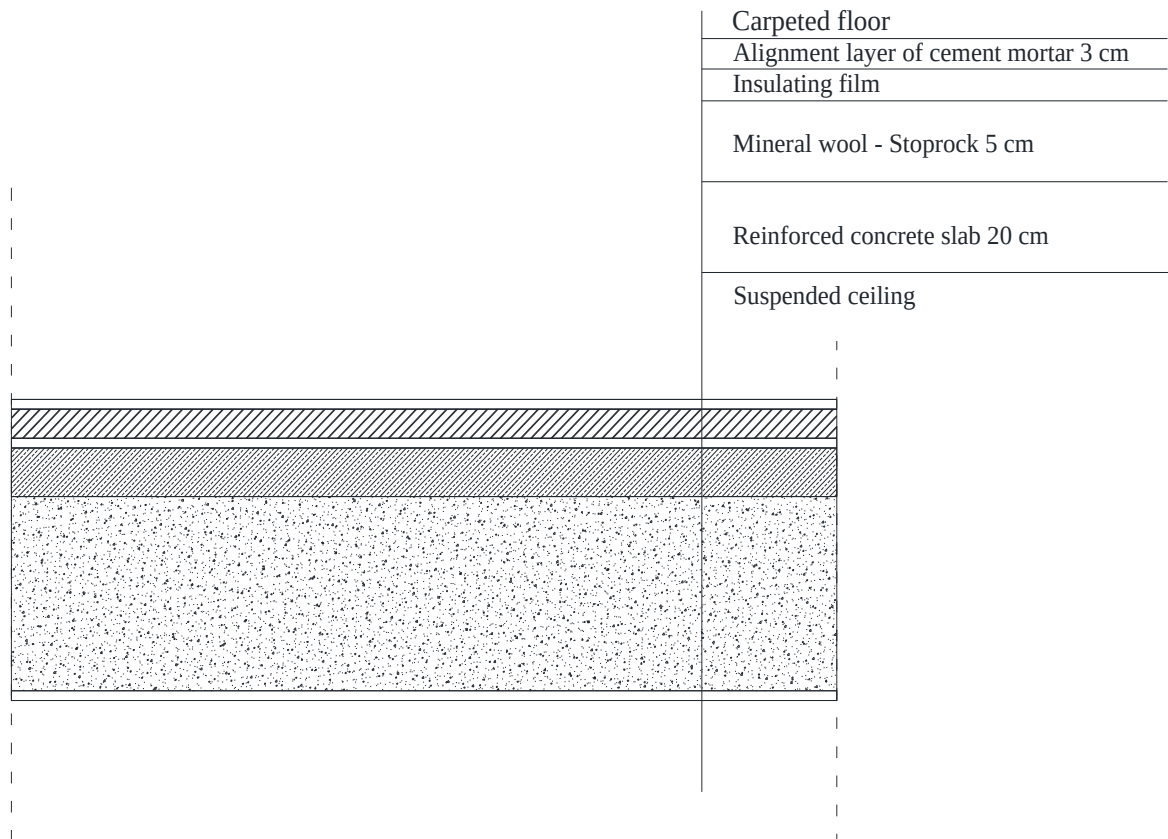


Figure 2.1: Cross-section through the ceiling.

2.2 Live Load

Table 2.3: Live load.

Type of loading	Characteristic value [kN/m ²]	Loading coefficient γ	Design value [kN/m ²]
LIVE LOADS			
Floor area , category A 2kN/m ²	2	1,5	3
Partitions loading 0,8 kN/m ²	0,8	1,5	1,2
	2,8	1,5	4,2

2.3 Roof Loading

Table 2.4: Roof loading.

Type of loading	Characteristic value [kN/m ²]	Loading coefficient γ	Design value [kN/m ²]
DEAD LOADS			
Folded sheet 0,4 kN/m ³	0,4	1,35	0,54
Supporting structure for folded sheet 1,5 kN/m ³	1,5	1,35	2,025
Mineral wool - Stoprock 5 cm 1,61 kN/m ³ ·0,05	0,08	1,35	0,108
	1,98	1,35	2,67
LIVE LOADING			
Roof area , category H (without access) 1 kN/m ²	1	1,5	1,5
	1	1,5	1,5

2.4 Wind Load

(Eurocode 1 : Actions on structures — General actions — Part 1-4: Wind actions)

According to Eurocode classification, wind actions are classified as variable, fixed, direct actions.

For the sake of simplicity it was assumed that the building is rectangular, and dimensions were taken from regular floor in the middle part of structure (Floor 13-39)

Height of the building : $h = 145 \text{ m}$

Dimensions of the typical floor plan:

Width : $b = 43,22 \text{ m}$

Height : $l = 24,10 \text{ m}$

- Terrain category: (EN 1991-1-4, point 4.3.2)

It is assumed that the area where building is built belongs to category IV: Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m

Minimum height: $z_{\min} = 10 \text{ m}$

The roughness length: $z_0 = 1 \text{ m}$

$z_{\max}$ has to be taken as : $z_{\max} = 200 \text{ m}$ (acc. Table 4.1, EN 1991-1-4)

- The reference height: (EN 1991-1-4, point 6.3)

$$z_s = 0,6 \cdot h = 0,6 \cdot 145 = 87 \text{ m} \quad (2.1)$$

- The directional factor: (EN 1991-1-4, point 4.2)

The value of the directional factor, c_{dir} , for various wind directions may be found in the National Annex. The recommended value is 1,0.

- The season factor: (EN 1991-1-4, point 4.2)

The value of the season factor, c_{season} may be given in the National Annex.

The recommended value is 1,0

- The fundamental value of the basic wind velocity: (EN 1991-1-4, point 4.2)

The fundamental value of the basic wind velocity, $V_{b,0}$ is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights.

According to TS498 (Turkish Standard 498)

$$V_{b,0} = 42 \text{ m/s}$$

The fundamental value of the basic wind velocity was taken for the reference height.

The basic wind velocity (EN 1991-1-4, point 4.2- Equation 4.1)

$$\begin{aligned} V_b &= c_{dir} \cdot c_{season} \cdot V_{b,0} \\ V_b &= 1 \cdot 1 \cdot 42 = 42 \text{ m/s} \end{aligned} \quad (2.2)$$

- Terrain factor depending on the roughness length z_0 :
(EN 1991-1-4, point 4.3- Equation 4.5)

$$k_r = 0,19 \cdot \left(\frac{z_0}{z_{0,II}} \right)^{0,07} \quad (2.3)$$

where: $z_{0,II} = 0,05 \text{ m}$ (Terrain category II, Table 4.1)

$$k_r = 0,19 \cdot \left(\frac{1}{0,05} \right)^{0,07} = 0,23$$

- The roughness factor (EN 1991-1-4, point 4.3-Equation 4.4)
accounts for the variability of the mean wind velocity at the site of the structure

$$c_r(z_s) = k_r \cdot \ln \left(\frac{z_s}{z_0} \right) = 0,23 \cdot \ln \left(\frac{87}{1} \right) = 1,03 \quad (2.4)$$

- Terrain orography factor (EN 1991-1-4, point 4.3)

Orography (e.g. hills, cliffs etc.) does not increase wind velocities.

The influence of the orography is expressed in definition of fundamental base wind velocity

$$c_0(z_s) = 1$$

- The mean wind velocity (EN 1991-1-4, point 4.3—Equation 4.3)

The mean wind velocity $V_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity.

$$V_m(z_s) = c_r(z_s) \cdot c_0(z_s) \cdot V_b \quad (2.5)$$

$$V_m(87) = 1,03 \cdot 1 \cdot 42 = 43,26 \text{ m/s}$$

- The turbulence intensity (EN 1991-1-4, point 4.4 —Equation 4.7)

The turbulence intensity $I_v(z)$ at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity.

$$\text{dla } z_{min} = 10 \text{ m} \leq z_s = 87 \text{ m} \leq z_{max} = 200 \text{ m}$$

$$I_v(z_s) = \frac{k_1}{c_0(z_s) \cdot \ln\left(\frac{z_s}{z_0}\right)} \quad (2.6)$$

where: k_1 - the turbulence factor. The recommended value is 1,0

$$I_v(87) = \frac{1}{1 \cdot \ln\left(\frac{87}{1}\right)} = 0,22$$

- The basic velocity pressure: (EN 1991-1-4, point 4.5—Equation 4.10)

$$q_b(z_s) = \frac{1}{2} \cdot \rho \cdot V_m^2 \quad (2.7)$$

where : ρ — is the air density, which depends on the altitude, temperature and barometr pressure to be expected in the region during wind storms

$$\rho = 1,25 \text{ kg/m}^3$$

$$q_b(87) = \frac{1}{2} \cdot 1,25 \cdot 43,26^2 \cdot 0,001 = 1,17 \frac{\text{kN}}{\text{m}^2}$$

- The peak velocity pressure: (EN 1991-1-4, point 4.5-Equation 4.8)

The peak velocity pressure $q_p(z_s)$ at height z , which includes mean and short-term velocity fluctuations, should be determined from formula:

$$q_p(z_s) = (1 + 7 \cdot I_v(z_s)) \cdot q_b \quad (2.8)$$

$$q_p(87) = (1 + 7 \cdot 0,22) \cdot 1,17 \cdot 0,001 = 2,97 \text{ kN/m}^2$$

- The fundamental flexural frequency:

The fundamental flexural frequency n_1 of multi-storey buildings with a height larger than 50 m can be estimated using Expression (F.2):

$$n_1 = \frac{46}{h} [\text{Hz}] \quad (2.9)$$

where: h is the height of the structure in m

$$\begin{aligned} n_1 &= \frac{46}{145} = 0,32 \text{ Hz} \\ T_1 &= \frac{1}{0,59} = 3,15 \end{aligned} \quad (2.10)$$

2.4.1 Structural Factor

This factor takes into account the effect of the wind due to the non-simultaneous occurrence of the peak on the surface of the structure c_s , together with the effect of vibration construction caused by turbulent wind interaction c_d .

$$c_s c_d = \frac{1 + 2 \cdot k_p \cdot I_v(z_s) \cdot \sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z_s)} \quad (2.11)$$

k_p - is the peak factor defined as the ratio of the maximum value of the fluctuating part of the response to its standard deviation

B^2 - is the background factor, allowing for the lack of full correlation of the pressure on the structure surface

R^2 - is the resonance response factor, allowing for turbulence in resonance with the vibration mode

- The peak factor: (EN 1991-1-4; Annex B.2-Equation B.4)

$$k_p = \sqrt{2 \cdot \ln(\nu \cdot T)} + \frac{0,6}{\sqrt{2 \cdot \ln(\nu \cdot T)}} \quad (2.12)$$

where: T - is the averaging time for the mean wind velocity

ν – is the up-crossing frequency

$$T = 600 \text{ s (EN 1991-1-4; Annex B.2)}$$

The up-crossing frequency: (Equation B.5)

$$\nu = \sqrt{\frac{R^2}{B^2 + R^2}} \quad (2.13)$$

$$\nu = \sqrt{\frac{0,45^2}{0,51^2 + 0,45^2}} = 0,66 \text{ Hz}$$

- The turbulent length scale: (Equation B.1)

$$L(z_s) = L_t \cdot \left(\frac{z_s}{z_t}\right)^\alpha \quad (2.14)$$

$$L(87) = 300 \cdot \left(\frac{87}{200}\right)^{0,67} = 171,75$$

where: z_t – reference height

L_t – reference length scale

α – empirical factor

$$L_t = 300 \text{ m}$$

$$z_t = 200 \text{ m}$$

$$\alpha = 0,67 + 0,05 \ln(z_0) \quad (2.15)$$

$$\alpha = 0,67 + 0,05 \ln(1) = 0,67$$

- The non-dimensional frequency: (Equation B.2)

$$f_L(z_s, n) = \frac{n \cdot L(z_s)}{V_m(z_s)} \quad (2.16)$$

$$f_L(87; 0,32) = \frac{0,32 \cdot 171,75}{43,26} = 1,27$$

- The wind distribution over frequencies : (Equation B.2)
is expressed by the non-dimensional power spectral density function $S_L(z_s, n)$, equal to the variance impulses of wind.

$$S_L(z_s, n) = \frac{6,8 \cdot f_L(z_s, n)}{(1 + 10,2 \cdot f_L(z_s, n))^{\frac{5}{3}}} \quad (2.17)$$

$$S_L(87; 0,32) = \frac{6,8 \cdot 1,27}{(1 + 10,2 \cdot 1,27)^{\frac{5}{3}}} = 0,11$$

- The background factor: (Equation B.3)

Takes into account the lack of full correlation of the pressure on the structure surface.

$$B^2 = \frac{1}{1 + 0,9 \left(\frac{b+h}{L(z_s)} \right)^{0,63}} \quad (2.18)$$

$$B^2 = \frac{1}{1 + 0,9 \left(\frac{43,22 + 145}{171,75} \right)^{0,63}} = 0,51$$

According to EN 1991-1-4; Annex B.2 It is on the safe side to use $B^2 = 1$

- The aerodynamic admittance functions : (Equation B.7 ; B.8)

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2} (1 - e^{-2\cdot\eta_h}) \quad (2.19)$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2\eta_b^2} (1 - e^{-2\cdot\eta_b}) \quad (2.20)$$

$$\eta_h = \frac{4,6 \cdot h}{L(z_s)} \cdot f_L(z_s, n) \quad (2.21)$$

$$\eta_h = \frac{4,6 \cdot 145}{171,75} \cdot 1,27 = 4,93$$

$$\eta_b = \frac{4,6 \cdot b}{L(z_s)} \cdot f_L(z_s, n) \quad (2.22)$$

$$\eta_b = \frac{4,6 \cdot 43,22}{171,75} \cdot 1,27 = 1,47$$

$$R_h = \frac{1}{4,93} - \frac{1}{2 \cdot 4,93^2} (1 - e^{-2\cdot 4,93}) = 0,18$$

$$R_b = \frac{1}{1,47} - \frac{1}{2 \cdot 1,47^2} (1 - e^{-2\cdot 1,47}) = 0,46$$

- The resonance response factor R^2 (Equation B.6) :

Allowing for turbulence in resonance with the considered vibration mode of the structure

$$R^2 = \frac{\pi^2}{2 \cdot \delta} \cdot S_L(z_s, n) \cdot R_h(\eta_h) \cdot R_b(\eta_b) \quad (2.23)$$

where: δ - is the total logarithmic decrement of damping

The value of the logarithmic decrement of damping was taken as for monolithic concrete buildings $\delta = 0,1$

$$R^2 = \frac{\pi^2}{2 \cdot 0,1} \cdot 0,11 \cdot 0,18 \cdot 0,46 = 0,45$$

Having calculated the response factor of the resonance, it can be proceed to calculate the up-crossing frequency.

- The peak factor : (EN 1991-1-4; Annex B.2, Equation B.4)

$$k_p = \sqrt{2 \cdot \ln(v \cdot T)} + \frac{0,6}{\sqrt{2 \cdot \ln(v \cdot T)}} \quad (2.24)$$

where: T- is the averaging time for the mean wind velocity

v – is the up-crossing frequency

$T = 600 \text{ s}$ (EN 1991-1-4; Annex B.2)

$$k_p = \sqrt{2 \cdot \ln(0,66 \cdot 600)} + \frac{0,6}{\sqrt{2 \cdot \ln(0,66 \cdot 600)}} = 3,63$$

Hence the structural factor is equal:

$$c_s c_d = \frac{1 + 2 \cdot k_p \cdot I_v(z_s) \cdot \sqrt{B^2 + R^2}}{1 + 7 \cdot I_v(z_s)}$$

$$c_s c_d = \frac{1 + 2 \cdot 3,63 \cdot 0,22 \cdot \sqrt{0,51^2 + 0,45}}{1 + 7 \cdot 0,22} = 0,92$$

2.4.2 Aerodynamic Coefficients

The effect of the wind on the external surfaces of buildings is described by external pressure coefficients. Internal pressure coefficients give the effect of the wind on the internal surfaces of buildings.

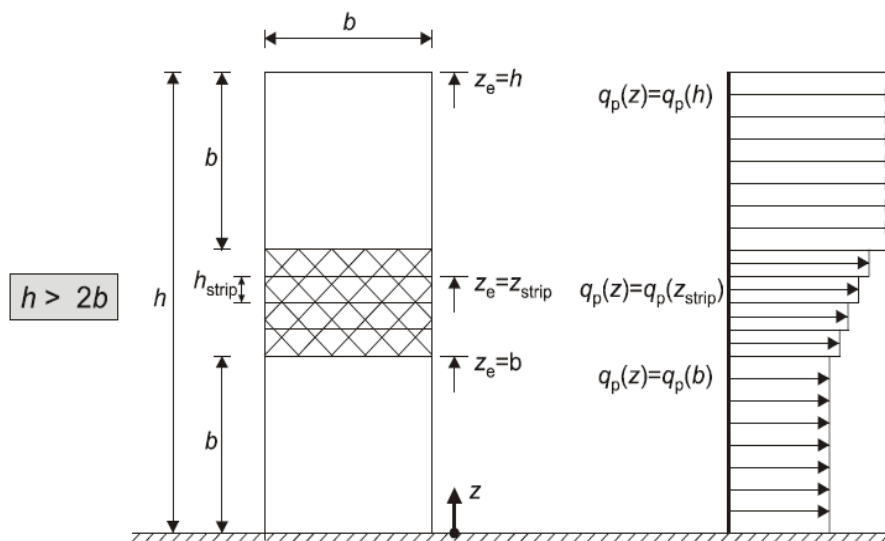
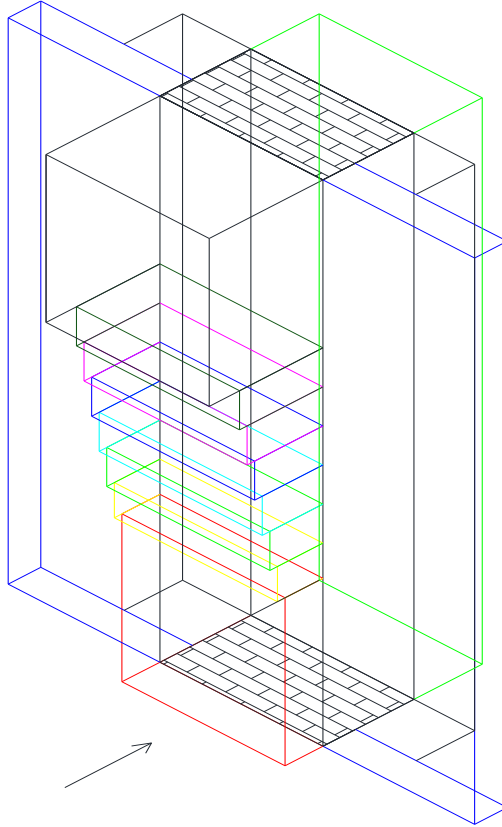


Figure 2.2: Design wind pressure in N-S direction (Figure 7.4).

A building, whose height h is greater than $2b$ may be considered to be in multiple parts, comprising : a lower part extending upwards from the ground by a height equal to b ; an upper part extending downwards from the top by a height equal to b and a middle region, between the upper and lower parts, which may be divided into horizontal strips with a height h strip.



$$D_1 : q_p(z) = q_p(b)$$

$$D_2 : q_p(z) = q_p(z_{2str})$$

$$D_3 : q_p(z) = q_p(z_{3str})$$

$$D_4 : q_p(z) = q_p(z_{4str})$$

$$D_5 : q_p(z) = q_p(z_{5str})$$

$$D_6 : q_p(z) = q_p(z_{6str})$$

$$D_7 : q_p(z) = q_p(z_{7str})$$

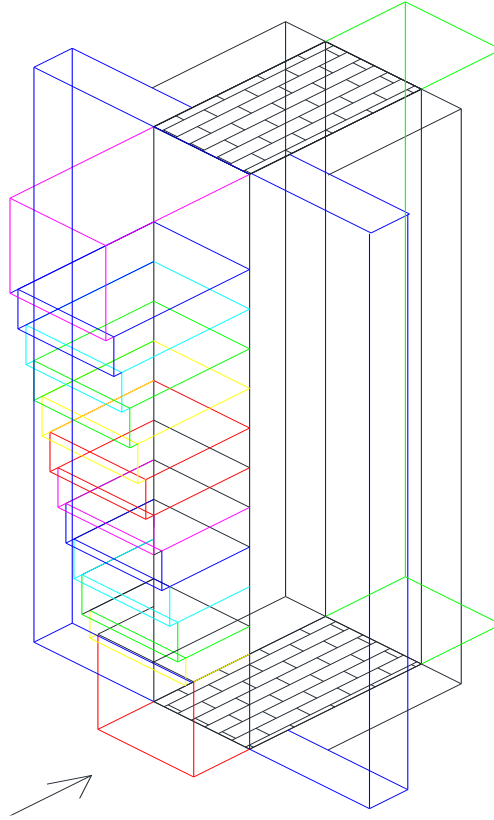
$$D_8 : q_p(z) = q_p(z_{8str})$$

$$A : q_p(z) = q_p(z_{Astr})$$

$$B : q_p(z) = q_p(z_{Bstr})$$

$$E : q_p(z) = q_p(z_{Estr})$$

Figure 2.3: Design wind pressure in N-S direction.



$$\begin{aligned}
 D_1 : q_p(z) &= q_p(b) & A : q_p(z) &= q_p(z_{Astr}) \\
 D_2 : q_p(z) &= q_p(z_{2str}) & B : q_p(z) &= q_p(z_{Bstr}) \\
 D_3 : q_p(z) &= q_p(z_{3str}) & E : q_p(z) &= q_p(z_{Estr}) \\
 D_4 : q_p(z) &= q_p(z_{4str}) \\
 D_5 : q_p(z) &= q_p(z_{5str}) \\
 D_6 : q_p(z) &= q_p(z_{6str}) \\
 D_7 : q_p(z) &= q_p(z_{7str}) \\
 D_8 : q_p(z) &= q_p(z_{8str}) \\
 D_9 : q_p(z) &= q_p(z_{9str}) \\
 D_{10} : q_p(z) &= q_p(z_{10str}) \\
 D_{11} : q_p(z) &= q_p(z_{12str}) \\
 D_{12} : q_p(z) &= q_p(z_{12str})
 \end{aligned}$$

Figure 2.4: Design wind pressure in W-E direction.

Wind pressure on surface: (EN 1991-1-4; Point 5.2- Equation 5.1)

$$w_e = q_p(z_s) \cdot c_{p,net} \quad (2.25)$$

where: $q_p(z_s)$ - is the peak velocity pressure

(z_s) - is the reference height for the external pressure given in Section 7

$c_{p,net}$ - is the pressure coefficient for the external pressure, see Section 7

2.4.3 Wind Forces

The wind force acting on a structure may be determined by summation of vectors of the forces calculated from the external and internal pressures

- Wind pressure:

$$w_x = q_p(z_s) \cdot c_{pe} \cdot c_s c_d \quad (2.26)$$

$$c_{pe} = c_{pe}(D) - c_{pe}(E) \quad (2.27)$$

- Wind force : (EN 1991-1-4; pkt. 5.3)

$$F_w = w_x \cdot A_{ref} \quad (2.28)$$

Table 2.5 : Pressure coefficients for the external pressure N-S direction.

z_e	$c_r(z_e)$	$V_m(z_e)$	$I_v(z_e)$	q_{pze} [kN/m ²]	$C_{pe}(A)$	$C_{pe}(B)$	$C_{pe}(D)$	$C_{pe}(E)$	$C_s C_d$
145	1,145	48,075	0,201	3,476	-1,2	-0,8	0,8	-0,7	0,92
102	1,064	44,677	0,216	3,136					
92	1,040	43,680	0,221	3,039					
82	1,014	42,569	0,227	2,932					
72	0,984	41,313	0,234	2,813					
62	0,949	39,868	0,242	2,678					
52	0,909	38,169	0,253	2,524					
43	0,865	36,333	0,266	2,361					

Table 2.6 : Design wind pressure in N-S direction.

ze	Wa [kN/m²]	Wb [kN/m²]	Wd [kN/m²]	We [kN/m²]	wx [kN/m²]	Aref [m²]	Wx [kN]
145	-4,172	-2,781	2,781	-2,433	4,797	1849	8870,193
102			2,509		4,327	430	1860,728
92			2,431		4,193	430	1803,067
82			2,345		4,046	430	1739,634
72			2,250		3,881	430	1669,044
62			2,143		3,696	430	1589,329
52			2,019		3,483	387	1347,788
43			1,888		3,258	1849	6023,341
							24903,125

Table 2.7 : Pressure coefficients for the external pressure W-E direction.

ze	Cr (ze)	Vm(ze)	Iv(ze)	qpze [kN/m²]	Cpe(A)	Cpe(B)	Cpe(D)	Cpe(E)	CsCd
145	1,145	48,075	0,201	3,476	-1,2	-0,8	0,8	-0,7	0,92
121	1,103	46,327	0,209	3,299					
111	1,083	45,494	0,212	3,216					
101	1,061	44,582	0,217	3,126					
91	1,037	43,575	0,222	3,028					
81	1,011	42,450	0,228	2,920					
71	0,980	41,177	0,235	2,800					
61	0,946	39,711	0,243	2,664					
51	0,904	37,981	0,254	2,507					
41	0,854	35,873	0,269	2,320					
31	0,790	33,172	0,291	2,090					
24	0,731	30,700	0,315	1,887					

Table 2.8 : Design wind pressure in W-E direction.

ze	Wa [kN/m²]	Wb [kN/m²]	Wd [kN/m²]	We [kN/m²]	Wx [kN/m²]	Aref [m²]	Wx [kN]
145	-4,172	-2,781	2,781	-2,433	4,797	576,000	2763,240
121			2,639		4,553	240,000	1092,727
111			2,573		4,438	240,000	1065,226
101			2,501		4,314	240,000	1035,455
91			2,423		4,179	240,000	1002,979
81			2,336		4,030	240,000	967,214
71			2,240		3,864	240,000	927,361
61			2,131		3,676	240,000	882,281
51			2,005		3,459	240,000	830,255
41			1,856		3,202	240,000	768,512
31			1,672		2,884	168,000	484,475
24			1,509		2,603	576,000	1499,553
							13319,279

2.5 Seismic Load

2.5.1 Response Spectrum Method

To design a structure against earthquake, one of the most recommended methods by design specifications is method called response spectrum analysis, where maximum responses for a system are estimated.

The method include the calculation of the maximum values of the structure's response defined for example as displacements and later member forces in all modes of vibration.

Response spectra is expressed by curves which specify earthquake ground motion and time period or frequency. The peak structural responses are defined and that can be used for obtaining lateral forces developed in structure due to earthquake.

The earthquake loading will be represented by an acceleration response spectrum, modified to account for inelastic deformation of the structure.

The elastic design spectrum will be obtained through probabilistic seismic hazard analysis, which provides the most rational framework for handling the large uncertainties associated with the models for seismicity and ground-motion prediction.

Parameters:

- Type of response spectrum: 1

Eurocode 8 suggest two different types of response spectrum model. Type 1 is more appropriate for for more seismic regions of southern Europe (refers to earthquake size close to M7) and second one (Type 2)for the less seismic regions of central and northern Europe. Because building is located in Turkey, Type 1 spectrum model was chosen

- Direction: Horizontal and Vertical

Eurocode 8 defines the vertical response spectrum independently, thats why the frequency content of the vertical response is different than the horizontal one.

Horizontal seismic action: two orthogonal components with the same response spectrum.

Vertical seismic action:

$T < 0,15 \text{ s}$ the vertical ordinates = 0,15 horizontal

$T > 0,15 \text{ s}$ the vertical ordinates = 0,5 horizontal

According to Table 3.4. EN 1998-1:2004- Recommended values of parameteres describing vertical elastic response spectra

Type 1:

$$\frac{a_{vg}}{a_g} = 0,9 \quad (2.29)$$
$$a_{vg} = 0,9 \cdot 0,3g = 0,3g$$

If a_{vg} is greater than 0,25 g the vertical component of the seismic action should be taken into account.

- Importance class: III (Table 4.3 of EN 1998-1:2004)

Buildings whose seismic resistance is of importance in view of the consequences associated with collapse

The reference peak ground acceleration a_{gr} and the design ground acceleration a_g was chosen as for high seismic zone.

$$a_{gr} = 0,25g$$
$$a_g = \gamma_{III} \cdot a_{gr} = 1,2 \cdot 0,25g = 0,3g \quad (2.30)$$

- Ground acceleration

$$\frac{a_g}{g} = 0,3$$

For importance class III: $\gamma_{III} = 1,2$

- Ground type: A (clause 3.1.2 and Table 3.1 EN 1998-1:2004)

Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.

- Horizontal elastic response spectrum data
(clause 3.2.2.2 and Table 3.2 EN 1998-1:2004)

$$S = 1$$

$$T_B = 0,15 \text{ s}$$

$$T_C = 0,4 \text{ s}$$

$$T_D = 2 \text{ s}$$

- Vertical elastic response spectrum data
(clause 3.2.2.2 and Table 3.4 EN 1998-1:2004)

$$T_B = 0,05 \text{ s}$$

$$T_C = 0,15 \text{ s}$$

$$T_D = 1 \text{ s}$$

- Lower bound factor β

$$\beta = 0,2$$

- Behaviour factor q

According to Journal of Civil Engineering and Architecture: Seismic Safety of RC Framed Buildings :

$$q = 5,85$$

This is a reduction factor of the design response spectrum in relation to the elastic response spectrum. The behavior factor q reduces the design spectrum and in this way decreases seismic loads and the resulting internal forces. However, as specified in clause 3.2.2.5 the displacement based on the design response spectrum is multiplied by the behavior factor q . Thus, usually such displacement is not reduced in relation to the elastic response spectrum.

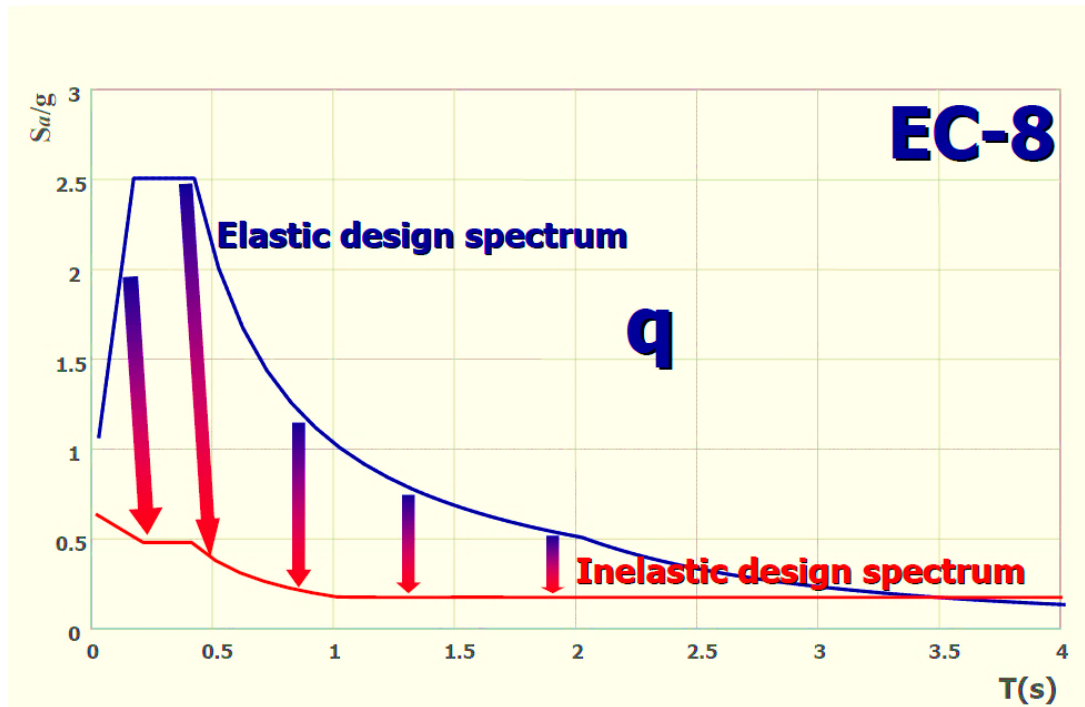


Figure 2.5: Impact of behaviour factor q to elastic design spectrum.

- Mass source

Seismic mass:

$$m = G + \Psi_{Ei} \cdot Q \quad (2.31)$$

G -Dead load

Q -Imposed Load

Ψ_{Ei} - Combination coefficient defined in 3.2.4(2)P in EC8

(Equation 4.2 EN 1998-1:2004)

$$\Psi_{Ei} = \phi \cdot \Psi_{2i} \quad (2.32)$$

Value of ϕ for independently occupied story is 0,5 according to Table 4.2 in compliance with EN 1991-1-1:2002

Ψ_{2i} - Load factor for combinations

$$\Psi_{2Q} = 0,3$$

$$\Psi_{EQ} = 0,5 \cdot 0,3 = 0,15$$

$$m = G + 0,15 \cdot Q$$

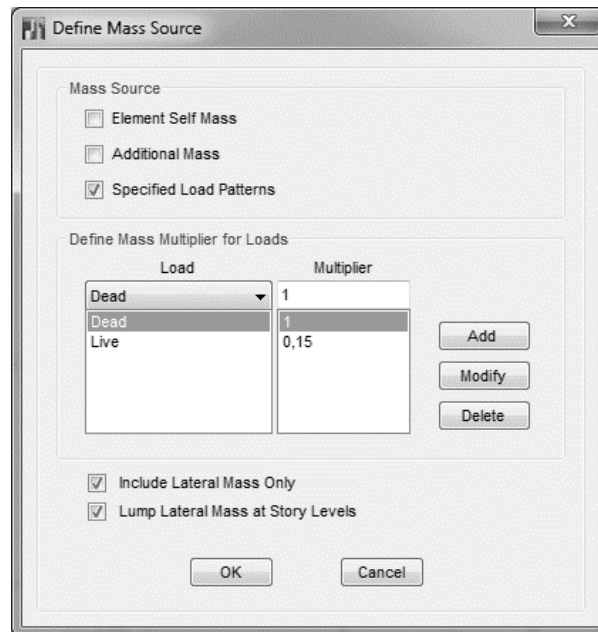


Figure 2.6: Mass source according to EN 1998-1:2004.

According to parameters discussed above, response spectrum curves in horizontal and vertical direction were created with the help of ETABS 2013.

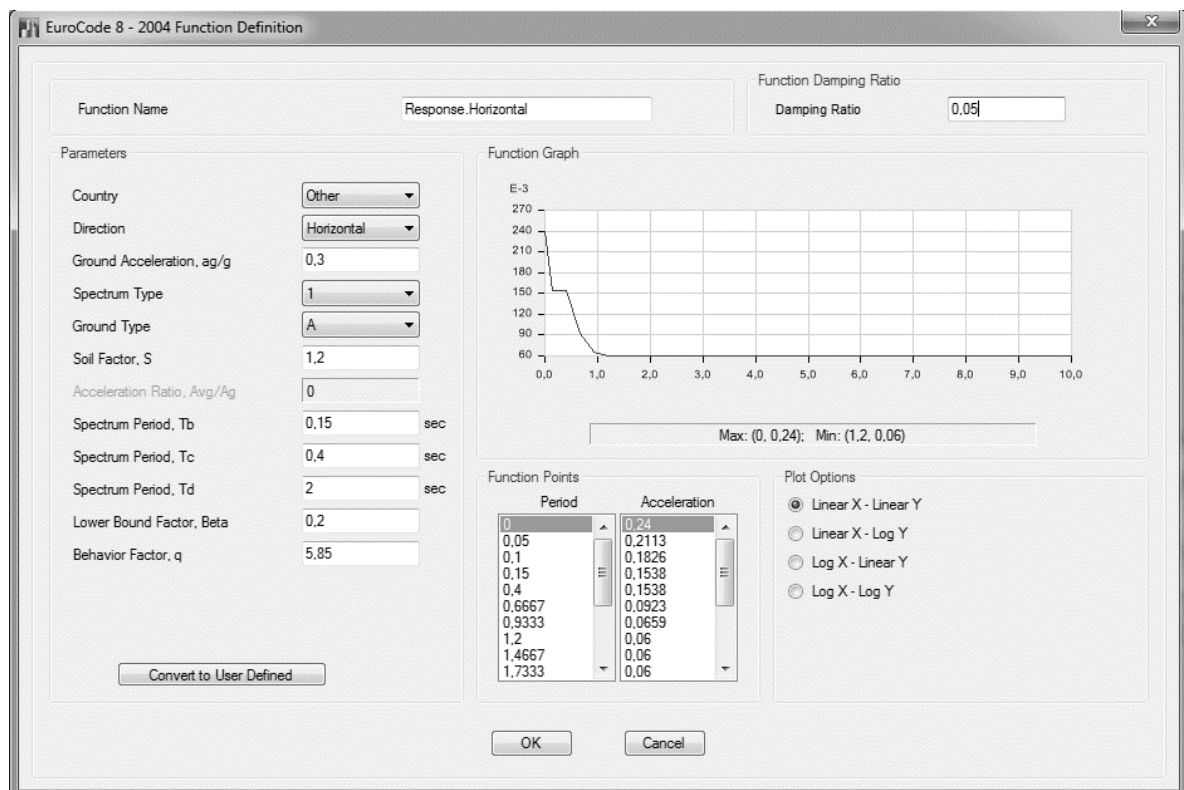


Figure 2.7: Horizontal response spectrum parameters.

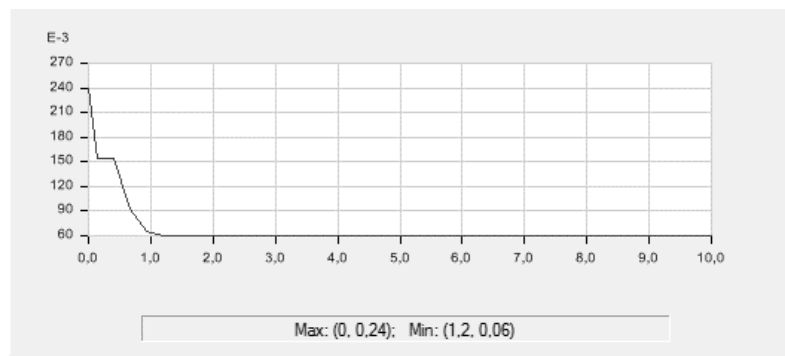


Figure 2.8: Horizontal response spectrum curve.

EuroCode 8 - 2004 Function Definition

Function Name: Response.Vertical

Function Damping Ratio: 0.05

Parameters:

- Country: Other
- Direction: Vertical
- Ground Acceleration, a_g/g : 0.3
- Spectrum Type: 1
- Ground Type:
- Soil Factor, S :
- Acceleration Ratio, Avg/A_g : 0.9
- Spectrum Period, T_b : 0.05 sec
- Spectrum Period, T_c : 0.15 sec
- Spectrum Period, T_d : 1 sec
- Lower Bound Factor, β : 0.2
- Behavior Factor, q : 5.85

Function Graph:

Max: (0, 0.18); Min: (0.291667, 0.06)

Function Points:

Period	Acceleration
0	0.18
0.0167	0.1585
0.0333	0.1369
0.05	0.1154
0.15	0.1154
0.2917	0.06
0.4333	0.06
0.575	0.06
0.7167	0.06
0.8583	0.06

Plot Options:

- ☒ Linear X - Linear Y
- ☐ Linear X - Log Y
- ☐ Log X - Linear Y
- ☐ Log X - Log Y

Convert to User Defined

OK Cancel

Figure 2.9: Vertical response spectrum parameters.

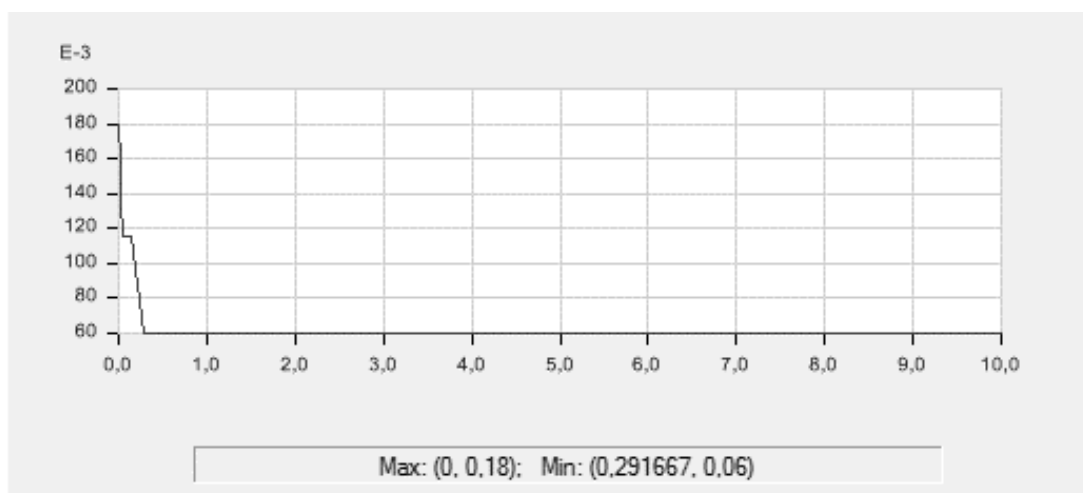


Figure 2.10: Vertical response spectrum curve.

Scale factor was not reduced in definition of earthquake load case in any direction. Reduction factor β , which transform the design response spectrum in relation to the elastic response spectrum was used while response spectrum curves were defined.

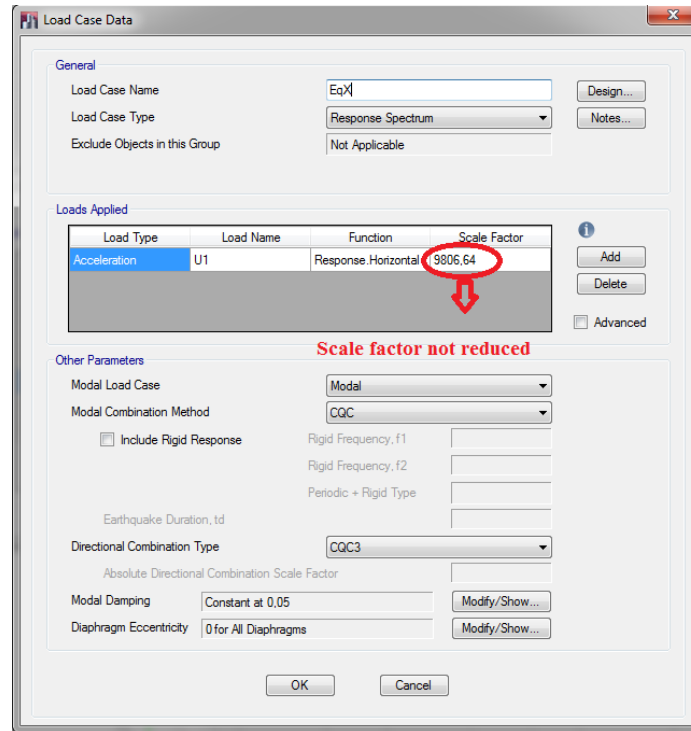


Figure 2.11: Earthquake load case definition in ETABS 2013.

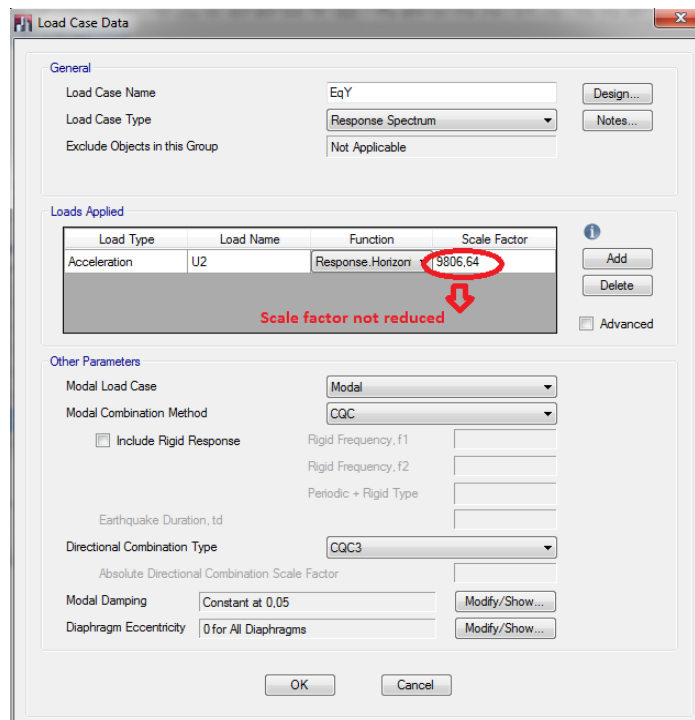


Figure 2.12: Earthquake load case definition in ETABS 2013.

2.5.2 Newmark Seismic Combinations:

$$E_1 = E_x + 0,3E_y + 0,3E_z \quad (2.32)$$

$$E_2 = E_x - 0,3E_y + 0,3E_z \quad (2.33)$$

$$E_3 = E_x - 0,3E_y - 0,3E_z \quad (2.34)$$

$$E_4 = E_x + 0,3E_y - 0,3E_z \quad (2.35)$$

$$E_5 = 0,3E_x + E_y + 0,3E_z \quad (2.36)$$

$$E_6 = 0,3E_x - E_y + 0,3E_z \quad (2.37)$$

$$E_7 = 0,3E_x - E_y - 0,3E_z \quad (2.38)$$

$$E_8 = 0,3E_x + E_y - 0,3E_z \quad (2.39)$$

$$E_9 = 0,3E_x + 0,3E_y + E_z \quad (2.40)$$

$$E_{10} = 0,3E_x - 0,3E_y + E_z \quad (2.41)$$

$$E_{11} = 0,3E_x - 0,3E_y - E_z \quad (2.42)$$

$$E_{12} = 0,3E_x + 0,3E_y - E_z \quad (2.43)$$

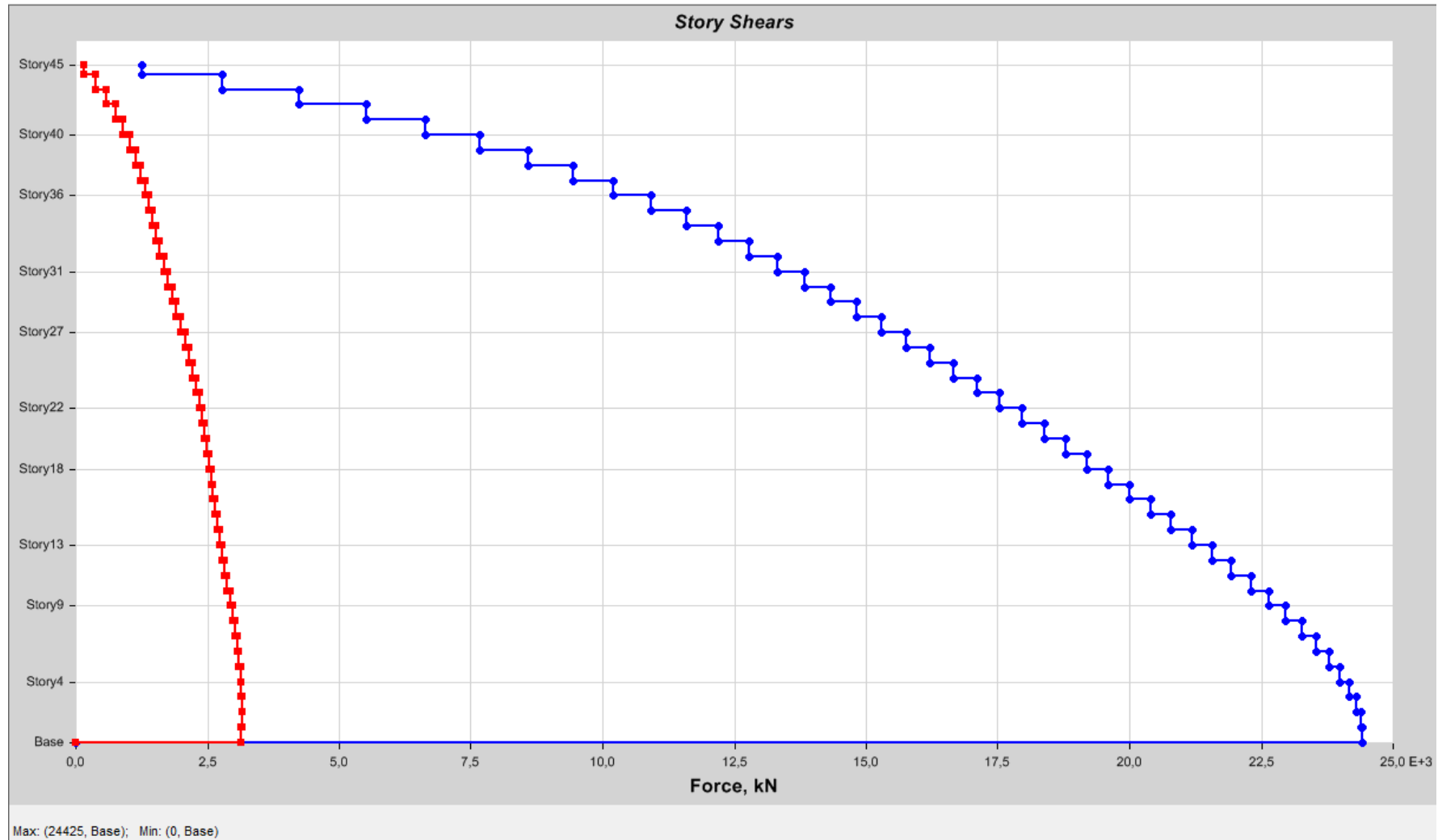


Figure 2.13: Story shear due to earthquake load in X direction- EN 1998-1:2004.

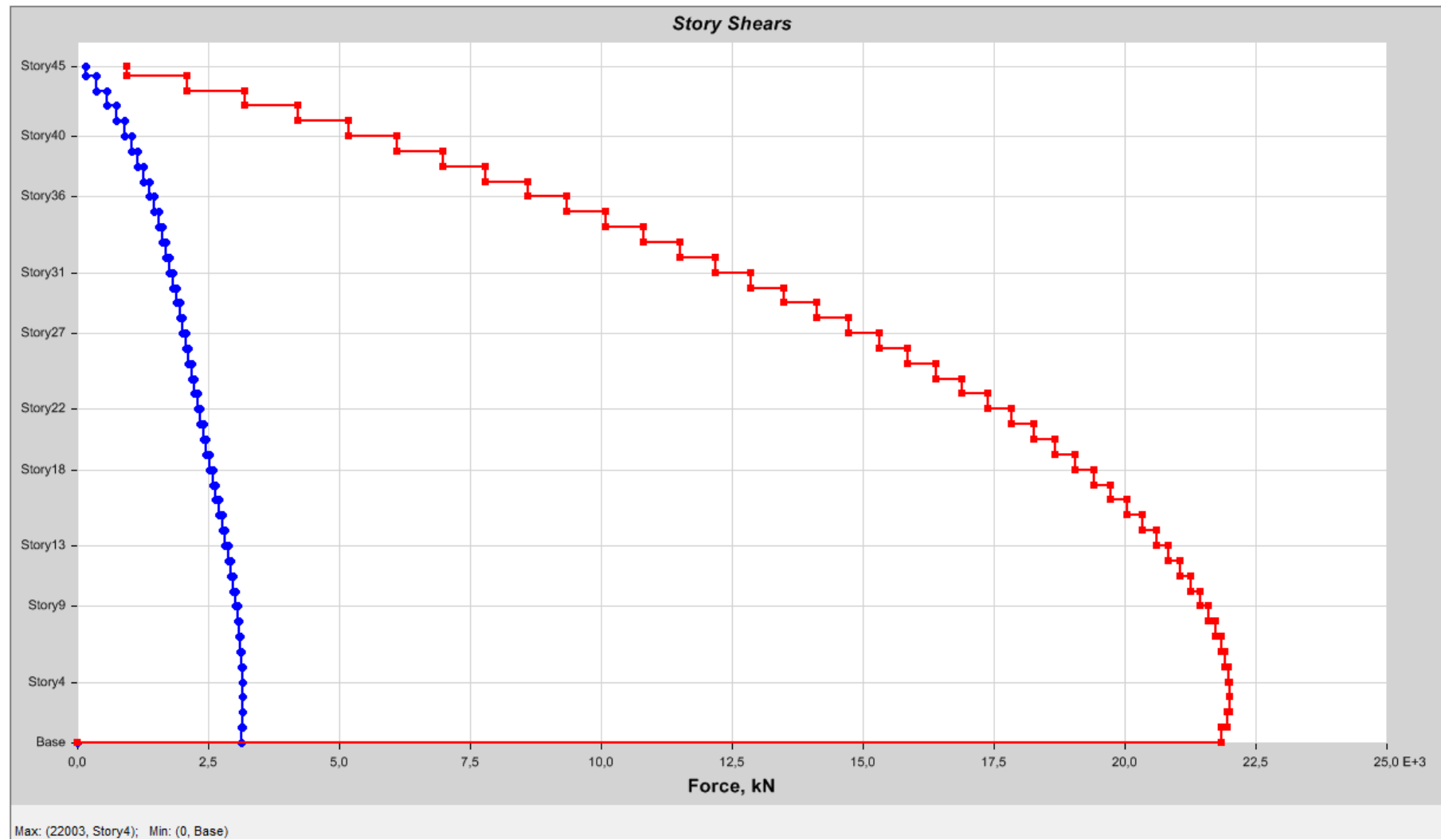


Figure 2.14: Story shear due to earthquake load in Y direction- EN 1998-1:2004.

2.6 Snow load

Snow loads in general is classified as variable/accidental,direct,fixed,static action.

Snow loads on the roof should be determined as follows:

$$S = u_i \cdot c_e \cdot c_t \cdot s_k \quad (2.45)$$

where: u_i is the roof shape coefficient

c_e is the exposure coefficient, usually taken as 1,0

c_t is the thermal coefficient, set to 1,0 for normal situations

s_k is the characteristic value of ground snow load for the relevant altitude

Roof shape coefficient

Shape coefficients are needed for an adjustment of the ground snow load to a snow load on the roof taking into account effects caused by non-drifted and drifted snow load arrangements. The roof shape coefficient depends on the roof angle.

It has been assumed that the roof in designed building is flat.

$$0^\circ \leq \alpha \leq 30^\circ$$

$$\mu_1 = 0,8$$

Snow load on the ground

The characteristic value depends on the climatic region.

For a site in Istanbul (Maslak) which belongs to II snow zone in Turkey, snow load value according to TS 498 is assumed as:

$$s_k = 0,75 \text{ kN/m}^2$$

Snow load on the roof:

$$s = 0,8 \cdot 1 \cdot 1 \cdot 0,75 = 0,6 \text{ kN/m}^2$$

2.7 Load Combinations

Two main criteria, namely the ultimate limit state (ULS) and the serviceability limit state (SLS) have to be considered and satisfied when structure is calculated according to limit state design. This is a set of performance criteria (deflection, strength, buckling, vibration levels ect.) which has to be met when the structure is subjected to loads.

2.7.1 Ultimate Limit State (ULS)

Those limit states concern especially with the safety of people and safety of the structure. It should be verified where the equilibrium of the structure or any part of it can be loss or when failure by deformation , transformation is possible . To satisfy the ULS, the structure can not collapse when is subjected to the peak design load for which it was designed.

2.7.1.1 Persistent design situation

$$\sum_j \gamma_{G,j} \cdot G_{k,j} + \gamma_{Q,1} \cdot Q_{k,1} + \sum_{i \geq 2} \gamma_{Q,i} \cdot \psi_{Q,i} \cdot Q_{k,i} \quad (2.46)$$

Recommended values of factors for buildings (Table A1.1 of EN 1990)

- Wind: $\psi_0 = 0,6$; $\psi_1 = 0,2$; $\psi_2 = 0,0$
- Imposed load (Live load): $\psi_0 = 0,7$; $\psi_1 = 0,5$; $\psi_2 = 0,3$
- Snow: $\psi_0 = 0,5$; $\psi_1 = 0,2$; $\psi_2 = 0,0$

Moreover:

$$\gamma_G = 1,35 \text{ (1,0 when unfavorable)}$$

$$\gamma_{Q,1} = 1,5 \text{ (0,0 when unfavorable)}$$

$$\gamma_{Q,i} = 1,5 \text{ (0,0 when unfavorable)}$$

$$1. \quad 1,35 G + 1,5 W \quad (2.47)$$

$$2. \quad 0,9 G + 1,5 W \quad (2.48)$$

$$3. \quad 1,35 G + 1,5 Q \quad (2.49)$$

$$4. \quad 1,35 G + 1,5 S \quad (2.50)$$

$$5. \quad 1,35 G + 1,5 W + 1,5(0,7 \cdot Q + 0,5 \cdot S) \quad (2.51)$$

$$1,35G + 1,5 W + 1,05Q + 0,75S$$

$$1,35G + 1,5 W + 1,05S + 0,75Q$$

$$6. \quad 1,35 G + 1,5 Q + 1,5(0,6 \cdot W + 0,5 \cdot S) \quad (2.52)$$

$$1,35G + 1,5 Q + 0,9W + 0,75S$$

$$1,35G + 1,5 Q + 0,9S + 0,75W$$

$$7. \quad 1,35 G + 1,5 S + 1,5(0,6 \cdot W + 0,7 \cdot Q) \quad (2.53)$$

$$1,35G + 1,5 S + 0,9W + 1,05Q$$

$$1,35G + 1,5 S + 0,9Q + 1,05W$$

$$8. \quad 1,35 G + 1,5 W + 1,05(S + Q) \quad (2.54)$$

$$1,35G + 1,5 W + 1,05S + 1,05Q$$

$$9. \quad 1,35 G + 1,5 (S + Q) + 1,05W \quad (2.55)$$

$$1,35G + 1,5 S + 1,5Q + 1,05W$$

2.7.1.2 Seismic design:

$$G_k + \sum_{i \geq 1} \Psi_2 Q_k + E \quad (2.56)$$

$$G + 0,3 \cdot Q + 0,0 \cdot S + 0,0 \cdot W + E = G + 0,3 \cdot Q + E$$

Using Newmark seismic combinations:

$$10. \quad G + 0,3 \cdot Q + (E_x + 0,3E_y + 0,3E_z) \quad (2.57)$$

$$11. \quad G + 0,3 \cdot Q + (E_x - 0,3E_y + 0,3E_z) \quad (2.58)$$

$$12. \quad G + 0,3 \cdot Q + (E_x - 0,3E_y - 0,3E_z) \quad (2.59)$$

$$13. \quad G + 0,3 \cdot Q + (E_x + 0,3E_y - 0,3E_z) \quad (2.60)$$

$$14. \quad G + 0,3 \cdot Q + (0,3E_x + E_y + 0,3E_z) \quad (2.61)$$

$$15. \quad G + 0,3 \cdot Q + (0,3E_x - E_y + 0,3E_z) \quad (2.62)$$

$$16. \quad G + 0,3 \cdot Q + (0,3E_x - E_y - 0,3E_z) \quad (2.63)$$

$$17. \quad G + 0,3 \cdot Q + (0,3E_x + E_y - 0,3E_z) \quad (2.64)$$

$$18. \quad G + 0,3 \cdot Q + (0,3E_x + 0,3E_y + E_z) \quad (2.65)$$

$$19. \quad G + 0,3 \cdot Q + (0,3E_x - 0,3E_y + E_z) \quad (2.66)$$

$$20. \quad G + 0,3 \cdot Q + (0,3E_x - 0,3E_y - E_z) \quad (2.67)$$

$$21. \quad G + 0,3 \cdot Q + (0,3E_x + 0,3E_y - E_z) \quad (2.68)$$

2.7.2 Serviceability Limit State (SLS)

To satisfy the SLS criteria, a structure must be functional for its use subject to routine loading, and as such the structure must not cause occupant discomfort under normal conditions. If elements of the structure don't fall within predetermined vibration criteria or don't deflect more than building regulation's limits assumed previously SLS is satisfied. In general, in cases where the SLS requirements are not met, element will not necessarily fail structurally. The main idea of SLS is to provide comfort and sense of security to people who are going to use the structure.

However, the main purpose of this thesis is to compare regulations, and because of this SLS in design beam, column and shear wall will not be checked.

2.7.2.1 Persistent design situation

a) Characteristic combinations

$$\sum_j G_{k,j} + Q_{k,1} + \sum_{i \geq 2} \Psi_{0,i} Q_{k,i} \quad (2.69)$$

$$22. G + Q \quad (2.70)$$

$$23. G + W \quad (2.71)$$

$$24. G + S \quad (2.72)$$

$$25. G + Q + 0,5 \cdot S + 0,6 \cdot W \quad (2.73)$$

$$26. G + S + 0,7 \cdot Q + 0,6 \cdot W \quad (2.74)$$

$$27. G + W + 0,7 \cdot Q + 0,5 \cdot S \quad (2.75)$$

b) Quasi-permanent combination

$$\sum_j G_{k,j} + \sum_{i \geq 2} \Psi_{2,i} Q_{k,i} \quad (2.76)$$

$$28. G + 0,3 \cdot Q + 0,0 \cdot S + 0,0 \cdot W = G + 0,3 \cdot Q \quad (2.77)$$

2.7.2.2 Seismic design

Lateral storey displacements have to be determined according to specific requirements of EN 1998-1

3. DESIGN ACCORDING TO AMERICAN REGULATIONS

ASCE 7-10 was prepared by the Committee on Minimum Design Loads for Buildings and Other Structures of the Codes and Standards Activities Division of the Structural Engineering Institute of ASCE. Provides requirements for general structural design.

3.1 Dead Load

Minimum design dead load values showed in the tables below give just some idea about weight of construction elements. However, in reality, on the site, because of other factors like precision and weather conditions they may exceeded mentioned limits.

Table 3.1 : Minimum design dead load for floor.

Type of loading	Characteristic value [kN/m ²]
Carpeted Floor 0,07 kN/m ²	0,07
Alignment layer of cement mortar 3 cm 23 kN/m ³ · 0,03	0,69
Insulating film 0,05 kN/m ²	0,05
Mineral wool - Stoprock 5 cm 1,61 kN/m ³ · 0,05	0,08
Reinforced concrete slab 18 cm 25 kN/m ³ · 0,018 m	4,5
Suspended ceiling 0,01 kN/m ²	0,01
	5,4

Table 3.2 : Minimum design dead load for floor edges.

Type of loading	Characteristic value [kN/m ²]
Glazing 19 mm 24 kN/m ³ · 0,019	0,456
Supporting structure 0,3 kN/m ²	0,3
	0,756

3.2 Live Load

According to Table 1607.1 from IBC 2012 live load was assumed as follow:

Table 3.3 : Minimum design live load.

Live load	Minimum Live Load [kN/m ²]
Floor area	2,64
Roof	0,958

3.3 Wind Calculations

The Analytical Procedure (Method 2) of ASCE 6.5 may be used to determine the wind forces. For the sake of simplicity the building is assumed to be a regular-shaped (having no unusual geometrical irregularity in spatial form).

The building does not have response characteristics making it subject to across wind loading, vortex shedding, instability due to galloping or flutter. It is assumed that the site location is such that channeling effects or buffeting in the wake of upwind obstructions need not be determine the wind forces.

The design procedure is used to determine the wind forces on the building in both the N-S and E-W directions.

Basic wind speed V and wind directionality factor K_d
(ASCE 6.5.4, Figure 6-1, Table 6-4)

Basic wind speed is nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category

$$V = 42 \text{ m/s}$$

The fundamental value of the basic wind velocity was taken for the reference height. The wind directionality factor accounts for the fact that the probability that the maximum wind may not affect the structural component or system in its weakest orientation.

$$K_d = 0,85 \quad (\text{main wind-force-resisting systems})$$

Importance factor (ASCE Table 6-1)

The factor of the building or structure, based on its occupancy type, which functions as an implicit adjustment factor to the return period. Importance factor depends on the category of the structure.

For non-Hurricane Prone Regions importance factor is equal to:

$$I = 1$$

Category of building: (ASCE Table 1-1)

Building and other structures that represent low hazard on human life in the event of failure including (all buildings and other structures except those listed in Category I, II and III)- Category II

Velocity pressure exposure coefficient k_z (ASCE 6.5.6)

K_z is the velocity pressure exposure coefficient that is defined according to system or component design cases and terrain category

Values of K_z are to be determined from Table 6-3. In lieu of linear interpolation K_z may be calculated at any height z above ground level by the equation given at the bottom of ASCE Table 6-3.

$$K_z = \begin{cases} 2.01 \left(\frac{15}{z_g} \right)^{\frac{2}{\alpha}} & \text{for } z < 15 \text{ ft } (4,572 \text{ m }) \\ 2.01 \left(\frac{z}{z_g} \right)^{\frac{2}{\alpha}} & \text{for } 15 \text{ ft } \leq z \leq z_g (4,572 \text{ m }) \end{cases} \quad (3.1)$$

where: α : 3 –second gust speed power law exponent from ASCE Table 6-2

z_g : nominal height of the atmospheric boundary layer from ASCE Table 6-2

For Exposure C:

$$\alpha = 9,5$$

$$z_g = 900 \text{ ft} = 274,32 \text{ m}$$

Table 3.4: Velocity pressure exposure coefficients.

Level	Height above ground level z [m]	KZ	Level	Height above ground level z [m]	KZ
45	145,00	1,758	22	71,50	1,514
44	143,00	1,752	21	68,25	1,500
43	139,75	1,744	20	65,00	1,484
42	136,50	1,735	19	61,75	1,468
41	133,25	1,727	18	58,50	1,452
40	130,00	1,718	17	55,25	1,434
39	126,75	1,708	16	52,00	1,416
38	123,50	1,699	15	48,75	1,397
37	120,25	1,690	14	45,50	1,377
36	117,00	1,680	13	42,25	1,356
35	113,75	1,670	12	39,00	1,333
34	110,50	1,660	11	35,75	1,309
33	107,25	1,649	10	32,50	1,283
32	104,00	1,639	9	29,25	1,255
31	100,75	1,628	8	26,00	1,224
30	97,50	1,617	7	22,75	1,190
29	94,25	1,605	6	19,50	1,152
28	91,00	1,593	5	16,25	1,109
27	87,75	1,581	4	13,00	1,058
26	84,50	1,569	3	9,75	0,996
25	81,25	1,556	2	6,50	0,914
24	78,00	1,542	1	3,25	1,090
23	74,75	1,529			

Topographic factor K_{zt} (ASCE 6.5.7)

Assumed that the building is situated on level ground and not on hill, ridge or escarpment $K_{zt} = 1$

3.3.1 Gust Effect Factors G and Gf

Effects due to wind gust depend on whether a building is rigid or flexible.

A rigid building has a fundamental frequency n_1 greater than or equal to 1 Hz, while a flexible building has a fundamental natural frequency less than 1 Hz (ASCE 6.2)

Natural frequency in the N-S and E-W direction: (Equation C 6-17)

$$n_1 = \frac{100}{H} \quad (3.2)$$
$$H = 145 \text{ m} = 475,72 \text{ ft}$$

where: H is the height in ft above the base to the highest level of the structure

$$n_1 = \frac{100}{475,72} = 0,21 \text{ Hz}$$

Since n_1 is less than 1 Hz, the building is considered flexible.
(Equation 6.8)

$$G_f = 0,925 + \left(\frac{1 + 1,7 \cdot I_z \sqrt{g_Q^2 \cdot Q^2 + g_R^2 \cdot R^2}}{1 + 1,7 g_v \cdot I_z} \right) \quad (3.3)$$

Where : g_R is a peak factor for resonant response

g_Q is a peak factor for background response

g_V is a peak factor for wind response

R is the resonant response factor

$$g_Q = g_V = 3,4$$
$$g_R = \sqrt{2 \cdot \ln(3600 \cdot n_1)} + \frac{0,577}{\sqrt{2 \cdot \ln(3600 \cdot n_1)}} \quad (3.4)$$

$$g_R = \sqrt{2 \cdot \ln(3600 \cdot 0,21)} + \frac{0,577}{\sqrt{2 \cdot \ln(3600 \cdot 0,21)}} = 3,80$$

Intensity of turbulence at height $\bar{z}$ (ASCE Table 6-2, Equation 6.5)

$$I_{\bar{z}} = c \cdot \left(\frac{10}{\bar{z}} \right)^{\frac{1}{6}} \quad (3.5)$$

$$I_{\bar{z}} = 0,2 \cdot \left(\frac{10}{87} \right)^{\frac{1}{6}} = 0,14$$

Where: $\bar{z}$ is an equivalent height of structure

$$\bar{z} = 0,6 \cdot h \geq z_{min} = 15ft (4,57 m)$$

$$\bar{z} = 0,6 \cdot h = 0,6 \cdot 145m = 87 m$$

3.3.1.1 Background response- wind direction N-S

(Equation 6.6)

$$Q = \sqrt{\frac{1}{1 + 0,63 \cdot \left(\frac{B + h}{L_{\bar{z}}} \right)^{0,63}}} \quad (3.6)$$

Where: B - horizontal dimension of building measured normal to wind direction, in ft

h -mean roof height of a building or height of other structure, except that eave height shall be used for roof angle θ

Integral length scale of turbulence at equivalent height $\bar{z}$

(Equation 6-7, Table 6-2 for exposure C)

$$L_{\bar{z}} = l \left(\frac{\bar{z}}{10} \right)^{\bar{\epsilon}} \quad (3.7)$$

$$L_{\bar{z}} = 152,4 \cdot \left(\frac{87}{10} \right)^{\frac{1}{5}} = 234,9 m$$

$$Q = \sqrt{\frac{1}{1 + 0,63 \cdot \left(\frac{43 + 145}{234,9} \right)^{0,63}}} = 0,8$$

The Mean hourly wind speed (m/s) at height $\bar{z}$

(Equation 6-14)

$$\bar{V}_{\bar{z}} = \bar{b} \cdot \left(\frac{\bar{z}}{10} \right)^{\bar{a}} \cdot V \quad (3.8)$$

$$\bar{V}_{\bar{z}} = 0,65 \cdot \left(\frac{87}{10} \right)^{\frac{1}{6,5}} \cdot 42 = 38,08 m/s$$

The resonance response factor: (Equation 6-10)

$$R = \sqrt{\frac{1}{\beta} \cdot R_n \cdot R_h \cdot R_B \cdot (0,53 + 0,47 \cdot R_L)} \quad (3.9)$$

The damping inherent in the structural systems of tall buildings cannot currently be predicted using detailed analysis method. Common practice over several decades has been to assume damping ratios of approximately 0,010 to 0,020 for concrete building $\beta = 0,01$

Reduced frequency:

(Equation 6.12)

$$N_1 = \frac{n_1 \cdot L_{\bar{z}}}{\bar{V}_{\bar{z}}} \quad (3.10)$$

$$N_1 = \frac{0,21 \cdot 234,9}{38,08} = 1,3$$

(Equation 6-11)

$$R_n = \frac{7,47 \cdot N_1}{(1 + 10,3 \cdot N_1)^{\frac{5}{3}}} \quad (3.11)$$

$$R_n = \frac{7,47 \cdot 1,3}{(1 + 10,3 \cdot 1,3)^{\frac{5}{3}}} = 0,11$$

(Equation 6.13a)

$$R_h = \frac{1}{\eta_h} - \frac{1}{2 \cdot \eta_h^2} \cdot (1 - e^{-2 \cdot \eta_h}) \quad (3.12)$$

$$R_h = \frac{1}{3,68} - \frac{1}{2 \cdot 3,68^2} \cdot (1 - e^{-2 \cdot 3,68}) = 0,24$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot \eta_b^2} \cdot (1 - e^{-2 \cdot \eta_b}) \quad (3.13)$$

$$R_b = \frac{1}{1,09} - \frac{1}{2 \cdot 1,09^2} \cdot (1 - e^{-2 \cdot 1,09}) = 0,54$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2 \cdot \eta_L^2} \cdot (1 - e^{-2 \cdot \eta_L}) \quad (3.14)$$

$$R_L = \frac{1}{0,61} - \frac{1}{2 \cdot 0,61^2} \cdot (1 - e^{-2 \cdot 0,61}) = 0,69$$

$$\eta_h = \frac{4,6 \cdot n_1 \cdot h}{\bar{V}_z} = \frac{4,6 \cdot 0,21 \cdot 145}{38,08} = 3,68 \quad (3.15)$$

$$\eta_b = \frac{4,6 \cdot n_1 \cdot B}{\bar{V}_z} = \frac{4,6 \cdot 0,21 \cdot 43}{38,08} = 1,09 \quad (3.16)$$

$$\eta_L = \frac{4,6 \cdot n_1 \cdot L}{\bar{V}_z} = \frac{4,6 \cdot 0,21 \cdot 24}{38,08} = 0,61 \quad (3.17)$$

$$R = \sqrt{\frac{1}{0,01} \cdot 0,11 \cdot 0,24 \cdot 0,54 \cdot (0,53 + 0,47 \cdot 0,69)} = 1,1$$

$$G_f = 0,925 + \left(\frac{1 + 1,7 \cdot 0,14 \sqrt{3,4^2 \cdot 0,8^2 + 3,8^2 \cdot 1,1^2}}{1 + 1,7 \cdot 3,4 \cdot 0,14} \right) = 2,16$$

3.3.1.2 Background response- wind direction E-W

$$Q = \sqrt{\frac{1}{1 + 0,63 \cdot \left(\frac{23 + 145}{234,9} \right)^{0,63}}} = 0,814$$

The resonance response factor: (Equation 6.13 a)

$$R = \sqrt{\frac{1}{\beta} \cdot R_n \cdot R_h \cdot R_B \cdot (0,53 + 0,47 \cdot R_L)}$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2 \cdot \eta_h^2} \cdot (1 - e^{-2 \cdot \eta_h})$$

$$R_h = \frac{1}{3,68} - \frac{1}{2 \cdot 3,68^2} \cdot (1 - e^{-2 \cdot 3,68}) = 0,24$$

$$R_b = \frac{1}{\eta_b} - \frac{1}{2 \cdot \eta_b^2} \cdot (1 - e^{-2 \cdot \eta_b})$$

$$R_b = \frac{1}{0,61} - \frac{1}{2 \cdot 0,61^2} \cdot (1 - e^{-2 \cdot 0,61}) = 0,69$$

$$R_L = \frac{1}{\eta_L} - \frac{1}{2 \cdot \eta_L^2} \cdot (1 - e^{-2 \cdot \eta_L})$$

$$R_L = \frac{1}{1,09} - \frac{1}{2 \cdot 1,09^2} \cdot (1 - e^{-2 \cdot 1,09}) = 0,54$$

$$\eta_h = \frac{4,6 \cdot n_1 \cdot h}{\bar{V}_z} = \frac{4,6 \cdot 0,21 \cdot 145}{38,08} = 3,68$$

$$\eta_b = \frac{4,6 \cdot n_1 \cdot B}{\bar{V}_z} = \frac{4,6 \cdot 0,21 \cdot 24}{38,08} = 0,61$$

$$\eta_L = \frac{4,6 \cdot n_1 \cdot L}{\bar{V}_z} = \frac{4,6 \cdot 0,21 \cdot 43}{38,08} = 1,09$$

$$R = \sqrt{\frac{1}{0,01} \cdot 0,11 \cdot 0,24 \cdot 0,69 \cdot (0,53 + 0,47 \cdot 0,54)} = 1,2$$

$$G_f = 0,925 + \left(\frac{1 + 1,7 \cdot I_z \sqrt{g_Q^2 \cdot Q^2 + g_R^2 \cdot R^2}}{1 + 1,7 \cdot g_v \cdot I_z} \right)$$

$$G_f = 0,925 + \left(\frac{1 + 1,7 \cdot 0,14 \sqrt{3,4^2 \cdot 0,814^2 + 3,8^2 \cdot 1,2^2}}{1 + 1,7 \cdot 3,4 \cdot 0,14} \right) = 2,18$$

Enclosure Classification: (ASCE 6.5.9)

It is assumed that the building is enclosed

Internal pressure coefficients GC_{pi} (ASCE 6.5.11.1)

Internal pressure coefficients are to be determined from *ASCE Figure 6-5*, based on building enclosure classification.

$$GC_{pi} = \pm 0,18$$

Plus and minus signs signify pressures acting toward and away from the internal surfaces, respectively. Two cases should be considered to determine the critical load requirements.

External pressure coefficients C_p (ASCE 6.5.11.2)

External pressure coefficients for Main Wind Force Resisting Systems (MWFRSs) C_p are given in Figs. 6-6, 6-7, and 6-8.

The pressure coefficient values and gust effect factor in Fig. 6-10 shall not be separated.

- Wind in N-S direction:

Windward wall: $C_p = 0,8$

Leeward wall: $\frac{L}{B} = \frac{24}{43} = 0,56 \longrightarrow C_p = -0,5$

Side wall: $C_p = -0,7$

- Wind in E-W direction:

Windward wall: $C_p = 0,8$

Leeward wall: $\frac{L}{B} = \frac{43}{24} = 1,8 \longrightarrow C_p = -0,35$

Side wall: $C_p = -0,7$

Velocity pressure q_z, q_h (ASCE 6.5.10)

Evaluated at height z shall be calculated by the following equation:

$$q_z = 0,613 \cdot K_z \cdot K_{zt} \cdot K_d \cdot V^2 \cdot I \quad \left[\frac{N}{m^2} \right] \quad (3.18)$$

Table 3.5: Velocity pressure Exposure coefficients.

Level	Height above ground level z [m]	KZ	q _z [N/m ²]	Level	Height above ground level z [m]	KZ	q _z [N/m ²]
45	145,00	1,758	1615,405	22	71,50	1,514	1391,992
44	143,00	1,752	1610,689	21	68,25	1,500	1378,426
43	139,75	1,744	1602,912	20	65,00	1,484	1364,340
42	136,50	1,735	1594,991	19	61,75	1,468	1349,686
41	133,25	1,727	1586,920	18	58,50	1,452	1334,411
40	130,00	1,718	1578,692	17	55,25	1,434	1318,449
39	126,75	1,708	1570,300	16	52,00	1,416	1301,729
38	123,50	1,699	1561,736	15	48,75	1,397	1284,162
37	120,25	1,690	1552,992	14	45,50	1,377	1265,644
36	117,00	1,680	1544,060	13	42,25	1,356	1246,051
35	113,75	1,670	1534,930	12	39,00	1,333	1225,230
34	110,50	1,660	1525,591	11	35,75	1,309	1202,990
33	107,25	1,649	1516,033	10	32,50	1,283	1179,093
32	104,00	1,639	1506,244	9	29,25	1,255	1153,227
31	100,75	1,628	1496,210	8	26,00	1,224	1124,983
30	97,50	1,617	1485,917	7	22,75	1,190	1093,798
29	94,25	1,605	1475,349	6	19,50	1,152	1058,871
28	91,00	1,593	1464,490	5	16,25	1,109	1018,998
27	87,75	1,581	1453,320	4	13,00	1,058	972,235
26	84,50	1,569	1441,819	3	9,75	0,996	915,099
25	81,25	1,556	1429,963	2	6,50	0,914	840,226
24	78,00	1,542	1417,726	1	3,25	1,090	1001,970
23	74,75	1,529	1405,080				

Design wind pressure_(ASCE 6.5.12)

Sign Convention. Positive pressure acts toward the surface and negative pressure acts away from the surface.

(Equation 6-19)

$$p = q \cdot G_f \cdot C_p - q_i \cdot (G \cdot C_{pi}) \quad (3.19)$$

Where: $q = q_z$ for windward walls evaluated at height z above the ground

$q = q_h$ for leeward walls, side walls, and roofs, evaluated at height h

$q_i = q_h$ for windward walls, side walls, leeward walls, and roofs

Table 3.6 : Design wind pressure in N-S direction.

Location	Level	Height above ground level z [m]	External pressure				Internal pressure		
			q [kN/m ²]	Gf	Cp	q·Gf·Cp	qi [kN/m ²]	G·Cpi [+,-]	qi·G·Cpi [+,-]
Windward	45	145,00	1,615	2,16	0,8	2,791	1,615	0,18	0,291
	44	143,00	1,611	2,16	0,8	2,783	1,615	0,18	0,291
	43	139,75	1,603	2,16	0,8	2,770	1,615	0,18	0,291
	42	136,50	1,595	2,16	0,8	2,756	1,615	0,18	0,291
	41	133,25	1,587	2,16	0,8	2,742	1,615	0,18	0,291
	40	130,00	1,579	2,16	0,8	2,728	1,615	0,18	0,291
	39	126,75	1,570	2,16	0,8	2,713	1,615	0,18	0,291
	38	123,50	1,562	2,16	0,8	2,699	1,615	0,18	0,291
	37	120,25	1,553	2,16	0,8	2,684	1,615	0,18	0,291
	36	117,00	1,544	2,16	0,8	2,668	1,615	0,18	0,291
	35	113,75	1,535	2,16	0,8	2,652	1,615	0,18	0,291
	34	110,50	1,526	2,16	0,8	2,636	1,615	0,18	0,291
	33	107,25	1,516	2,16	0,8	2,620	1,615	0,18	0,291
	32	104,00	1,506	2,16	0,8	2,603	1,615	0,18	0,291
	31	100,75	1,496	2,16	0,8	2,585	1,615	0,18	0,291
	30	97,50	1,486	2,16	0,8	2,568	1,615	0,18	0,291
	29	94,25	1,475	2,16	0,8	2,549	1,615	0,18	0,291
	28	91,00	1,464	2,16	0,8	2,531	1,615	0,18	0,291
	27	87,75	1,453	2,16	0,8	2,511	1,615	0,18	0,291
	26	84,50	1,442	2,16	0,8	2,491	1,615	0,18	0,291
	25	81,25	1,430	2,16	0,8	2,471	1,615	0,18	0,291
	24	78,00	1,418	2,16	0,8	2,450	1,615	0,18	0,291
	23	74,75	1,405	2,16	0,8	2,428	1,615	0,18	0,291
	22	71,50	1,392	2,16	0,8	2,405	1,615	0,18	0,291
	21	68,25	1,378	2,16	0,8	2,382	1,615	0,18	0,291
	20	65,00	1,364	2,16	0,8	2,358	1,615	0,18	0,291
	19	61,75	1,350	2,16	0,8	2,332	1,615	0,18	0,291
	18	58,50	1,334	2,16	0,8	2,306	1,615	0,18	0,291
	17	55,25	1,318	2,16	0,8	2,278	1,615	0,18	0,291
	16	52,00	1,302	2,16	0,8	2,249	1,615	0,18	0,291
	15	48,75	1,284	2,16	0,8	2,219	1,615	0,18	0,291
	14	45,50	1,266	2,16	0,8	2,187	1,615	0,18	0,291
	13	42,25	1,246	2,16	0,8	2,153	1,615	0,18	0,291
	12	39,00	1,225	2,16	0,8	2,117	1,615	0,18	0,291
	11	35,75	1,203	2,16	0,8	2,079	1,615	0,18	0,291
	10	32,50	1,179	2,16	0,8	2,037	1,615	0,18	0,291
	9	29,25	1,153	2,16	0,8	1,993	1,615	0,18	0,291
	8	26,00	1,125	2,16	0,8	1,944	1,615	0,18	0,291
	7	22,75	1,094	2,16	0,8	1,890	1,615	0,18	0,291
	6	19,50	1,059	2,16	0,8	1,830	1,615	0,18	0,291
	5	16,25	1,019	2,16	0,8	1,761	1,615	0,18	0,291
	4	13,00	0,972	2,16	0,8	1,680	1,615	0,18	0,291
	3	9,75	0,915	2,16	0,8	1,581	1,615	0,18	0,291
	2	6,50	0,840	2,16	0,8	1,452	1,615	0,18	0,291
	1	3,25	1,002	2,16	0,8	1,731	1,615	0,18	0,291
Leeward	-	all	1,615	2,16	-0,5	-1,745	1,615	0,18	0,291
Side	-	all	1,615	2,16	-0,7	-2,442	1,615	0,18	0,291

Table 3.7: Wind forces in N-S direction.

Level	Height above ground level z [m]	Tributary height	Windward		Leeward		Total design wind force [kN]
			External design wind force $g z \cdot G_f \cdot C_p$	Design wind force P [kN]	External design wind force $g h \cdot G_f \cdot C_p$	Design wind force P [kN]	
45	145,00	0,375	2,791	45,012	-1,745	28,132	73,144
44	143,00	3,25	2,783	388,962	-1,745	243,813	632,775
43	139,75	3,25	2,770	387,084	-1,745	243,813	630,897
42	136,50	3,25	2,756	385,171	-1,745	243,813	628,984
41	133,25	3,25	2,742	383,222	-1,745	243,813	627,035
40	130,00	3,25	2,728	381,235	-1,745	243,813	625,048
39	126,75	3,25	2,713	379,209	-1,745	243,813	623,022
38	123,50	3,25	2,699	377,140	-1,745	243,813	620,954
37	120,25	3,25	2,684	375,029	-1,745	243,813	618,842
36	117,00	3,25	2,668	372,872	-1,745	243,813	616,685
35	113,75	3,25	2,652	370,667	-1,745	243,813	614,480
34	110,50	3,25	2,636	368,412	-1,745	243,813	612,225
33	107,25	3,25	2,620	366,104	-1,745	243,813	609,917
32	104,00	3,25	2,603	363,740	-1,745	243,813	607,553
31	100,75	3,25	2,585	361,317	-1,745	243,813	605,130
30	97,50	3,25	2,568	358,831	-1,745	243,813	602,644
29	94,25	3,25	2,549	356,279	-1,745	243,813	600,092
28	91,00	3,25	2,531	353,657	-1,745	243,813	597,470
27	87,75	3,25	2,511	350,959	-1,745	243,813	594,773
26	84,50	3,25	2,491	348,182	-1,745	243,813	591,995
25	81,25	3,25	2,471	345,319	-1,745	243,813	589,132
24	78,00	3,25	2,450	342,364	-1,745	243,813	586,177
23	74,75	3,25	2,428	339,310	-1,745	243,813	583,123
22	71,50	3,25	2,405	336,149	-1,745	243,813	579,963
21	68,25	3,25	2,382	332,873	-1,745	243,813	576,687
20	65,00	3,25	2,358	329,472	-1,745	243,813	573,285
19	61,75	3,25	2,332	325,933	-1,745	243,813	569,746
18	58,50	3,25	2,306	322,244	-1,745	243,813	566,057
17	55,25	3,25	2,278	318,390	-1,745	243,813	562,203
16	52,00	3,25	2,249	314,352	-1,745	243,813	558,165
15	48,75	3,25	2,219	310,110	-1,745	243,813	553,923
14	45,50	3,25	2,187	305,638	-1,745	243,813	549,451
13	42,25	3,25	2,153	300,906	-1,745	243,813	544,720
12	39,00	3,25	2,117	295,878	-1,745	243,813	539,691
11	35,75	3,25	2,079	290,508	-1,745	243,813	534,321
10	32,50	3,25	2,037	284,737	-1,745	243,813	528,550
9	29,25	3,25	1,993	278,490	-1,745	243,813	522,304
8	26,00	3,25	1,944	271,670	-1,745	243,813	515,483
7	22,75	3,25	1,890	264,139	-1,745	243,813	507,952
6	19,50	3,25	1,830	255,705	-1,745	243,813	499,518
5	16,25	3,25	1,761	246,076	-1,745	243,813	489,889
4	13,00	3,25	1,680	234,783	-1,745	243,813	478,596
3	9,75	3,25	1,581	220,985	-1,745	243,813	464,799
2	6,50	3,25	1,452	202,905	-1,745	243,813	446,718
1	3,25	4,875	1,731	362,946	-1,745	365,720	728,665
							25382,782

Table 3.8: Design wind pressure in E-W direction.

Location	Level	Height above ground level z [m]	External pressure				Internal pressure		
			q [kN/m ²]	G_f	C_p	$q \cdot G_f \cdot C_p$	q_i [kN/m ²]	$G \cdot C_{pi}$ [+, -]	$q_i \cdot G \cdot C_{pi}$ [+, -]
Windward	45	145,00	1,615	2,18	0,8	2,817	1,615	0,18	0,291
	44	143,00	1,611	2,18	0,8	2,809	1,611	0,18	0,290
	43	139,75	1,603	2,18	0,8	2,795	1,603	0,18	0,289
	42	136,50	1,595	2,18	0,8	2,782	1,595	0,18	0,287
	41	133,25	1,587	2,18	0,8	2,768	1,587	0,18	0,286
	40	130,00	1,579	2,18	0,8	2,753	1,579	0,18	0,284
	39	126,75	1,570	2,18	0,8	2,739	1,570	0,18	0,283
	38	123,50	1,562	2,18	0,8	2,724	1,562	0,18	0,281
	37	120,25	1,553	2,18	0,8	2,708	1,553	0,18	0,280
	36	117,00	1,544	2,18	0,8	2,693	1,544	0,18	0,278
	35	113,75	1,535	2,18	0,8	2,677	1,535	0,18	0,276
	34	110,50	1,526	2,18	0,8	2,661	1,526	0,18	0,275
	33	107,25	1,516	2,18	0,8	2,644	1,516	0,18	0,273
	32	104,00	1,506	2,18	0,8	2,627	1,506	0,18	0,271
	31	100,75	1,496	2,18	0,8	2,609	1,496	0,18	0,269
	30	97,50	1,486	2,18	0,8	2,591	1,486	0,18	0,267
	29	94,25	1,475	2,18	0,8	2,573	1,475	0,18	0,266
	28	91,00	1,464	2,18	0,8	2,554	1,464	0,18	0,264
	27	87,75	1,453	2,18	0,8	2,535	1,453	0,18	0,262
	26	84,50	1,442	2,18	0,8	2,515	1,442	0,18	0,260
	25	81,25	1,430	2,18	0,8	2,494	1,430	0,18	0,257
	24	78,00	1,418	2,18	0,8	2,473	1,418	0,18	0,255
	23	74,75	1,405	2,18	0,8	2,450	1,405	0,18	0,253
	22	71,50	1,392	2,18	0,8	2,428	1,392	0,18	0,251
	21	68,25	1,378	2,18	0,8	2,404	1,378	0,18	0,248
	20	65,00	1,364	2,18	0,8	2,379	1,364	0,18	0,246
	19	61,75	1,350	2,18	0,8	2,354	1,350	0,18	0,243
	18	58,50	1,334	2,18	0,8	2,327	1,334	0,18	0,240
	17	55,25	1,318	2,18	0,8	2,299	1,318	0,18	0,237
	16	52,00	1,302	2,18	0,8	2,270	1,302	0,18	0,234
	15	48,75	1,284	2,18	0,8	2,240	1,284	0,18	0,231
	14	45,50	1,266	2,18	0,8	2,207	1,266	0,18	0,228
	13	42,25	1,246	2,18	0,8	2,173	1,246	0,18	0,224
	12	39,00	1,225	2,18	0,8	2,137	1,225	0,18	0,221
	11	35,75	1,203	2,18	0,8	2,098	1,203	0,18	0,217
	10	32,50	1,179	2,18	0,8	2,056	1,179	0,18	0,212
	9	29,25	1,153	2,18	0,8	2,011	1,153	0,18	0,208
	8	26,00	1,125	2,18	0,8	1,962	1,125	0,18	0,202
	7	22,75	1,094	2,18	0,8	1,908	1,094	0,18	0,197
	6	19,50	1,059	2,18	0,8	1,847	1,059	0,18	0,191
	5	16,25	1,019	2,18	0,8	1,777	1,019	0,18	0,183
	4	13,00	0,972	2,18	0,8	1,696	0,972	0,18	0,175
	3	9,75	0,915	2,18	0,8	1,596	0,915	0,18	0,165
	2	6,50	0,840	2,18	0,8	1,465	0,840	0,18	0,151
	1	3,25	1,002	2,18	0,8	1,747	1,002	0,18	0,180
Leeward	-	all	1,615	2,18	-0,35	-1,233	1,615	0,18	0,291
Side	-	all	1,615	2,18	-0,7	-2,465	1,615	0,18	0,291

Table 3.9: Wind forces in E-W direction.

Level	Height above ground level [m]	Tributary height z	Windward		Leeward		Total design wind force [kN]
			External design wind force $g_z \cdot G_f \cdot C_p$	Design wind force P [kN]	External design wind force $g_h \cdot G_f \cdot C_p$	Design wind force P [kN]	
45	145,00	0,375	2,817	25,355	-1,233	11,093	36,448
44	143,00	3,25	2,809	219,105	-1,233	96,139	315,244
43	139,75	3,25	2,795	218,047	-1,233	96,139	314,187
42	136,50	3,25	2,782	216,970	-1,233	96,139	313,109
41	133,25	3,25	2,768	215,872	-1,233	96,139	312,011
40	130,00	3,25	2,753	214,753	-1,233	96,139	310,892
39	126,75	3,25	2,739	213,611	-1,233	96,139	309,750
38	123,50	3,25	2,724	212,446	-1,233	96,139	308,585
37	120,25	3,25	2,708	211,257	-1,233	96,139	307,396
36	117,00	3,25	2,693	210,042	-1,233	96,139	306,181
35	113,75	3,25	2,677	208,800	-1,233	96,139	304,939
34	110,50	3,25	2,661	207,529	-1,233	96,139	303,668
33	107,25	3,25	2,644	206,229	-1,233	96,139	302,368
32	104,00	3,25	2,627	204,897	-1,233	96,139	301,037
31	100,75	3,25	2,609	203,532	-1,233	96,139	299,672
30	97,50	3,25	2,591	202,132	-1,233	96,139	298,271
29	94,25	3,25	2,573	200,695	-1,233	96,139	296,834
28	91,00	3,25	2,554	199,218	-1,233	96,139	295,357
27	87,75	3,25	2,535	197,698	-1,233	96,139	293,837
26	84,50	3,25	2,515	196,134	-1,233	96,139	292,273
25	81,25	3,25	2,494	194,521	-1,233	96,139	290,660
24	78,00	3,25	2,473	192,856	-1,233	96,139	288,995
23	74,75	3,25	2,450	191,136	-1,233	96,139	287,275
22	71,50	3,25	2,428	189,356	-1,233	96,139	285,495
21	68,25	3,25	2,404	187,510	-1,233	96,139	283,649
20	65,00	3,25	2,379	185,594	-1,233	96,139	281,733
19	61,75	3,25	2,354	183,601	-1,233	96,139	279,740
18	58,50	3,25	2,327	181,523	-1,233	96,139	277,662
17	55,25	3,25	2,299	179,351	-1,233	96,139	275,491
16	52,00	3,25	2,270	177,077	-1,233	96,139	273,216
15	48,75	3,25	2,240	174,687	-1,233	96,139	270,826
14	45,50	3,25	2,207	172,168	-1,233	96,139	268,307
13	42,25	3,25	2,173	169,503	-1,233	96,139	265,642
12	39,00	3,25	2,137	166,670	-1,233	96,139	262,810
11	35,75	3,25	2,098	163,645	-1,233	96,139	259,784
10	32,50	3,25	2,056	160,394	-1,233	96,139	256,534
9	29,25	3,25	2,011	156,876	-1,233	96,139	253,015
8	26,00	3,25	1,962	153,034	-1,233	96,139	249,173
7	22,75	3,25	1,908	148,791	-1,233	96,139	244,931
6	19,50	3,25	1,847	144,040	-1,233	96,139	240,180
5	16,25	3,25	1,777	138,616	-1,233	96,139	234,756
4	13,00	3,25	1,696	132,255	-1,233	96,139	228,394
3	9,75	3,25	1,596	124,483	-1,233	96,139	220,622
2	6,50	3,25	1,465	114,298	-1,233	96,139	210,437
1	3,25	4,875	1,747	204,450	-1,233	144,209	348,659
							12460,045

3.4 Seismic Load

For seismic analysis again response spectrum method was chosen. For a given earthquake motion parameters like percentage of critical damping are defined. Response spectrum shows a plot of earthquake-related responses including deflection velocity and acceleration.

3.4.1 Response Spectrum Method

For Istanbul (Turkey):

$$S_s = 1,65$$

$$S_1 = 0,75$$

(According to **Url-1**)

Site class A: Hard rock

Importance factor: (ASCE Table 6-1)

The factor of the building or structure, based on its occupancy type, which functions as an implicit adjustment factor to the return period. Importance factor depends on the category of the structure.

For non-Hurricane Prone Regions importance factor is equal to: $I = 1$

Category of building: (ASCE Table 1-1)

Building and other structures that represent low hazard on human life in the event of failure including (all buildings and other structures except those listed in Category I, II and III) - Category II

Calculated values for response spectrum curve

F_a and F_v are given in IBC Tables 1613.5.3(1) and 1613.5.3(2) and ASCE Tables 11.4.1 and 11.4.2 respectively.

$$F_a = 0,8$$

$$F_v = 0,8$$

$$S_{D1} = \frac{2}{3} \cdot F_a \cdot S_s = \frac{2}{3} \cdot 0,8 \cdot 1,65 = 0,4 \quad (3.20)$$

$$S_{DS} = \frac{2}{3} \cdot F_a \cdot S_v = \frac{2}{3} \cdot 0,8 \cdot 0,75 = 0,88 \quad (3.21)$$

Seismic design category:

According to Table 11.6.1 and 11.6.2 building has to be considered as located in SDC D.

Long-Period Transition Period assumed 12 sec.

In both directions, a building frame system is utilized. In order to satisfy the provisions of ASCE 14.2.2.5 for structures assigned to SDC D, the building frame system must have special reinforced concrete shear walls.

For those system seismic load reduction factor according to behavior of structure is considered as $R = 6$ and the deflection amplification factor $C_d = 5$

Explanation note:

According to the Table 12.2-1 of ASCE 7-10, resisting system should be considered as a dual system with special moment frame capable of resisting at least 25% of prescribed forces, special reinforced concrete shear wall. For this system reduction factor is equal to 7 and deflection amplification factor 5.5. To increase safety margin however, these values were reduced to 6 and 5 respectively.

Mass source:

According to clause 12.7.2 ASCE 7-10 only dead load should be taken into consideration to calculate seismic weight.

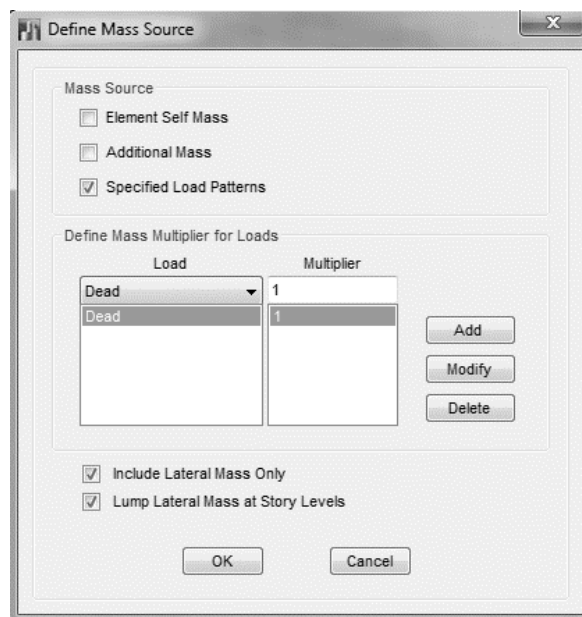


Figure 3.1: Mass source according to ASCE 7-10.

Function Name: Response Spectrum ASCE 7.10

Function Damping Ratio: Damping Ratio: 0,05

Parameters

Ss and S1 from USGS - User Specified

Site Latitude (degrees): ?

Site Longitude (degrees): ?

Site Zip Code (5-Digits): ?

0.2 Sec Spectral Accel, Ss: 1,65

1 Sec Spectral Accel, S1: 0,75

Long-Period Transition Period: 12

Site Class: A

Site Coefficient, Fa: 0,8

Site Coefficient, Fv: 0,8

Calculated Values for Response Spectrum Curve

SDS = (2/3) * Fa * Ss: 0,88

SD1 = (2/3) * Fv * S1: 0,4

Convert to User Defined

Function Graph

Max: (0,090909, 0,88); Min: (10, 0,04)

Function Points

Period	Acceleration
0	0,88
0,0909	0,88
0,4545	0,6667
0,6	0,5
0,8	0,4
1	0,3333
1,2	0,2857
1,4	0,25
1,6	0,2222
1,8	0,2222

Plot Options

☒ Linear X - Linear Y

☐ Linear X - Log Y

☐ Log X - Linear Y

☐ Log X - Log Y

OK Cancel

Figure 3.2: Seismic parameters in ETABS.

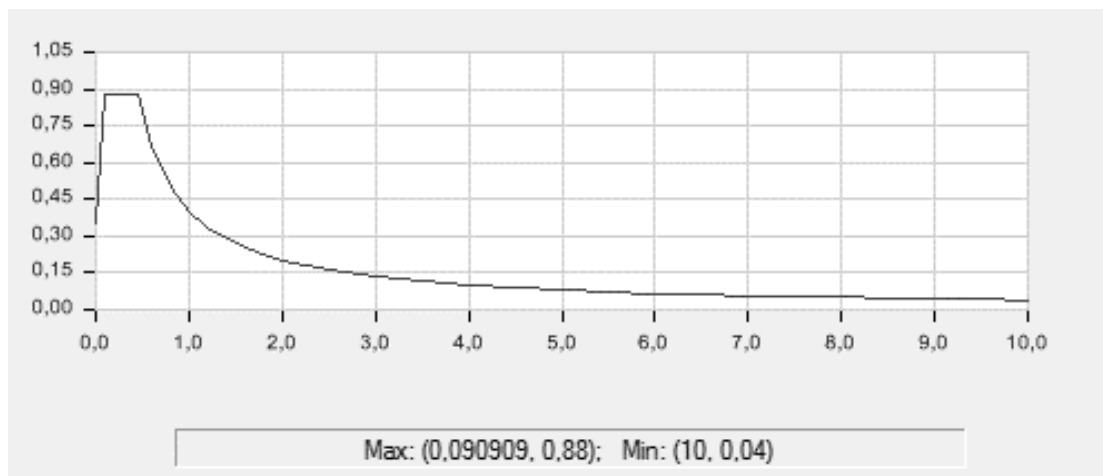


Figure 3.3: Response spectrum curve.

ASCE 11.7.2 states that each structure shall be analyzed for the effects of lateral forces applied independently in each of the two orthogonal directions. For purpose of analysis two direction X and Y were chosen and earthquake forces according to response spectrum method were obtained.

System seismic load reduction factor according to behavior of structure was applied not in the definition of response spectrum curve but later, while earthquake load case was defined. Scale factor 9806,64 were divided into $R = 6$

General

Load Case Name: EqX

Load Case Type: Response Spectrum

Exclude Objects in this Group: Not Applicable

Loads Applied

Load Type	Load Name	Function	Scale Factor
Acceleration	U1	Response Spectru...	1634,44

Other Parameters

Modal Load Case: Modal

Modal Combination Method: CQC

☐ Include Rigid Response

Rigid Frequency, f1:

Rigid Frequency, f2:

Periodic + Rigid Type:

Earthquake Duration, td:

Directional Combination Type: CQC3

Absolute Directional Combination Scale Factor:

Modal Damping: Constant at 0,05

Diaphragm Eccentricity: 0,05 for All Diaphragms

seismic load reduction factor = 6

OK Cancel

Figure 3.4: Seismic load in X direction.

General

Load Case Name: EqY

Load Case Type: Response Spectrum

Exclude Objects in this Group: Not Applicable

Loads Applied

Load Type	Load Name	Function	Scale Factor
Acceleration	U2	Response Spectru...	1634,44

Other Parameters

Modal Load Case: Modal

Modal Combination Method: CQC

☐ Include Rigid Response

Rigid Frequency, f1:

Rigid Frequency, f2:

Periodic + Rigid Type:

Earthquake Duration, td:

Directional Combination Type: CQC3

Absolute Directional Combination Scale Factor:

Modal Damping: Constant at 0,05

Diaphragm Eccentricity: 0,05 for All Diaphragms

OK Cancel

Figure 3.5: Seismic load in Y direction.

3.4.2 Equivalent Lateral Static Load

To fulfill the requirements of the code, results obtained by response spectra method should be compared with results obtained by Equivalent Lateral Static load method. Just simple seismic parameters required by ETABS 2013 and results obtained for this analysis are shown below.

Figure 3.6: ELSL method's seismic parameters in X direction.

Figure 3.7: ELSL method's seismic parameters in Y direction.

Base shear forces obtained by ELSL method:

$$V_{b,x} = 31989 \text{ kN}$$

$$V_{b,y} = 32099 \text{ kN}$$

3.4.3 Scaling design values of combined respond

According to clause 12.9.4.1 ASCE 7-10 : if the modal base shear is less than 85 % of the calculated base shear using second method, the forces should be scaled.

$$\text{Scaling factor} = 0,85 \cdot \frac{V}{V_t} \quad (3.22)$$

V - the equivalent lateral force procedure base shear

V_t - the response spectrum procedure base shear

Table 3.10: Scaling design values of combined respond.

	ASCE7-10 ELSL Results [kN]	ASCE 7-10 RSA Results [kN]	Scalled factor	Scalled Design Values [kN]
EQX	31989	16195	1,679	27190,65
EQY	32099	8723,33	3,128	27284,15

According to these results, scale factor in load case definition of response spectrum method was changed as shown below:

$$1,68 \cdot \frac{9806,64}{6} = 2745,86$$

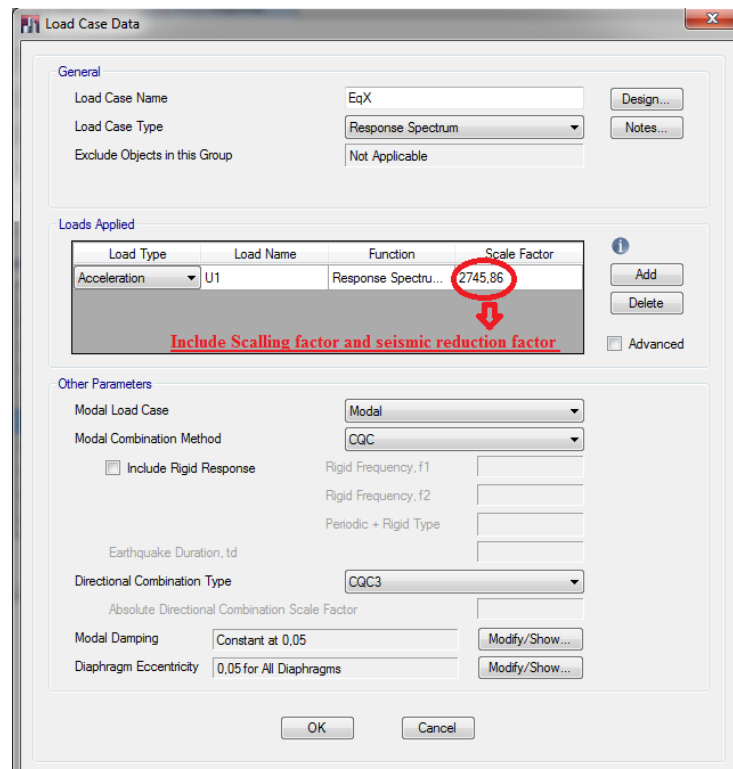


Figure 3.8: Scaling design values of earthquake load case in X direction.

$$3,13 \cdot \frac{9806,64}{6} = 5115,8$$

Load Case Data

General

Load Case Name: EqY [Design...]

Load Case Type: Response Spectrum [Notes...]

Exclude Objects in this Group: Not Applicable

Loads Applied

Load Type	Load Name	Function	Scale Factor
Acceleration	U2	Response Spectru...	5115,8

Include Scalling factor and seismic reduction factor

Other Parameters

Modal Load Case: Modal

Modal Combination Method: CQC

☐ Include Rigid Response

Rigid Frequency, f1: []

Rigid Frequency, f2: []

Periodic + Rigid Type: []

Earthquake Duration, td: []

Directional Combination Type: CQC3

Absolute Directional Combination Scale Factor: []

Modal Damping: Constant at 0,05 [Modify/Show...]

Diaphragm Eccentricity: 0,05 for All Diaphragms [Modify/Show...]

[OK] [Cancel]

Figure 3.9 : Scaling design values of earthquake load case in Y direction.

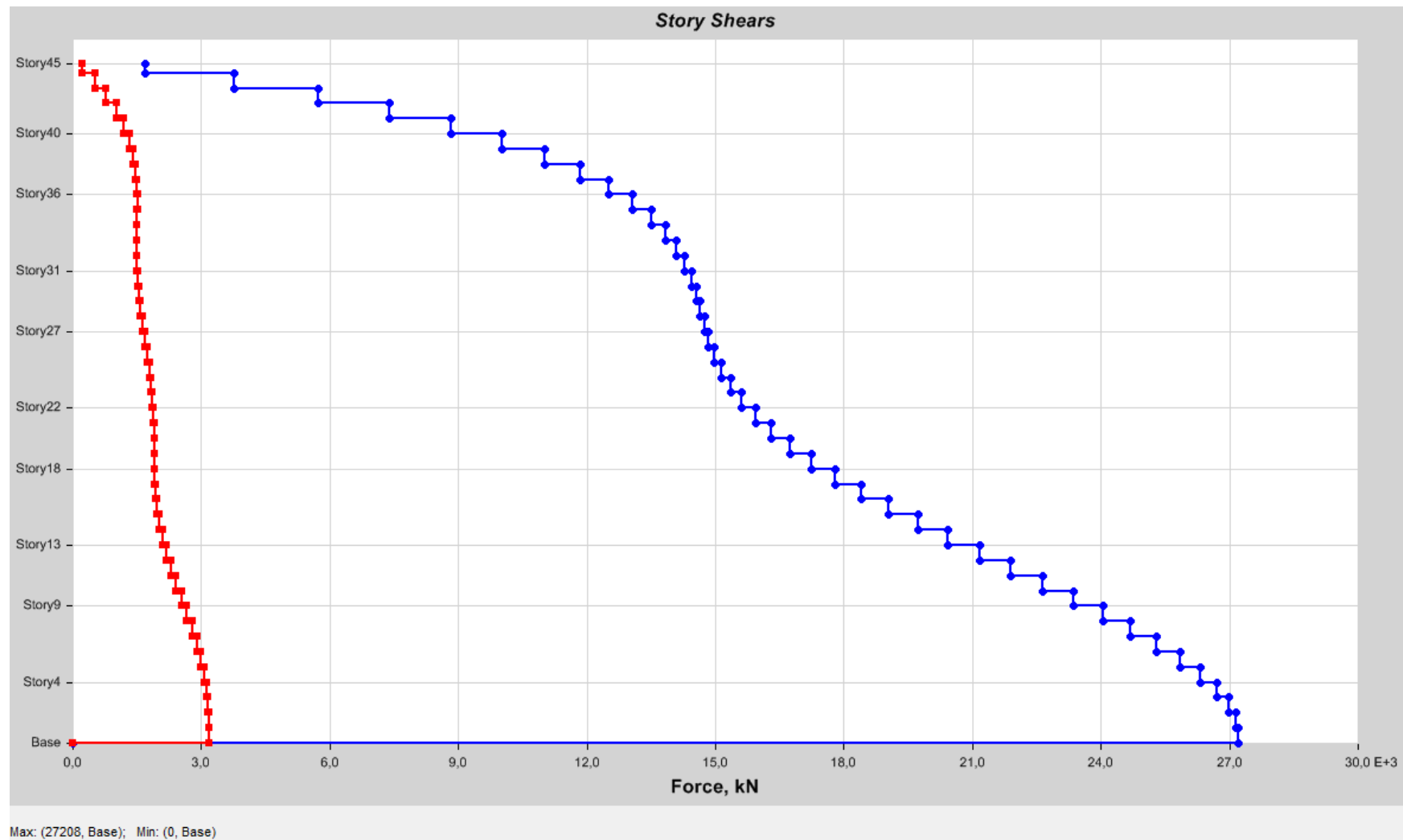


Figure 3.10 : Story shear due to earthquake load in X direction - ASCE 7-10.

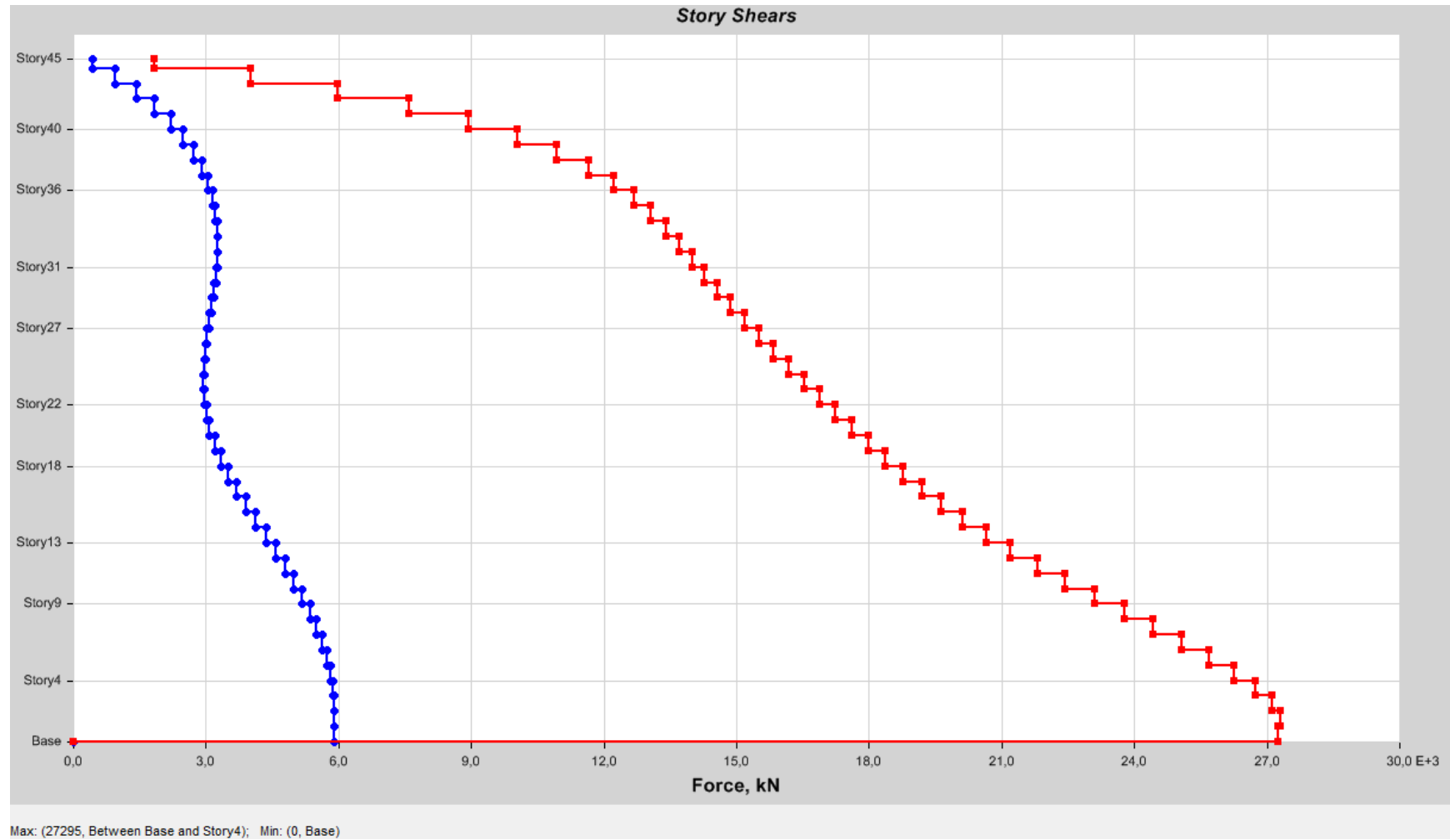


Figure 3.11: Story shear due to earthquake load in Y direction-ASCE 7-10.

3.5 Snow Load

The flat roof snow load shall be calculated using the following formula:

$$p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g \quad (3.23)$$

where: C_e is the exposure coefficient

C_t is the thermal coefficient

I_s is the importance factor

p_g is the ground snow load

Exposure coefficient C_e according to Table 7.2 for terrain category B in terms of full exposure of the roof is equal:

$$C_e = 0.9$$

Thermal Factor C_t according to Table 7.3 for structure not mentioned as indicated below is equal:

$$C_t = 1$$

Importance factor I_s taken from Table 1.5-2: Importance Factors by Risk Category of Buildings and Other Structures for Snow, is equal:

$$I_s = 0.8$$

Ground snow load taken from TS 498, for a construction located in Istanbul is equal:

$$p_g = 0,75 \text{ kN/m}^2$$

Snow load on the roof:

$$p_f = 0,7 \cdot 0,9 \cdot 1 \cdot 0,8 \cdot 0,75 = 0,378 \text{ kN/m}^2$$

3.6 Load Combinations

Structure shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations, which are called basic load combinations.

$$1) \quad 1.4 D \quad (3.24)$$

$$2) \quad 1.2 D + 1.6 L + 0.5 S \quad (3.25)$$

$$3) \quad 1.2 D + 1.6 S + 0.5 W \quad (3.26)$$

$$1.2 D + 1.6 S + L$$

$$4) \quad 1.2 D + 1.0 W + 1.0 L + 0.5 S \quad (3.27)$$

$$5) \quad 1.2 D + 1.0 E + 1.0 L + 0.2 S \quad (3.28)$$

$$6) \quad 0.9 D + 1.0 W \quad (3.29)$$

$$7) \quad 0.9 D + 1.0 E \quad (3.30)$$

Where: D- dead load

L- Live load

S-Snow load

W- wind load

E- earthquake load

Combinations 5 and 7 include earthquake effects which were applied simultaneously and independently in two orthogonal directions X and Y. The earthquake load is not required in Z direction.

$$E_1 = E_x + 0.3E_y \quad (3.31)$$

$$E_2 = E_x - 0.3E_y \quad (3.32)$$

$$E_3 = 0.3E_x + E_y \quad (3.33)$$

$$E_4 = 0.3E_x - E_y \quad (3.34)$$

$$E_5 = 0.3E_x + 0.3E_y \quad (3.35)$$

$$E_6 = 0.3E_x - 0.3E_y \quad (3.36)$$

For the sake of simplicity the direction Z was omitted and thus, effect of vertical seismic forces was not included in analysis.

$$E = E_h = \rho Q_e + 0,2 S_{DS} \cdot D \quad (3.37)$$

Where: Q_e –effect of horizontal seismic forces

ρ – redundancy factor [1,3 for structures in SDC D]

The load combination 5 and 7 has to be rewrite as follow:

Combination 5:

$$1,2 D + [1,3 (E_x + 0,3 E_y) + 0,2 S_{DS} \cdot D] + 1,0 L + 0,5 S \quad (3.38)$$

$$1,2 D + [1,3 (E_x - 0,3 E_y) + 0,2 S_{DS} \cdot D] + 1,0 L + 0,5 S \quad (3.39)$$

$$1,2 D + [1,3 (0,3 E_x + E_y) + 0,2 S_{DS} \cdot D] + 1,0 L + 0,5 S \quad (3.40)$$

$$1,2 D + [1,3 (0,3 E_x - E_y) + 0,2 S_{DS} \cdot D] + 1,0 L + 0,5 S \quad (3.41)$$

$$1,2 D + [1,3 (0,3 E_x + 0,3 E_y) + 0,2 S_{DS} \cdot D] + 1,0 L + 0,5 S \quad (3.42)$$

$$1,2 D + [1,3 (0,3 E_x - 0,3 E_y) + 0,2 S_{DS} \cdot D] + 1,0 L + 0,5 S \quad (3.43)$$

Combination 7:

$$0,9 D + 1,3 (E_x + 0,3 E_y) + 0,2 S_{DS} \cdot D \quad (3.44)$$

$$0,9 D + 1,3 (E_x - 0,3 E_y) + 0,2 S_{DS} \cdot D \quad (3.45)$$

$$0,9 D + 1,3 (0,3 E_x + E_y) + 0,2 S_{DS} \cdot D \quad (3.46)$$

$$0,9 D + 1,3 (0,3 E_x - E_y) + 0,2 S_{DS} \cdot D \quad (3.47)$$

$$0,9 D + 1,3 (0,3 E_x + 0,3 E_y) + 0,2 S_{DS} \cdot D \quad (3.48)$$

$$0,9 D + 1,3 (0,3 E_x - 0,3 E_y) + 0,2 S_{DS} \cdot D \quad (3.49)$$

4. STRUCTURE DESIGN

4.1 Eurocode 2-2004

4.1.1 Beam Design B46-floor 10

Beam is designed for major direction flexure and shear only.

To choose the most critical beam (B46) in all structure, maximum shear force V_2 and maximum bending moment M_3 were checked in a few station points within beam length for all combinations and for each floor. As a result it can be seen from the figure shown below that beam located at floor 10 were selected for design.

Story	Beam	Load Case/Combo	Station m	P kN	V2 kN	V3 kN	T kN-m	M2 kN-m	M3 kN-m
Story10	B46	Comb11 Min	0	0	-456.9155	0	-42.0616	0	-815.1998
Story10	B46	Comb5 Min	0	0	-456.9152	0	-42.0616	0	-815.199
Story10	B46	DW15 Min	0	0	-456.9143	0	-42.0617	0	-815.1971
Story10	B46	DW16 Min	0	0	-456.9143	0	-42.0617	0	-815.1971
Story11	B46	Comb11 Min	0	0	-456.2053	0	-42.2466	0	-813.8038

Figure 4.1: Maximum shear force in beam B46.

Story	Beam	Load Case/Combo	Station m	P kN	V2 kN	V3 kN	T kN-m	M2 kN-m	M3 kN-m
Story10	B46	Comb19 Min	0	0	-448.3056	0	-44.3749	0	-828.106
Story10	B46	Comb32 Min	0	0	-448.3056	0	-44.3749	0	-828.106
Story10	B46	Comb20 Min	0	0	-448.3056	0	-44.3749	0	-828.106
Story16	B46	Comb17 Min	0	0	-447.8669	0	-44.6003	0	-827.4081
Story16	B46	Comb18 Min	0	0	-447.8669	0	-44.6003	0	-827.4081

Figure 4.2: Maximum bending moment in beam B46.

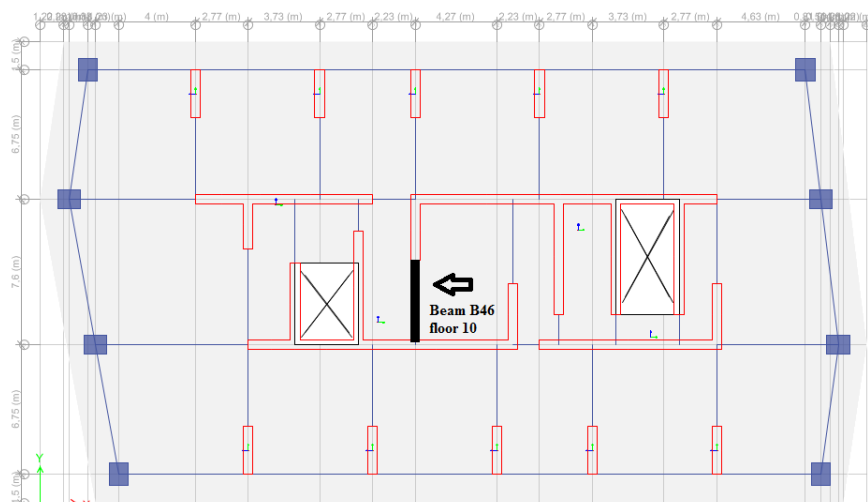


Figure 4.3: Location of beam B46.

4.1.1.1 Flexure design

Concrete: C40/50, Steel: A615Gr60

Reinforcing Yield Strength $f_y = 413,69 \text{ MPa}$

Concrete Compressive Strength $f'_c = 40 \text{ MPa}$ $f_{ctm} = 3,5 \text{ MPa}$

Beam Width $b = 50 \text{ cm}$

Depth to Tension Reinforcing $d = 76 \text{ cm}$

Total Beam Depth $h = 80 \text{ cm}$

All beams are designed for major direction flexure and shear only.

Factored moment at beam section is given by factoring the moments for load cases with the design load combination factors. Positive moments are responsible for bottom steel and negative moments for top reinforcement.

The effective strength and depth of the compression zone is given by:

(EC2 Equation 3.19)

$$\lambda = 0,8 \text{ for } f_{ck} \leq 50 \quad (4.1)$$

(EC2 Equation 3.21)

$$\eta = 1 \text{ for } f_{ck} \leq 50 \text{ MPa} \quad (4.2)$$

The reinforcing steel area is determined based on whether the normalized moment is greater less or equal to so called normalized section capacity for singly reinforced beam.

Design concrete compressive strength

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} = 1 \cdot \frac{40}{1,5} = 26,67 \text{ MPa} \quad (4.3)$$
$$f_{ck} = 40 \text{ MPa}$$

Material coefficient taking account of long-term effects on the compressive strength (EC2 3.1.6)

$$\alpha_{cc} = 1$$

Material partial factor for concrete

(EC2 2.4.2.4)

$$\gamma_c = 1,5$$

Design yield strength of reinforcement

(EC2 3.2)

$$f_{yd} = \frac{f_{yk}}{\gamma_s} = \frac{413,69}{1,15} = 359,73 \text{ MPa} \quad (4.4)$$

For a singly reinforced section, subjected only to design moment M , these assumptions are represented in the following diagram:

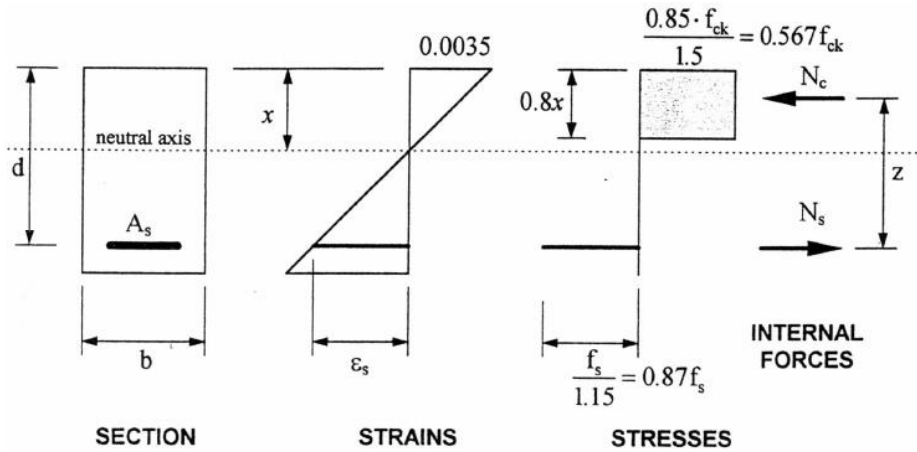


Figure 4.4: Rectangular beam design.

Normalised bending resistance:

$$K = \frac{M_{Ed}}{b \cdot d^2 \cdot \eta \cdot f_{ck}} \quad (4.5)$$

K' –Maximum value of M (with no compression steel and when distance from edge to neutral axis after redistribution is at maximum value).

$$K' = 0,6\delta - 0,18\delta^2 - 0,21 \quad (4.6)$$

If $K \leq K'$ no compression steel needed-singly reinforced beam

Required steel:

$$A_s = \frac{M_{Ed}}{f_{yd} \cdot z} \quad (4.7)$$

Level arm z :

$$z = \frac{d}{2} \left[1 + \sqrt{1 - 3,53K} \right] \leq 0,95 d \quad (4.8)$$

Level of $0,95d$ is considered as a good practice, but is not required in EC2.

The minimum flexural tensile steel reinforcement, required in a beam section is given as the maximum of the following two values:

(EC2 Equation 9.1.N)

$$A_{s,min} = 0,26 \left(\frac{f_{ctm}}{f_{yk}} \right) \cdot b_t \cdot d \quad (4.9)$$

$$A_{s,min} = 0,0013 \cdot b_t \cdot d \quad (4.10)$$

(EC2 Table 3.1)

$$f_{ctm} = 0,30 f_{ck}^{\frac{2}{3}} \text{ for } f_{ck} \leq 50 \text{ MPa} \quad (4.11)$$

The maximum flexural steel reinforcement, permitted as either tension or compression reinforcement is defined as:

(EC2 9.2.1.1.(3))

$$A_{s,max} = 0,04 \cdot A_c \quad (4.12)$$

A_c – gross cross sectional area

Critical combination : 19 (D + 0,3 L + 0,3 Ex – Ey - 0,3 Ez)

Station location: $l = 0,0$

- Top steel: $M_{Ed3} = -825,6 \text{ kNm}$

$$K = \frac{825,6 \cdot 10^6}{500 \cdot 760^2 \cdot 1 \cdot 40} = 0,0715$$

$\delta = 1$ when there is no redistribution

$$K' = 0,208$$

$K \leq K'$ no compression steel needed-singly reinforced beam
 $0,0715 \leq 0,208$

Level arm z :

$$z = \frac{d}{2} \left[1 + \sqrt{1 - 3,53K} \right] \leq 0,95 d$$

$$z = \frac{760}{2} \left[1 + \sqrt{1 - 3,53 \cdot 0,0715} \right] = 708,56 \text{ mm} \leq 0,95 d = 722 \text{ mm}$$

Required steel:

$$A_s = \frac{M_{Ed}}{f_{yd} \cdot z}$$

$$A_s = \frac{825,6 \cdot 10^6}{359,73 \cdot 708,56} = 3239,04 \text{ mm}^2 = 32,39 \text{ cm}^2$$

Reinforcement applied:

$3\phi 26 + 4\phi 26$

$$A_{s,top} = \frac{7 \cdot \pi \cdot 2.6^2}{4} = 37,16 \text{ cm}^2$$

- Bottom steel: $M_{Ed3} = 756,31 \text{ kNm}$

$$M_{Ed3} = 756,31 \text{ kNm}$$

$$K = \frac{756,31 \cdot 10^6}{500 \cdot 760^2 \cdot 1 \cdot 40} = 0,0656$$

$\delta = 1$ when there is no redistribution

$$K' = 0,208$$

$K \leq K'$ no compression steel needed-singly reinforced beam
 $0,0656 \leq 0,208$

Level arm z:

$$z = \frac{d}{2} \left[1 + \sqrt{1 - 3,53K} \right] \leq 0,95 d$$

$$z = \frac{760}{2} \left[1 + \sqrt{1 - 3,53 \cdot 0,0656} \right] = 713,11 \text{ mm} \leq 0,95 d = 722 \text{ mm}$$

Required steel:

$$A_s = \frac{M_{Ed}}{f_{yd} \cdot z}$$

$$A_s = \frac{756,31 \cdot 10^6}{359,73 \cdot 713,11} = 2948,27 \text{ mm}^2$$

Reinforcement applied:

$3\phi 26 + 4\phi 22$

$$A_{s,bottom} = \frac{3 \cdot \pi \cdot 2.6^2}{4} + \frac{4 \cdot \pi \cdot 2.2^2}{4} = 31,13 \text{ cm}^2$$

Station location: $l = 2,4 \text{ m}$

- Bottom steel: $M_{Ed3} = -206,4 \text{ kNm}$

$$K = \frac{206,4 \cdot 10^6}{500 \cdot 760^2 \cdot 1 \cdot 40} = 0,0179$$

$\delta = 1$ when there is no redistribution

$$K' = 0,208$$

$K \leq K'$ no compression steel needed-singly reinforced beam

$$0,0656 \leq 0,208$$

Level arm z :

$$z = \frac{d}{2} \left[1 + \sqrt{1 - 3,53K} \right] \leq 0,95 d$$

$$z = \frac{760}{2} \left[1 + \sqrt{1 - 3,53 \cdot 0,0179} \right] = 747,8 \text{ mm}$$

Required steel:

$$A_s = \frac{M_{Ed}}{f_{yd} \cdot z}$$

$$A_s = \frac{206,4 \cdot 10^6}{359,73 \cdot 747,8} = 7,67 \text{ cm}^2$$

Reinforcement applied:

$3\phi 26$

$$A_{s,bottom} = \frac{3 \cdot \pi \cdot 2,6^2}{4} = 15,93 \text{ cm}^2$$

Station location: $l = 4,4$

- Top steel: $M_{Ed3} = -747,3 \text{ kNm}$

$$K = \frac{747,3 \cdot 10^6}{500 \cdot 760^2 \cdot 1 \cdot 40} = 0,065$$

$\delta = 1$ when there is no redistribution

$$K' = 0,208$$

$K \leq K'$ no compression steel needed-singly reinforced beam

$$0,0715 \leq 0,208$$

Level arm z :

$$z = \frac{d}{2} \left[1 + \sqrt{1 - 3,53K} \right] \leq 0,95 d$$
$$z = \frac{760}{2} \left[1 + \sqrt{1 - 3,53 \cdot 0,065} \right] = 713,57 \text{ mm}$$

Required steel:

$$A_s = \frac{M_{Ed}}{f_{yd} \cdot z}$$
$$A_s = \frac{747,3 \cdot 10^6}{359,73 \cdot 713,57} = 2911,27 \text{ mm}^2$$

Reinforcement applied:

$3\phi 26 + 4\phi 26$

$$A_{s,top} = \frac{7 \cdot \pi \cdot 2.6^2}{4} = 37,16 \text{ cm}^2$$

- Bottom steel: $M_{Ed3} = 659,4 \text{ kNm}$

$$K = \frac{659,4 \cdot 10^6}{500 \cdot 760^2 \cdot 1 \cdot 40} = 0,0571$$

$\delta = 1$ when there is no redistribution

$$K' = 0,208$$

$K \leq K'$ no compression steel needed-singly reinforced beam
 $0,0656 \leq 0,208$

Level arm z :

$$z = \frac{d}{2} \left[1 + \sqrt{1 - 3,53K} \right] \leq 0,95 d$$
$$z = \frac{760}{2} \left[1 + \sqrt{1 - 3,53 \cdot 0,0571} \right] = 719,55 \text{ mm}$$

Required steel:

$$A_s = \frac{M_{Ed}}{f_{yd} \cdot z}$$
$$A_s = \frac{659,4 \cdot 10^6}{359,73 \cdot 719,55} = 2547 \text{ mm}^2$$

Reinforcement applied:

$3\phi 26 + 2\phi 26$

$$A_{s,top} = \frac{5 \cdot \pi \cdot 2.6^2}{4} = 26,54 \text{ cm}^2$$

Minimum reinforcement requirements:

$$A_{s,min} = \max \left\{ \begin{array}{l} 0,26 \left(\frac{f_{ctm}}{f_{yk}} \right) \cdot b_t \cdot d \\ 0,0013 \cdot b_t \cdot d \end{array} \right.$$

$$A_{s,min} = \max \left\{ \begin{array}{l} 0,26 \left(\frac{0,35}{41,369} \right) \cdot 50 \cdot 76 = 8,359 \text{ cm}^2 \\ 0,0013 \cdot 50 \cdot 76 = 4,94 \text{ cm}^2 \end{array} \right.$$

$$A_{s,max} = 0,04 \cdot A_c = 0,04 \cdot 80 \cdot 50 = 160 \text{ cm}^2$$

Placement details:

Placement of longitudinal reinforcement was due to the results obtained with the help of ETABS.

Top reinforcement in the middle part of the beam ($3\phi 26 - \text{Area: } 15,92 \text{ cm}^2$) is provided between station point 1,2 m to 3,2 m. End parts are designed as ($3\phi 26 + 2\phi 26$).

Bottom reinforcement in the middle part of the beam ($3\phi 26 - \text{Area: } 15,96 \text{ cm}^2$) is provided between station point 1,2 m to 3,2 m. End parts are designed as ($3\phi 26 + 4\phi 22$ or $3\phi 26 + 2\phi 26$).

According to these results, length of the end reinforcement was taken as

$1,5(l_b + h)$

Comb. ID	STATION LOC	TOP STEEL	BOTTOM STEEL	SHEAR STEEL
Comb19	0,0000	3202	2917	1816,85
Comb19	0,4000	2477	2345	1804,18
Comb19	0,8000	1777	1780	1791,50
Comb19	1,2000	1400	1400	1778,82
Comb19	1,2000	1400	1400	1412,99
Comb19	1,6000	1400	1400	1400,32
Comb19	2,0000	1400	1400	1390,05
Comb19	2,4000	1400	1400	1402,72
Comb19	2,8000	1400	1400	1415,40
Comb19	2,8000	1400	1400	1514,36
Comb19	3,2000	1400	1400	1527,04
Comb19	3,6000	1777	1685	1539,72
Comb19	4,0000	2320	2104	1552,39
Comb19	4,4000	2881	2525	1565,07
Comb19	4,4000	2881	2525	1565,07

Figure 4.5: Beam sectioning.

4.1.1.2 Shear design

V_{Ed} Factored shear force at a section

$V_{Rd,c}$ Design shear resistance without shear reinforcement 1

$V_{Rd,max}$ Shear force that can be carried without crushing of the notional concrete compressive struts

Required shear reinforcement in the form of stirrups or ties per unit spacing is calculated according to Eq.6.8 of EC2:

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot \cot \phi} \quad (4.13)$$

$$z = 0,9 d \quad (4.14)$$

The calculated shear reinforcement has to greater than the minimum reinforcement ratio (EC2 Eq.9.5N)

$$\rho_{w,min} = \frac{(0,08 \cdot \sqrt{f_{ck}})}{f_{yk}} \quad (4.15)$$

Critical combination : 19 (D + 0,3 L + 0,3 Ex – Ey - 0,3 Ez)

- Station location: $l = 0 \text{ mm}$

Shear force and shear reinforcement for V_{Ed2}

Factored shear force at section:

$$V_{Ed} = 447,04 \text{ kN}$$

Shear force that can be carried without requiring shear reinforcement

$$V_{Rd,c} = 222,86 \text{ kN}$$

Shear force that will be carried by transverse reinforcement by assuming that concrete shear capacity will not contribute to total shear capacity of the system:

$$V_{Rd,s} = 447,04 \text{ kN}$$

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot \cot \phi} = \frac{447,04}{0,9 \cdot 76 \cdot 35,973 \cdot \cot 45^\circ} = 1817 \frac{\text{mm}^2}{\text{m}}$$

$$z = 0,9 d = 0,9 \cdot 76 = 68,4 \text{ cm}$$

$$f_{ywd} = 359,73 \text{ MPa}$$

Provided:

$$s = 10 \text{ cm}, 20 \text{ cm}, 25 \text{ cm}$$

$$A_v = 2 \cdot \frac{\pi \cdot 1^2}{4} = 1,57 \text{ cm}^2$$

$$A_v = 2 \cdot \frac{\pi \cdot 1,2^2}{4} + 2 \cdot \frac{\pi \cdot 1^2}{4} = 3,83 \text{ cm}^2$$

For total length: 4,4 m there is 112,14 cm² (26 ties · 3,83 cm² + 8 ties · 1,57 cm²) of transversal reinforcement. It is greater than required amount 80 cm² (4,4 · 18,17).

Reinforcement requirements:

- Transverse reinforcement:

$$s_{l,max} = 0,75 \cdot d \cdot (1 + \cot \alpha) = 0,75 d \quad (4.16)$$

$$\rho_{w,min} = \frac{0,08 \cdot \sqrt{f_{ck}}}{f_{yk}} \rightarrow \rho_{w,min} = 0,0008 \quad (4.17)$$

Unsuprisingly ,critical column (C2) was chosen at the first floor. To confirm it maximum axil force P , shear forces and maximum bending moments were checked in a few station points within column length for all combinations and for each floor.

Figure 4.6: Maximum axil force in column C2.

Figure 4.7: Maximum bending moment in column C2.



4.1.2.1 Flexure design

Interaction diagram is a graph which illustrates the capacity of concrete element to resist a range of combinations of moments and axial force. When the location of neutral axis is changed, size of compression and tension zone is changing as well. Differences in strain distribution lead to different capacity of element.

To measure the load conditions of the column, usually capacity ratio is calculated. It takes into account load conditions with respect to the capacity of the column.

The volume interaction surface is described by points generated on the interaction failure surface.

The point with coordinates of internal forces after loading is placed in the 3D interaction space. If the point lies within the interaction volume, the column is designed properly. Although if the point lies outside the interaction volume, the column is overloaded.

ETABS has generated axial force biaxial moment interaction surfaces. For each of the load combinations, resultant forces were shown as a point on the interaction space.

Design internal forces:

Combination 5 : (1,35D+1,5W+1,05L+0,75S)

$$N_{Ed} = 28581,2 \text{ kN}$$

$$M_{Ed2} = -952,71 \text{ kNm}$$

$$M_{Ed3} = -1345,2 \text{ kNm}$$

This item is used for ductility considerations in seismic design. Program determined value to the highest ductility requirement. Etabs 2013 reports value of 120 cm^2 of longitudinal reinforcement. This is the minimum required limit shown as follows:

Minimum longitudinal reinforcement limits:

$$A_{st,min} = 0,01 \cdot A_g = 0,01 \cdot 120 \cdot 100 = 120 \text{ cm}^2 \quad (4.18)$$

28 bars $d = 25 \text{ mm}$ were chosen

$$A_{s,provided} = 137,44 \text{ cm}^2$$

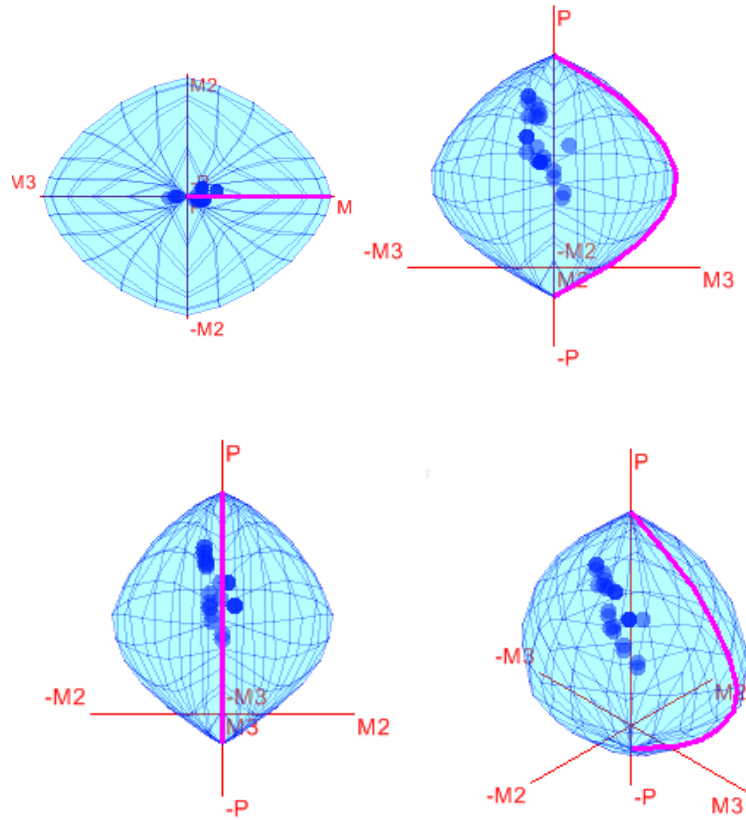


Figure 4.9: Interaction surface for column C2.

All points which represent the internal forces in each combination are inside the volume limited by critical curvature. It means that design of reinforcement fulfil the requirements.

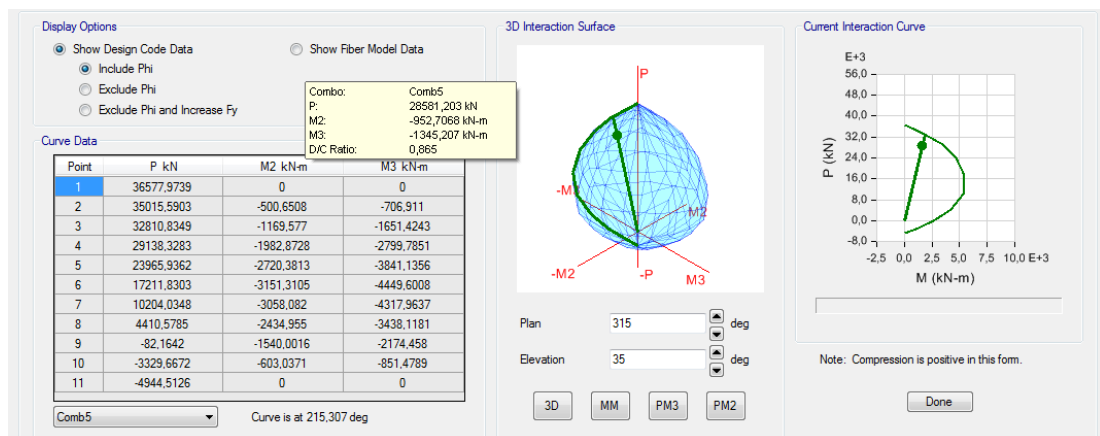


Figure 4.10: Interaction surface for column C2.

For the critical combination 5 , D/C ratio is equal to 0,865. That value is reduced because of assumption of high ductility class of RC element.

High class (DC high) corresponds to structures designed , dimensioned so that the response of sturcture is according to the considered failure mechanism, with a large amount of energy dissipated.

4.1.2.2 Shear design

To be able to calculate shear reinforcement, listed above values has to be calculated or taken from analysis results.

V_{Ed}	Factored shear force at a section
$V_{Rd,c}$	Design shear resistance without shear reinforcement
$V_{Rd,max}$	Shear force that can be carried without crushing of the notional concrete compressive struts

Maximum sustainable design shear force:

The angle between the concrete compression strut and the column axis perpendicular to the shear force is defined as ϕ and is assumed by program as the conservative value of 45 degrees.

$$V_{Rd,max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}}{(\cot\phi + \tan\phi)} \quad (4.19)$$

$$v_1 = 0,6 \cdot \left[1 - \frac{f_{ck}}{250} \right] = 0,6 \cdot \left[1 - \frac{40}{250} \right] = 0,504 \quad (4.20)$$

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} = 1 \cdot \frac{40}{1,5} = 26,67 \text{ MPa}$$

Minor shear V_{Ed2} :

$$z = 0,9 d = 0,9 \cdot 113,1 = 101,79 \text{ cm}$$

$$V_{Rd,max} = \frac{1 \cdot 100 \cdot 101,79 \cdot 0,504 \cdot 26,67}{(\cot 45 + \tan 45)} = 6841,14 \text{ kN}$$

Major shear V_{Ed3} :

$$z = 0,9 d = 0,9 \cdot 93,1 = 83,79 \text{ cm}$$

$$V_{Rd,max} = \frac{1 \cdot 100 \cdot 83,79 \cdot 0,504 \cdot 2,667}{(\cot 45 + \tan 45)} = 5631,39 \text{ kN}$$

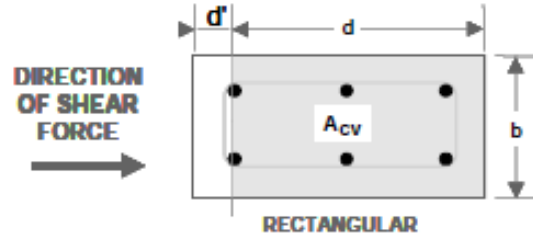


Figure 4.11: Direction of shear force .

Required shear reinforcement in the form of stirrups or ties per unit spacing is calculated according to Eq.6.8 of EC2:

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot \cot \phi}$$

$$z = 0,9 d$$

The calculated shear reinforcement has to be greater than the minimum reinforcement ratio (EC2 Eq.9.5N)

$$\rho_{w,min} = \frac{(0,08 \cdot \sqrt{f_{ck}})}{f_{yk}} \quad (4.21)$$

Shear forces for V_{Ed2} :

$$V_{Ed} = 243,24 \text{ kN}$$

$$V_{Rd,c} = 1427,05 \text{ kN}$$

Column Design Forces

244

z 360

Reload

Apply

Story	Column	Load Combo	Station m	P kN	V2 kN	V3 kN
Story1	C2	Comb11-4	0	-28585,3713	-243,2417	20,1055

Figure 4.12: Shear force V_2 in column C2.

Shear forces for V_{Ed3} :

$$V_{Ed} = 139,77 \text{ kN}$$

$$V_{Rd,c} = 1425,33 \text{ kN}$$

Column Design Forces							
214		z 360		Reload Apply			
	Story	Column	Load Combo	Station m	P kN	V2 kN	V3 kN
	Story1	C2	Comb13-1	0	-7671,6606	107,1979	139,7658

Figure 4.13: Shear force V_3 in column C2.

If V_{Ed} is greater than V_{Rdc} and less than $V_{Rd,max}$ the required shear reinforcement in the form of stirrups or ties per spacing is calculated as:

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot \cot \phi}$$

The maximum of all of the calculated $\frac{A_{sw}}{s}$ values, obtained from each design load combination, is reported for the major and minor directions of the column, along with the controlling combination name.

$$f_{ywd} = \frac{f_{yk}}{1,15} = \frac{413,69}{1,15} = 359,73 \text{ MPa}$$

Shear reinforcement for V_{Ed2} :

$$\frac{A_{sw}}{s} = \frac{243,24 \cdot 10^4}{101,79 \cdot 35,973 \cdot \cot(45^\circ)} = 664,28 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 15 \text{ cm}$$

$$A_v = 5 \cdot \frac{\pi \cdot 0,8^2}{4} = 2,51 \text{ cm}^2$$

For total height: 3,25 m there is 55,22 cm² (22 ties · 2,51 cm²) of transversional reinforcement. It is greater than required amount 21,58 cm² (3,25 · 6,64).

Shear reinforcement for V_{Ed3}

$$\frac{A_{sw}}{s} = \frac{139,77 \cdot 10^4}{83,79 \cdot 35,973 \cdot \cot(45^\circ)} = 463,71 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 15 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 0,8^2}{4} = 2,01 \text{ cm}^2$$

For total height: 3,25 m there is 44,22 cm² (22 ties · 2,01 cm²) of transversional reinforcement. It is greater than required amount 15,04 cm² (3,25 · 1,85 cm²).

Detailing arrangements:

Longitudinal bars:

1. Bars should have a diameter of not less than 12mm.
2. A minimum of four bars is required in a rectangular column (one bar in each corner)
3. The minimum amount of total longitudinal reinforcement $A_{s,min}$ should be derived from the following condition :

$$A_{s,min} = \frac{0,10 \cdot N_{sd}}{0,87 f_{yk}} \geq 0,003 A_c \quad (4.22)$$

4. Even at laps, the area of the reinforcement should not exceed the upper limit of 0.08A_c
5. The longitudinal bars should be distributed around the periphery of the section. For bars having a polygonal cross-section, at least one bar shall be placed at each corner.

Transverse reinforcement:

1. The diameter of the transverse reinforcement (links, loops or helical spiral reinforcement) should not be less than 6 mm or one quarter of the maximum diameter of the longitudinal bars, whichever is greater;
2. The transverse reinforcement should be adequately anchored.
3. The spacing of the transverse reinforcement along the column should not exceed the lesser of the following three distances :
 - 20 times the minimum diameter of the longitudinal bars
 - the least dimension of the column
 - 400mm.

$$s \leq \begin{cases} 20 \cdot \phi_{min} = 20 \cdot 12 \text{ mm} = 24 \text{ cm} \\ 100 \text{ cm} \\ 40 \text{ cm} \end{cases}$$

Distance should be reduced to 60% on the length greater than dimension of the column below and above the connection with beam or floor.

4. Every longitudinal bar placed in a corner should be held by transverse reinforcement.
5. The number of transverse reinforcement in cross section of the column should be adjusted to condition that maximum distance between the tie and longitudinal bar should not be greater than 150 mm

4.1.2.3 Splice length of longitudinal reinforcement

According to EC 2, Cl 8.7.3

$$l_{0,min} \geq \begin{cases} 0,3 \cdot l_{b,req} = 0,3 \cdot 24,3 \cdot 2,5 = 18,23 \text{ cm} \\ 200 \text{ mm} \end{cases} \quad (4.23)$$

$$l_{b,req} = \frac{\phi}{4} \cdot \left(\frac{\sigma_{sd}}{f_{bd}} \right) = 24,3 \phi \quad (4.24)$$

$$\sigma_{sd} = \frac{f_{yk}}{\gamma_s} = \frac{413,69}{1,15} = 359,73 \text{ MPa}$$

σ_{sd} – max stress in the bar

$$f_{bd} = 3,7 \text{ MPa}$$

$$l_{bd} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot \alpha_5 \cdot l_{b,req} \quad (4.25)$$

Alpha values according to EC 2, table 8.2:

For reinforcement bar in compression:

- Shape of bar: $\alpha_1 = 1$
- Concrete cover: $\alpha_2 = 1$
- Confinement by transverse reinforcement not welded to main reinforcement:
 $\alpha_3 = 1$
- Confinement by welded transverse reinforcement: $\alpha_4 = 0,7$
- Confinement by transverse pressure: $\alpha_5 = 1$

$$l_{bd} = 1 \cdot 1 \cdot 1 \cdot 0,7 \cdot 1 \cdot 24,3 \cdot 2,5 = 42,5 \text{ cm}$$

Splice length will be taken as the width of cross section of the column, equal to 100 cm.

4.1.3 Shear Wall Design, Core 4- floor 1

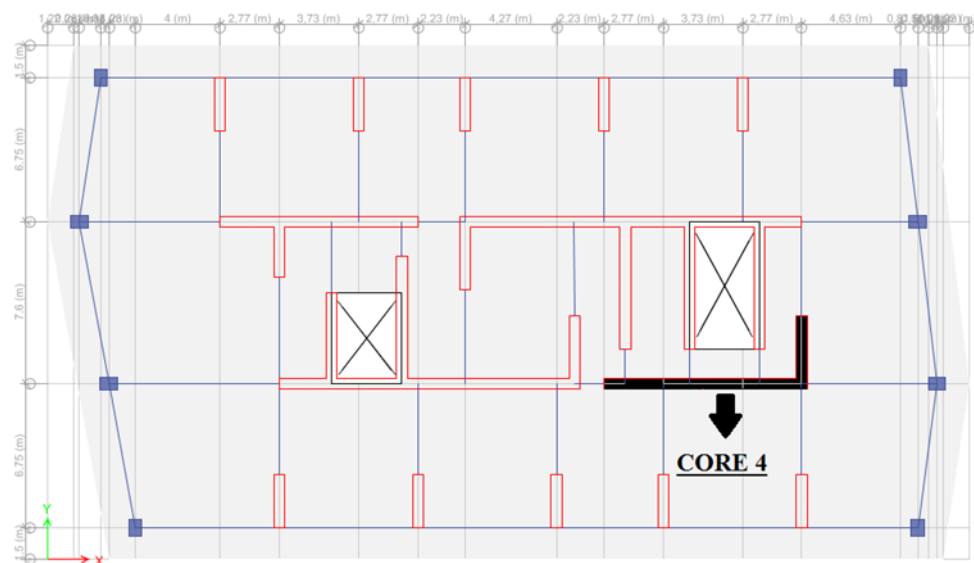


Figure 4.14: Location of Core 4.

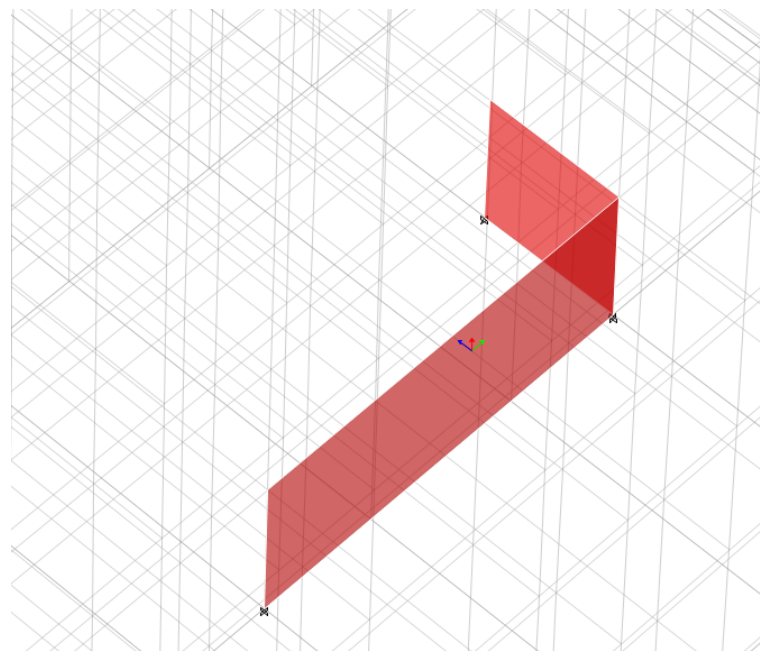


Figure 4.15: Location of Core 4.

GENERAL GEOMETRY:

LEG 2: L=9,27 m , thickness= 50 cm

Dimensions:

Height of Boundary element: 420 mm

Width of Boundary element: 2800 mm

Cover: 40 mm to
transverse
reinforcement

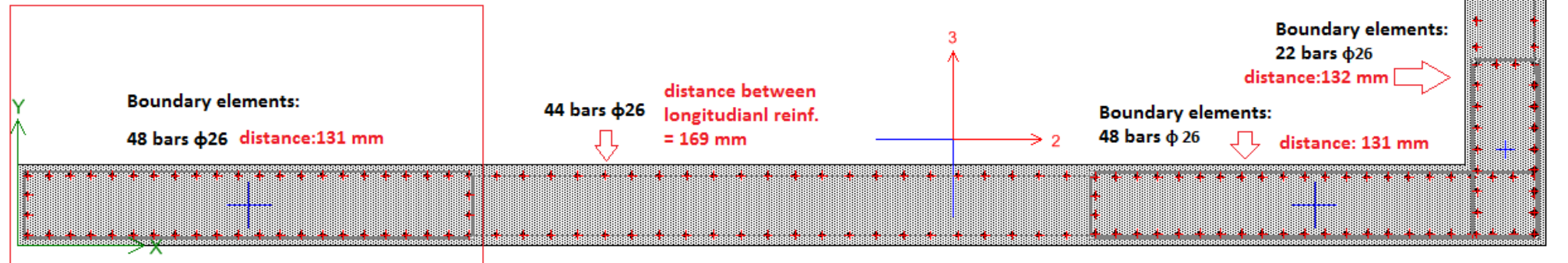


Figure 4.16: Shear wall- Detailing of reinforcement-Arrangement 1.

4.1.3.1 Arrangement 1

4.1.3.1.1 Design for flexure:

In ETABS 2013 , for calculating shear wall, option: General reinforcing pier section was specified. Program has created series of interaction surfaces based on the reinforcement defined in Section Designer window.

Eight ratios of steel reinforcing area to pier section area were defined. During calculations, pier area was held constant and bar area was modified to obtain these ratios. The relative size of each of the reinforcing bar to the other was always kept constant.

The eight reinforcing ratios used by the Program

1. $IP_{min} = 0,005$
2. $IP_{min} + \frac{IP_{max}-IP_{min}}{14} = 0,005 + \frac{0,04-0,005}{14} = 0,0075$
3. $IP_{min} + \frac{7}{3} \left(\frac{IP_{max}-IP_{min}}{14} \right) = 0,0108$
4. $IP_{min} + 4 \left(\frac{IP_{max}-IP_{min}}{14} \right) = 0,015$
5. $IP_{min} + 6 \left(\frac{IP_{max}-IP_{min}}{14} \right) = 0,02$
6. $IP_{min} + \frac{25}{3} \left(\frac{IP_{max}-IP_{min}}{14} \right) = 0,0258$
7. $IP_{min} + 11 \left(\frac{IP_{max}-IP_{min}}{14} \right) = 0,0325$
8. $IP_{max} = 0,04$

Minimum reinforcing ratio, IP_{min} , according to provisions of Eurocode 2 was chosen as 0.005 and the maximum, IP_{max} , is 0.04.

The next step was to develop interaction surfaces defined by series of PMM interaction curves spaced around 360 degree circle, for all eight of the ratios.

Each interaction curve that makes up the interaction surface was numerically described by a series of discrete points connected by straight lines. These points were determined by rotating a plane of linear strain about the neutral axis on the section of the pier. The program used the requirements of force equilibrium and strain compatibility to determine the axial load and moment resistance of the wall pier.

Then, program has used linear interpolation between the eight interaction surfaces to obtain reinforcing ratio that gives an ratio of demand to capacity equal to 1. That process was repeated for all design load combinations and the highest required reinforcing ratio was reported as a design ratio.

4.1.3.1.2 Design for shear:

V_{Ed}	Factored shear force at a section
$V_{Rd,c}$	Design shear resistance without shear reinforcement
$V_{Rd,max}$	Shear force that can be carried without crushing of the notional concrete compressive struts

The program only designs the pier for shear and reports how much shear reinforcing is required. The shear design is performed at stations at the top and bottom of the pier.

The following steps are involved in designing the shear reinforcing for a particular leg of a wall pier section for a particular design loading combination:

1. Determine the factored forces that are acting on the leg of the wall pier section.
2. Determine the factored shear resistance, $V_{Rd,c}$ that can be carried by the concrete.
3. Determine the required shear reinforcing to carry the balance of the shear force.

Maximum sustainable design shear force:

$$V_{Rd,max} = \frac{\alpha_{cw} \cdot b_w \cdot z \cdot v_1 \cdot f_{cd}}{(\cot\phi + \tan\phi)} \quad (4.26)$$

The coefficient α_{cw} takes account of the state of stress in the compression chord and is taken equal to 1, which is recommended for non prestressed structures.

$$\alpha_{cw} = 1$$

The strength reduction factor:

$$v_1 = 0,6 \cdot \left[1 - \frac{f_{ck}}{250} \right] = 0,6 \cdot \left[1 - \frac{40}{250} \right] = 0,504 \quad (4.27)$$

The angle between the concrete compression strut and the column axis perpendicular to the shear force is defined as ϕ and is assumed by program as the conservative value of 45 degrees.

$$f_{cd} = \alpha_{cc} \frac{f_{ck}}{\gamma_c} = 1 \cdot \frac{40}{1,5} = 26,67 \text{ MPa}$$

If V_{Ed} is greater than V_{Rdc} and less than $V_{Rd,max}$ the required shear reinforcement in the form of stirrups or ties per spacing is calculated as:

$$\frac{A_{sw}}{s} = \frac{V_{Ed}}{z \cdot f_{ywd} \cdot \cot\phi}$$

The maximum of all of the calculated $\frac{A_{sw}}{s}$ values, obtained from each design load combination, is reported for the major and minor directions of the column, along with the controlling combination name.

4.1.3.1.3 Boundary check elements:

Boundary elements are the heavily-reinforced, critical zones of shear walls normally located close to the edges of the wall or next to large wall openings.

According to the main characteristic of the shear wall , ETABS has calculated the maximum normalized compressive stress at the extreme fiber of the concrete pier.

$$V_d = \frac{N_{Ed}}{l_w \cdot t_p \cdot f_{cd}} \quad (4.28)$$

If the compressive stress exceeds 0,15 boundary elements are required.

For the each of the wall of the pier, boundary elements are required according to figure shown below.

The neutral axis depth, based on the axial force was estimated by program using formula 5.21 of EC8 and results are shown in the last column on the figure above.

(EC8 Eq.5.21)

$$\omega_v = \frac{A_{sv}}{l_w \cdot t_p} \cdot \frac{f_{yd}}{f_{cd}} \quad (4.29)$$

$$C \text{ Depth} = x_u = (v_d + \omega_v) \cdot \frac{l_w \cdot t_p}{(t_p - 2 \cdot cover)} \quad (4.30)$$

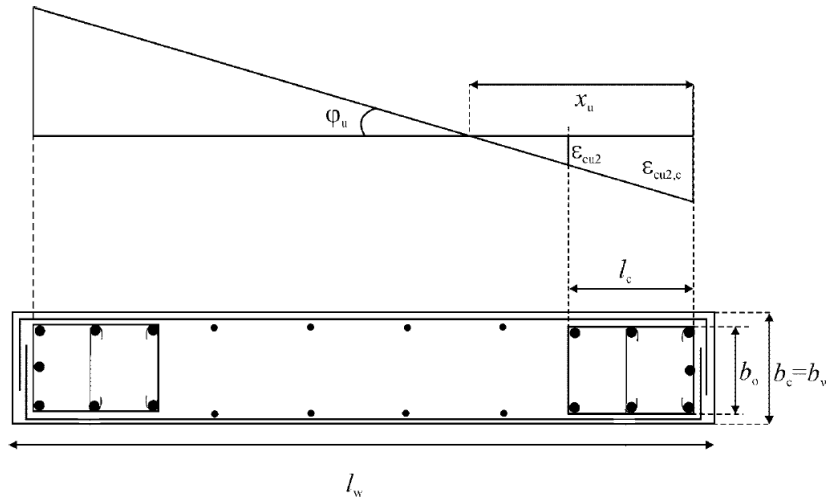


Figure 4.17: Length of edge of boundary element.

Length of boundary element is computed using clause EC8 5.4.3.4.2 (6):

$$l_c = x_u \left(1 - \frac{\epsilon_{cu2}}{\epsilon_{cu2,c}} \right) \quad (4.31)$$

Based on the ETABS results:

Table 4.1: Design forces in Core 4 – EC.

Station	ID	Governing	N_{Ed}	M_{Ed}
Location		Combination	kN	kN-m
Top-Left	Leg 1	DWal11	25011,76	-641,3754
Top-Right	Leg 1	DWal11	25011,76	710,0491
Top-Left	Leg 2	DWal12	63078,707	-4470,7831
Top-Right	Leg 2	DWal12	63078,707	1914,217
Bottom-Left	Leg 1	DWal11	25141,722	-1409,4554
Bottom-Right	Leg 1	DWal11	25141,722	1450,0116
Bottom-Left	Leg 2	DWal12	63455,189	-5869,615
Bottom-Right	Leg 2	DWal12	63455,189	3033,5171

For Leg1:

- Top

Normalised axial load

$$V_d = \frac{N_{Ed}}{l_w \cdot t_p \cdot f_{cd}}$$

$$V_d = \frac{25141,72}{320 \cdot 50 \cdot 2,67} = 0,589$$

Mechanical ratio of vertical web reinforcement:

$$\omega_v = \frac{A_{sv}}{l_w \cdot t_p} \cdot \frac{f_{yd}}{f_{cd}}$$

$$A_{sv} = 201,75 \text{ cm}^2$$

$$\omega_v = \frac{201,75}{320 \cdot 50} \cdot \frac{35,973}{2,67} = 0,17$$

$$\begin{aligned} C \text{ Depth} = x_u &= (v_d + \omega_v) \cdot \frac{l_w \cdot t_p}{(t_p - 2 \cdot cover)} = (0,589 + 0,17) \cdot \frac{320 \cdot 50}{(50 - 2 \cdot 4)} \\ &= 289,28 \text{ cm} \end{aligned}$$

The maximum strain of unconfined concrete: (EC 8.5.4.3.4.2(6))

$$\varepsilon_{cu2} = 0,0035$$

The maximum strain of confined concrete:

$$\varepsilon_{cu2,c} = 0,0035 + 0,1 \cdot \alpha \cdot \omega_{\omega d} = 0,0035 + 0,1 \cdot 0,258 = 0,029 \quad (4.32)$$

Ductility factor:

$$\mu_{\phi} = 6$$

α -the confinement effectiveness factor

$\omega_{\omega d}$ - the mechanical ratio of confinement reinforcement

$$\alpha \cdot \omega_{\omega d} = 30 \cdot \mu_{\phi} (v_d + \omega_{\eta}) \varepsilon_{sy,d} \cdot \frac{t_p}{(t_p - 2cover)} - 0,035 \quad (4.33)$$

$$\varepsilon_{sy,d} = \frac{f_{yd}}{E_s} \quad (4.34)$$

$$\varepsilon_{sy,d} = \frac{35,973}{20000} = 0,00179$$

$$\alpha \cdot \omega_{\omega d} = 30 \cdot 6(0,589 + 0,17)0,00179 \cdot \frac{50}{(50 - 2 \cdot 4)} - 0,035 = 0,258$$

Length of boundary elements:

$$l_c = x_u \left(1 - \frac{\varepsilon_{cu2}}{\varepsilon_{cu2,c}}\right)$$

$$l_c = 289,28 \cdot \left(1 - \frac{0,0035}{0,029}\right) = 254,68 \text{ cm}$$

LEG 2:

- Bottom

Normalised axial load

$$V_d = \frac{N_{Ed}}{l_w \cdot t_p \cdot f_{cd}}$$

$$V_d = \frac{63455,19}{927 \cdot 50 \cdot 2,67} = 0,513$$

Mechanical ratio of vertical web reinforcement:

$$\omega_v = \frac{A_{sv}}{l_w \cdot t_p} \cdot \frac{f_{yd}}{f_{cd}}$$

$$A_{sv} = 445,98 \text{ cm}^2$$

$$\omega_v = \frac{445,98}{927 \cdot 50} \cdot \frac{35,973}{2,67} = 0,1297$$

$$\begin{aligned} C \text{ Depth} = x_u &= (v_d + \omega_v) \cdot \frac{l_w \cdot t_p}{(t_p - 2 \cdot cover)} = (0,513 + 0,1297) \cdot \frac{927 \cdot 50}{(50 - 2 \cdot 4)} \\ &= 709,8 \text{ cm} \end{aligned}$$

The maximum strain of unconfined concrete (EC 8.5.4.3.4.2(6))

$$\varepsilon_{cu2} = 0,0035$$

The maximum strain of confined concrete:

$$\varepsilon_{cu2,c} = 0,0035 + 0,1 \cdot \alpha \cdot \omega_{\omega d} = 0,0035 + 0,1 \cdot 0,213 = 0,025$$

Ductility factor:

$$\begin{aligned} \mu_\phi &= 6 \\ \alpha \cdot \omega_{\omega d} &= 30 \cdot 6(v_d + \omega_\eta) \varepsilon_{sy,d} \cdot \frac{t_p}{(t_p - 2cover)} - 0,035 \end{aligned}$$

$$\varepsilon_{sy,d} = \frac{f_{yd}}{E_s}$$

$$\varepsilon_{sy,d} = \frac{35,973}{20000} = 0,00179$$

$$\alpha \cdot \omega_{\omega d} = 30 \cdot 6(0,513 + 0,1297)0,00179 \cdot \frac{50}{(50 - 2 \cdot 4)} - 0,035 = 0,213$$

Length of boundary elements:

$$\begin{aligned} l_c &= x_u \left(1 - \frac{\varepsilon_{cu2}}{\varepsilon_{cu2,c}}\right) \\ l_c &= 709,8 \cdot \left(1 - \frac{0,0035}{0,025}\right) = 609,6 \text{ cm} \end{aligned}$$

Table 4.2: Boundary Element Check-EC.

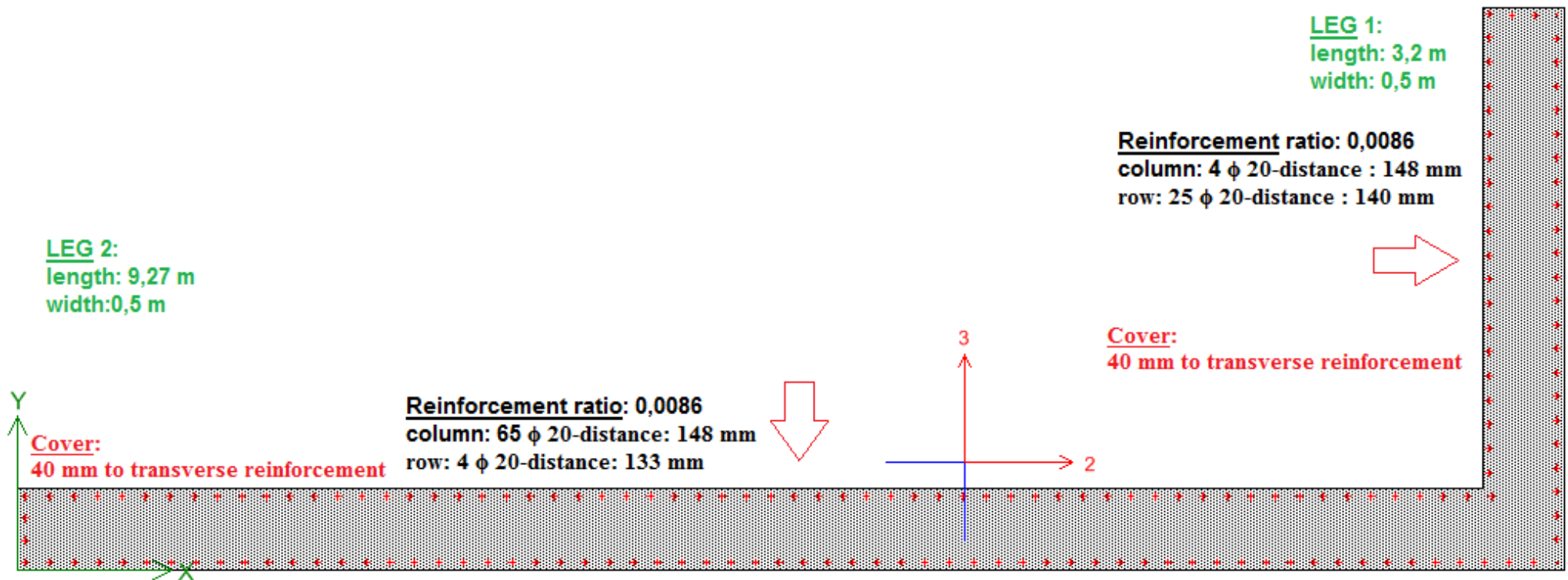
Station Location	ID	Edge Length (mm)	Governing Combo	N _{Ed} kN	M _{Ed} kN-m	Normalized Comp. Stress	Normalized Stress Limit	C Depth mm
Top-Left	Leg 1	2162,9	DWal11	25011,76	-641,3754	0,614	0,15	2466,3
Top-Right	Leg 1	2162,9	DWal11	25011,76	710,0491	0,617	0,15	2466,3
Top-Left	Leg 2	5475,4	DWal12	63078,707	-4470,7831	0,534	0,15	6340,8
Top-Right	Leg 2	5475,4	DWal12	63078,707	1914,217	0,52	0,15	6340,8
Bottom-Left	Leg 1	2205,5	DWal11	25141,722	-1409,4554	0,651	0,15	2509,6
Bottom-Right	Leg 1	2205,5	DWal11	25141,722	1450,0116	0,653	0,15	2509,6
Bottom-Left	Leg 2	5598,5	DWal12	63455,189	-5869,615	0,544	0,15	6466,2
Bottom-Right	Leg 2	5598,5	DWal12	63455,189	3033,5171	0,529	0,15	6466,2

According to results obtained above , shear wall has to have boundary elements at each end , which overlap to each other. It means that the whole length of the wall legs, mentioned above, has to be treated as the boundary element.

Eurocode for estimation of neutral axis depth uses axial force at the section of the wall. Because of that fact , distance from the extreme fibers exposed to compression to neutral axis is relatively too high , compare with ASCE, which uses different algorithm to calculate c depth value.

4.1.3.2 Arrangement 2

Assuming that the whole length of each of the shear wall leg has to be treated, as a boundary element, according to the provisions of Eurocode, flexure and shear detailing will be defined due to requirements for DCH, based on EC 2 and Ec 8. Pattern of longitudinal bars was taken to fullfil the requirements of maximum spacing of bars under flexure kept by transverse reinforcement, limited to 30 cm.



4.18: Shear wall- Detailing of reinforcement-Arrangement 2.

4.1.3.2.1 Design for flexure

Table 4.3: Flexure design forces for Core 4-EC.

Station	Required	Required	Current	Flexural	N _{Ed}	M _{Ed2}	M _{Ed3}
Location	Rebar Area (mm ²)	Reinf Ratio	Reinf Ratio	Combo	kN	kN-m	kN-m
Top	47037	0,0075	0,0086	DWal18	13792,199	-13558,679	26192,705
Bottom	52625	0,0084	0,0086	DWal18	14298,643	-14428,846	25938,653

Provided rebar area:

Number of bars: 178

Diameter : 20 mm

$$\text{Area of one bar: } \frac{\pi \cdot d^2}{4} = \frac{\pi \cdot 20^2}{4} = 315,16 \text{ mm}^2$$

$$\text{Total area: } 178 \cdot \frac{\pi \cdot 25^2}{4} = 55920,35 \text{ mm}^2$$

In both cases (top and bottom) , provided rebar area is greater than the required rebar area.

4.1.3.2.2 Design for shear:

Table 4.4: Shear design forces for Core 4-EC.

Station	ID	Rebar	Shear	N _{Ed}	V _{Ed}	V _{Rc}	V _{Rd}
Location		mm ² /m	Combo	kN	kN	kN	kN
Top	Leg 1	1285,16	DWal5	1836,4333	2958,768	887,3286	2958,768
Top	Leg 2	3600,56	DWal17	4649,9062	9605,3307	2310,2722	9605,3307
Bottom	Leg 1	1285,16	DWal5	2011,8815	2958,768	936,9745	2958,768
Bottom	Leg 2	3600,56	DWal17	5026,3888	9605,3307	2428,2356	9605,3307

LEG 1 -Provided:

$$s = 20 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

For total height: 3,25 m there is 50,24 cm² (16 · 3,14 cm²) of transversional reinforcement. It is greater than required amount 41,76 cm² (3,25 · 12,85 cm²).

LEG 2- Provided:

$$s = 10 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 1,2^2}{4} = 3,14 \text{ cm}^2$$

For total height: 3,25 m there is $149,28 \text{ cm}^2$ ($33 \cdot 3,14 \text{ cm}^2$) of transversional reinforcement. It is greater than required amount 117 cm^2 ($3,25 \cdot 36 \text{ cm}^2$).

Detailing requirements for high ductility class (DCH) to Eurocode 2 and 8:

- Wall dimensions:

Minimum width: $B_{w,min} = 150 \text{ mm or } \frac{H_s}{20}$ (5.4.1.2.3.(1)(2))

Minimum length/width ratio $\frac{L_w}{B_{w,min}}$ (5.1.2.(1))

Minimum critical zone height $H_{cr,min} = \frac{H_w}{6}$ (5.5.3.4.5.(1))

Critical zone height $H_{cr,max} = 2L_w, H_s$ (5.5.3.4.5 (1))

Minimum length of confined zone $L_{c,min} = 0,15L_w; 1,5B_w$ (5.4.3.4.2(6))

Minimum width of confined zone $B_{c,min}$

- Horizontal and vertical web mesh: (5.5.3.4.5(13.14))

Minimum reinforcement ratio: $\rho_{w,min} = 0,0025$

Minimum diameter: $d_{vh,min} = 8 \text{ mm}$

Minimum clear spacing between bars: $a_{min} = 50 \text{ mm}$

Maximum spacing between bar centers: $s_{vh,max} = 250 \text{ mm}, 25 d_{vh}$

Maximum bar diameter: $d_{vh,max} = \frac{B_{w0}}{8}$

Maximum spacing between S-shaped links: $d_{s,max} = 500 \text{ mm}$

- Longitudinal reinforcement in confined zone: (5.4.3.4.2(8))

Minimum diameter: $d_{bl,min} = 12 \text{ mm}$

Minimum reinforcement ratio: $\rho_{min} = 0,005$

Maximum reinforcement ratio: $\rho_{max} = 0,04$

Minimum clear spacing between bars: $a_{min} = 50 \text{ mm}$

Maximum spacing between bar centers: $a_{L,max} = 200 \text{ mm}$

- Shear reinforcement in confined zones: (5.5.3.4.5.(10))

Minimum diameter: $d_{w,min} = 6 \text{ mm}$

Maximum spacing between shear links centers $s_{cr,max} = \frac{b_0}{3}; 6d_{bl}; 125 \text{ mm}$

Anchoring length inside the concrete: $l_{bw} = 10d_{bw}$

4.1.4 Story drift

Story drift determination

Storey drifts is the total lateral displacement that occurs in a single storey of a multistorey building. It must be checked and compared with specified limits in codes, principally to limit damage to non-structural elements.

Drift in building frames is a result of flexural and shear mode contributions, due to the column axial deformations and to the diagonal and girder deformations. Storey drift has to be limited to assure other non-structural elements such as cladding, partitions and pipework to be able to accept the deflections imposed on them during an earthquake without failure. Failure of external cladding, blockage of escape routes by fallen partitions and ruptured firewater pipework all have serious safety implications.

Limitation of interstorey drift due to EC 8-1 was taken as for non-structural elements fixed in a way so as not to interfere with structural deformations .

$$d_r \leq \frac{0,01 \cdot h}{\nu} \quad (4.35)$$

d_r – interstorey drift

h - storey height

ν -the reduction factor taking into account the lower return period of the seismic action associated with the damage limitation requirement (in case of importance class I and II ; $\nu = 0,5$ and in case of importance class III and IV $\nu = 0,4$)

In different standards the design interstorey drift is typically indicated with the drift ratio that is the quotient of the displacement difference and the storey height.

$$Drift\ ratio = \frac{\Delta s}{h_{st}} \quad (4.36)$$

Drifts for each floor in the form of drift ratio are shown on the Figure 4.20 and Table 4.5 and 4.6.

Table 4.5: Allowable drift limit check in X direction- EC 8.

Story	Load Case/Combo	Item	Drift	Δs	Max inelastic response displacement	Allowable limit
Story45	EqX Max	Max Drift X	0,00070	0,00229	0,01338	0,065
Story44			0,00071	0,00232	0,01357	0,065
Story43			0,00072	0,00234	0,01371	0,065
Story42			0,00073	0,00237	0,01384	0,065
Story41			0,00073	0,00239	0,01396	0,065
Story40			0,00074	0,00241	0,01409	0,065
Story39			0,00075	0,00243	0,01420	0,065
Story38			0,00076	0,00247	0,01443	0,065
Story37			0,00077	0,00251	0,01466	0,065
Story36			0,00078	0,00254	0,01487	0,065
Story35			0,00079	0,00257	0,01506	0,065
Story34			0,00080	0,00261	0,01525	0,065
Story33			0,00081	0,00263	0,01540	0,065
Story32			0,00082	0,00266	0,01555	0,065
Story31			0,00082	0,00267	0,01565	0,065
Story30			0,00083	0,00269	0,01576	0,065
Story29			0,00083	0,00271	0,01586	0,065
Story28			0,00084	0,00272	0,01591	0,065
Story27			0,00084	0,00273	0,01595	0,065
Story26			0,00084	0,00273	0,01599	0,065
Story25			0,00084	0,00273	0,01599	0,065
Story24			0,00084	0,00273	0,01597	0,065
Story23			0,00084	0,00272	0,01591	0,065
Story22			0,00083	0,00271	0,01584	0,065
Story21			0,00083	0,00269	0,01574	0,065
Story20			0,00082	0,00267	0,01563	0,065
Story19			0,00081	0,00265	0,01548	0,065
Story18			0,00081	0,00262	0,01531	0,065
Story17			0,00079	0,00258	0,01510	0,065
Story16			0,00078	0,00254	0,01485	0,065
Story15			0,00077	0,00249	0,01458	0,065
Story14			0,00075	0,00244	0,01428	0,065
Story13			0,00073	0,00238	0,01394	0,065
Story12			0,00071	0,00232	0,01356	0,065
Story11			0,00069	0,00224	0,01312	0,065
Story10			0,00066	0,00216	0,01262	0,065
Story9			0,00064	0,00206	0,01207	0,065
Story8			0,00060	0,00196	0,01145	0,065
Story7			0,00057	0,00184	0,01074	0,065
Story6			0,00052	0,00170	0,00992	0,065
Story5			0,00047	0,00153	0,00897	0,065
Story4			0,00041	0,00135	0,00787	0,065
Story3			0,00034	0,00112	0,00654	0,065
Story2			0,00026	0,00085	0,00494	0,065
Story1			0,00014	0,00045	0,00262	0,065

Table 4.6: Allowable drift limit check in Y direction- EC 8.

Story	Load Case/Combo	Item	Drift	Δs	Max inelastic response displacement	Allowable limmit
Story45	EqY Max	Max Drift Y	0,00217	0,00707	0,04133	0,065
Story44			0,00221	0,00719	0,04206	0,065
Story43			0,00225	0,00732	0,04284	0,065
Story42			0,00230	0,00747	0,04371	0,065
Story41			0,00235	0,00763	0,04464	0,065
Story40			0,00240	0,00780	0,04561	0,065
Story39			0,00245	0,00796	0,04658	0,065
Story38			0,00250	0,00813	0,04757	0,065
Story37			0,00255	0,00829	0,04852	0,065
Story36			0,00260	0,00845	0,04945	0,065
Story35			0,00265	0,00860	0,05033	0,065
Story34			0,00269	0,00874	0,05114	0,065
Story33			0,00273	0,00887	0,05190	0,065
Story32			0,00277	0,00900	0,05263	0,065
Story31			0,00280	0,00908	0,05314	0,065
Story30			0,00283	0,00918	0,05373	0,065
Story29			0,00285	0,00927	0,05424	0,065
Story28			0,00288	0,00935	0,05470	0,065
Story27			0,00290	0,00942	0,05508	0,065
Story26			0,00291	0,00946	0,05536	0,065
Story25			0,00292	0,00950	0,05557	0,065
Story24			0,00293	0,00952	0,05571	0,065
Story23			0,00293	0,00953	0,05574	0,065
Story22			0,00293	0,00952	0,05569	0,065
Story21			0,00292	0,00949	0,05552	0,065
Story20			0,00291	0,00944	0,05525	0,065
Story19			0,00289	0,00938	0,05485	0,065
Story18			0,00286	0,00929	0,05436	0,065
Story17			0,00283	0,00919	0,05375	0,065
Story16			0,00278	0,00905	0,05293	0,065
Story15			0,00274	0,00890	0,05208	0,065
Story14			0,00269	0,00873	0,05105	0,065
Story13			0,00262	0,00852	0,04987	0,065
Story12			0,00255	0,00829	0,04852	0,065
Story11			0,00247	0,00803	0,04698	0,065
Story10			0,00238	0,00773	0,04521	0,065
Story9			0,00227	0,00738	0,04320	0,065
Story8			0,00215	0,00699	0,04088	0,065
Story7			0,00201	0,00653	0,03822	0,065
Story6			0,00185	0,00600	0,03512	0,065
Story5			0,00166	0,00538	0,03148	0,065
Story4			0,00143	0,00465	0,02719	0,065
Story3			0,00116	0,00376	0,02198	0,065
Story2			0,00082	0,00266	0,01555	0,065
Story1			0,00034	0,00111	0,00650	0,065

Lateral displacements for storeys in X and Y direction, obtained from the three-dimensional static, elastic analysis using the design seismic forces are seen on the Figure 4.21 and 4.22 . Maximum displacements are recorded on last floor (45) and theirs values are as follow:

Load combination : EX

Global X (blue) : 101,94 mm

Global Y (red) : 47,98 mm

Load combination : EY

Global X (blue) : 62,95 mm

Global Y (red) : **352,53** mm

$$\frac{H}{500} = \frac{145\text{ m}}{500} = 0,29\text{ m}$$

Displacement values from the calculations (but just in SLS, which not include drift) exceed the recommended (but not required) maximum values in case of drift - displacements top of the building $H / 500$. This indicates a low stiffness of the building. It will be recommended to undertake experimentals.

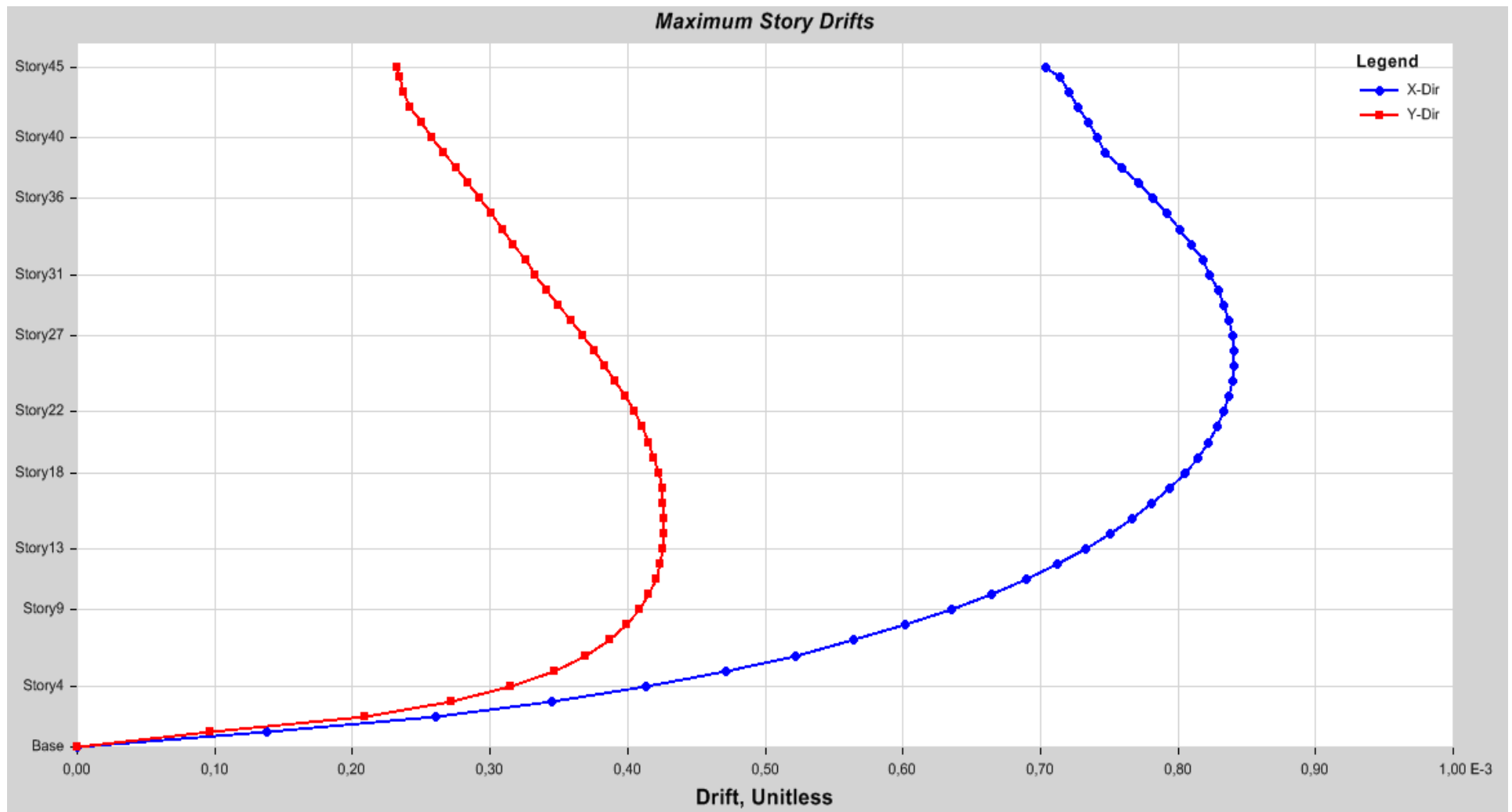


Figure 4.19:Critical drifts for storeys due to EqX load case-EC 8.1.

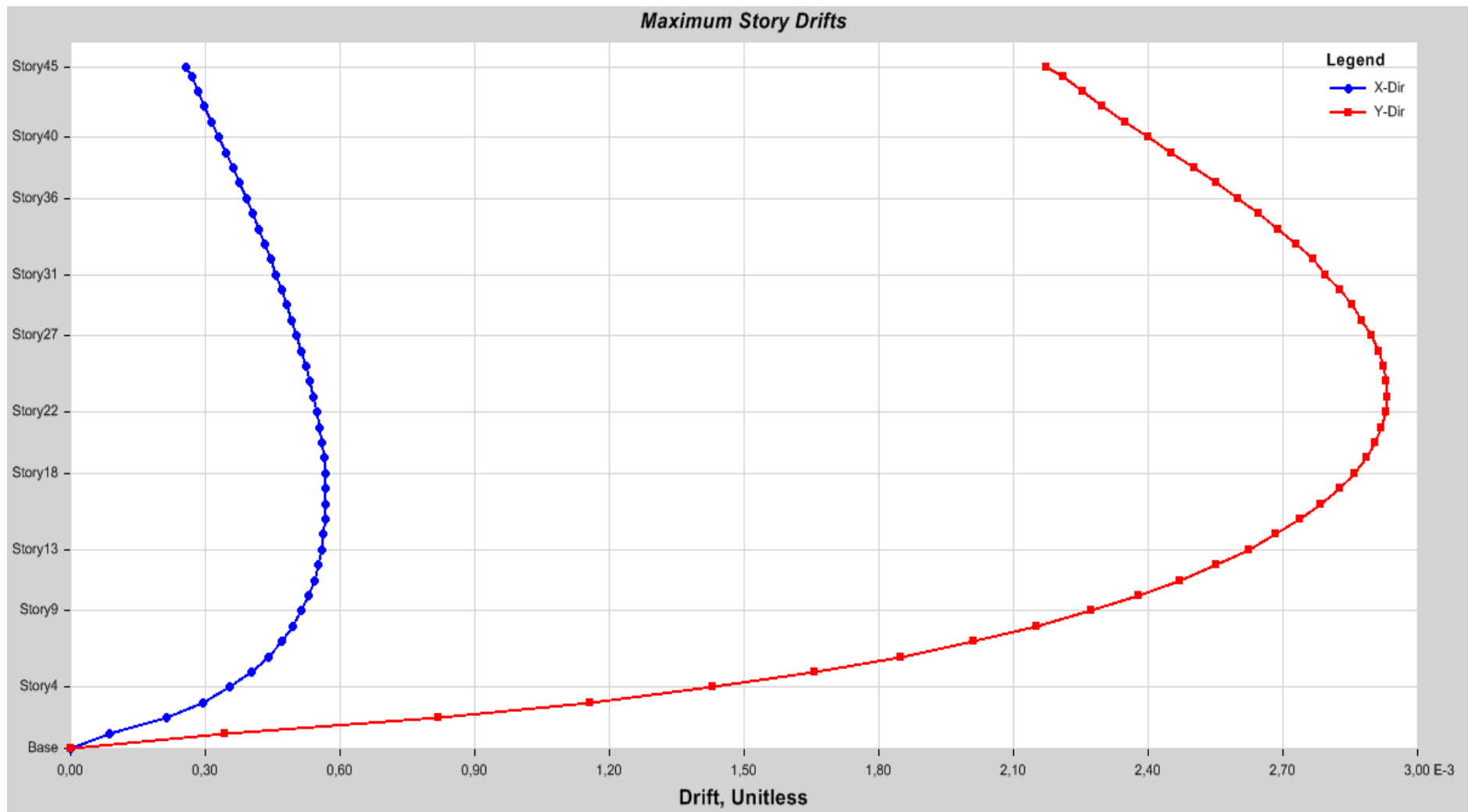


Figure 4.20:Critical drifts for storeys due to EqY load case-EC 8.1.

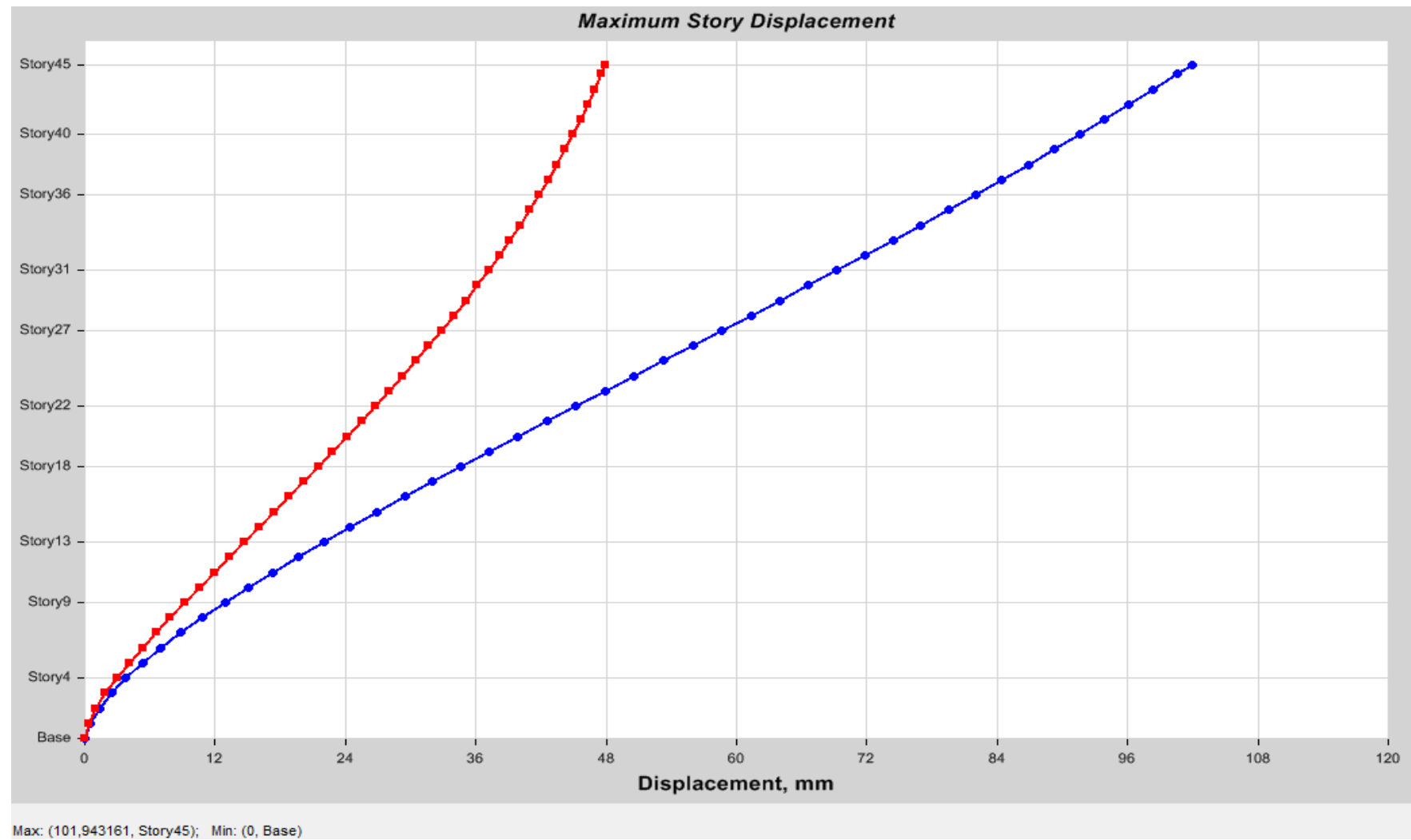


Figure 4.21: Maximum story displacement due to earthquake load in X direction- EN 1998-1:2004.

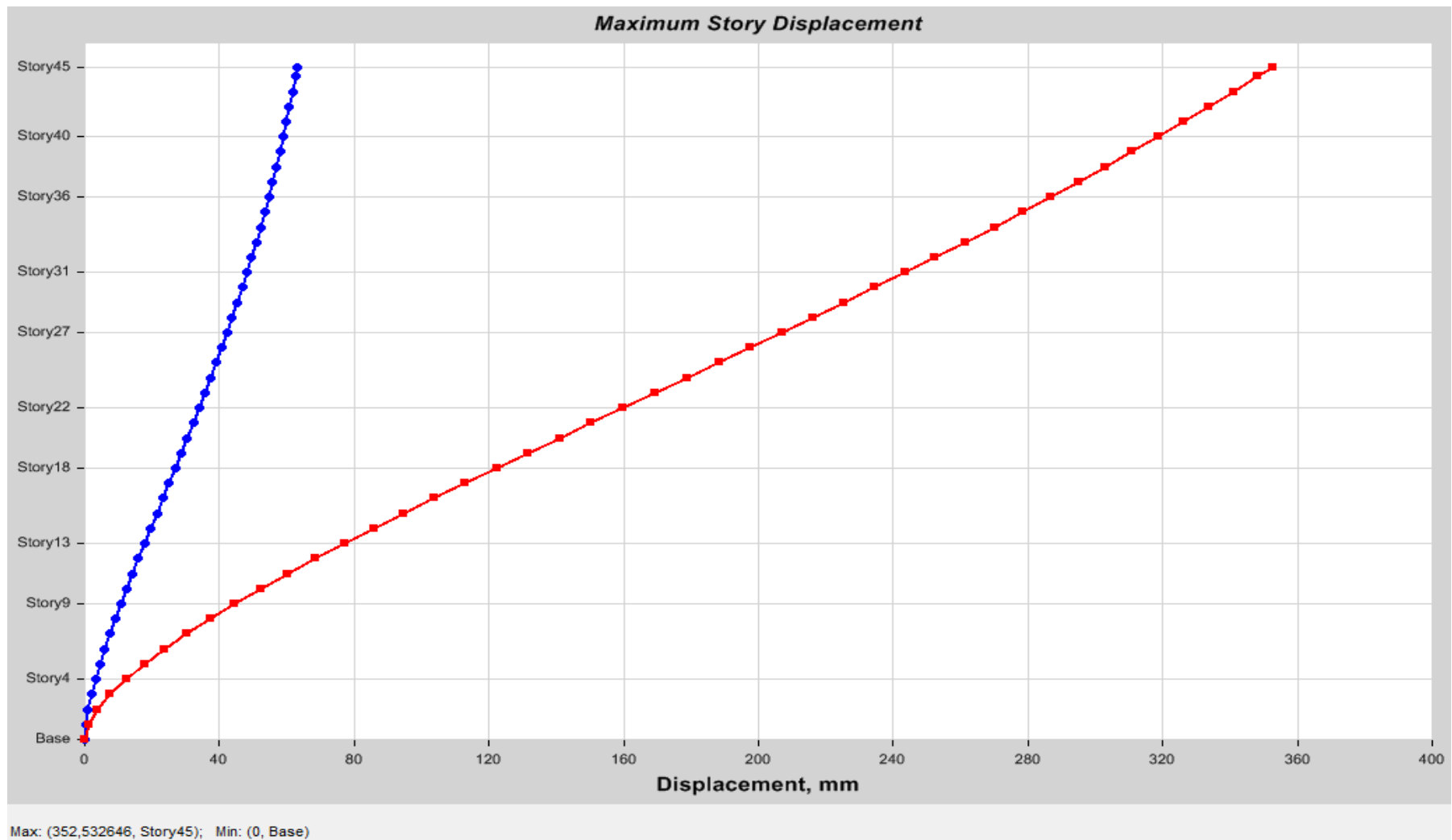


Figure 4.22: Maximum story displacement due to earthquake load in Y direction- EN 1998-1:2004.

4.2 ACI 318-11

4.2.1 Beam Design-B46 floor 10

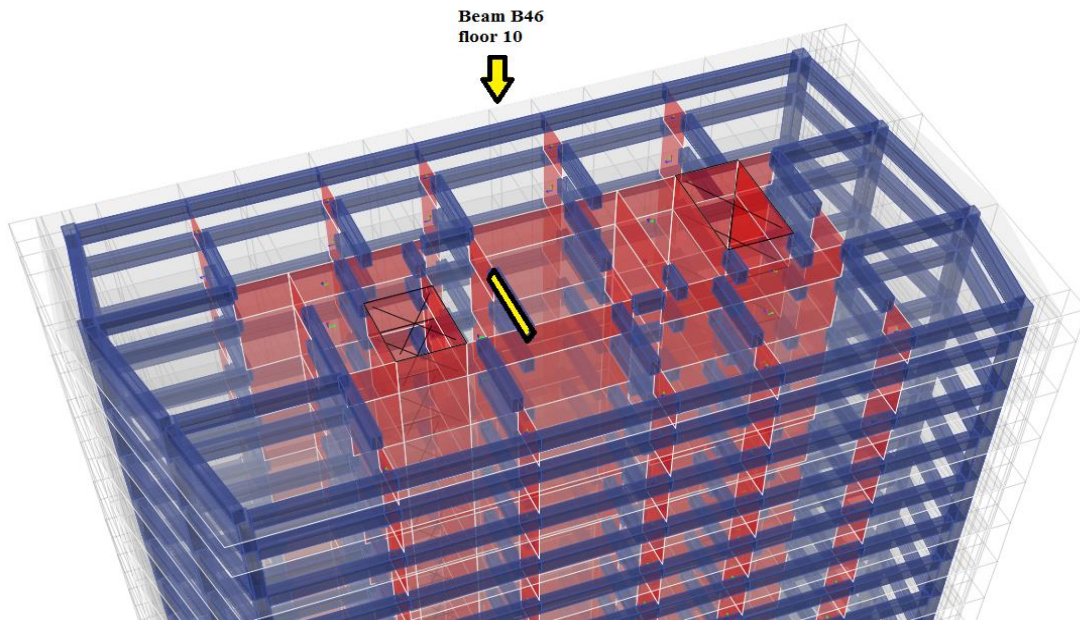


Figure 4.23: Location of beam B37.

The cracked section properties (assigned with member property reduction factors specification) are used solely in setting up the terms in the stiffness matrix. The stiffness in the analysis is as follow:

$$I_{eff} = 0,5 I_g \quad (4.37)$$

4.2.1.1 Flexure design

Concrete: C40/50, Steel: A615Gr60

Reinforcing Yield Strength $f_y = 413,69 \text{ MPa}$

Concrete Compressive Strength $f'_c = 40 \text{ MPa}$

Beam Width $b = 40 \text{ cm}$

Depth to Tension Reinforcing $d = 66 \text{ cm}$

Total Beam Depth $h = 70 \text{ cm}$

- Section 1 ($L = 0$) : COMB15

$$M_{u,top} = -1218,22 \text{ kNm}$$

$$M_{u,bottom} = 1112,84 \text{ kNm}$$

The depth of the compression block:

$$a_I = d - \sqrt{d^2 - \frac{2|M_u|}{0,85 \cdot f'_c \cdot \phi \cdot b}} \quad (4.38)$$

$$a_I = 76 - \sqrt{76^2 - \frac{2 \cdot 121822}{0,85 \cdot 4 \cdot 0,9 \cdot 50}} = 11,31 \text{ cm}$$

$$a_{II} = 76 - \sqrt{76^2 - \frac{2 \cdot 111284}{0,85 \cdot 4 \cdot 0,9 \cdot 50}} = 10,26 \text{ cm}$$

The maximum depth of the compression zone:

$$c_{max} = \frac{\varepsilon_{cmax}}{\varepsilon_{c,max} + \varepsilon_{s,min}} d = \frac{0,003}{0,003 + 0,005} \cdot 76 = 28,5 \text{ cm} \quad (4.39)$$

The maximum allowable depth of the rectangular compression block:

$$a_{max} = \beta_1 \cdot c_{max} = 0,75 \cdot 28,5 = 21,375 \text{ cm} \quad (4.40)$$

$$\beta_1 = 0,75$$

If $a \leq a_{max}$ then the area of tensile steel reinforcement is calculated according to ACI 10.3.4 and 10.3.5 as follow:

$$A_s = \frac{M_u}{\phi \cdot f_y \cdot \left(d - \frac{a}{2}\right)} \quad (4.41)$$

$$A_{s,top} = \frac{121822}{0,9 \cdot 41,369 \cdot \left(76 - \frac{11,31}{2}\right)} = 46,52 \text{ cm}^2$$

$$A_{s,bottom} = \frac{M_u}{\phi \cdot f_y \cdot \left(d - \frac{a}{2}\right)} = \frac{111284}{0,9 \cdot 41,369 \cdot \left(76 - \frac{10,26}{2}\right)} = 42,18 \text{ cm}^2$$

Reinforcement applied:

5 ϕ 26 + 4 ϕ 26

$$A_{s,top} = \frac{9 \cdot \pi \cdot 2,6^2}{4} = 47,78 \text{ cm}^2$$

4 ϕ 26 + 6 ϕ 22

$$A_{s,bottom} = \frac{4 \cdot \pi \cdot 2,6^2}{4} + \frac{6 \cdot \pi \cdot 2,2^2}{4} = 44,05 \text{ cm}^2$$

- Section 2 ($0,55 L \cong 2,4 m$) :

$$M_u = 304,55 \text{ kNm}$$

The depth of the compression block:

$$a_l = d - \sqrt{d^2 - \frac{2|M_u|}{0,85 \cdot f'_c \cdot \phi \cdot b}} = 76 - \sqrt{76^2 - \frac{2 \cdot 30455}{0,85 \cdot 4 \cdot 0,9 \cdot 50}} = 2,67 \text{ cm}$$

Required flexural reinforcement

$$A_{s,bottom} = \frac{M_u}{\phi \cdot f_y \cdot \left(d - \frac{a}{2}\right)} = \frac{30455}{0,9 \cdot 41,369 \cdot \left(76 - \frac{2,67}{2}\right)} = 10,96 \text{ cm}^2$$

Reinforcement applied:

4 ϕ 26

$$A_{s,bottom} = \frac{4 \cdot \pi \cdot 2,6^2}{4} = 21,2 \text{ cm}^2$$

- Section 3 ($L = 4,4 m$) :

$$M_{u,top} = -1105,45 \text{ kNm}$$

$$M_{u,bottom} = 968,22 \text{ kNm}$$

The depth of the compression block:

$$a_l = d - \sqrt{d^2 - \frac{2|M_u|}{0,85 \cdot f'_c \cdot \phi \cdot b}} = 76 - \sqrt{76^2 - \frac{2 \cdot 110545}{0,85 \cdot 4 \cdot 0,9 \cdot 50}} = 10,19 \text{ cm}$$

$$a_{ll} = 76 - \sqrt{76^2 - \frac{2 \cdot 968,22}{0,85 \cdot 4 \cdot 0,9 \cdot 50}} = 8,84 \text{ cm}$$

Required flexural reinforcement:

$$A_{s,top} = \frac{M_u}{\phi \cdot f_y \cdot \left(d - \frac{a}{2}\right)} = \frac{110545}{0,9 \cdot 41,369 \cdot \left(76 - \frac{10,19}{2}\right)} = 41,87 \text{ cm}^2$$

$$A_{s,bottom} = \frac{M_u}{\phi \cdot f_y \cdot \left(d - \frac{a}{2}\right)} = \frac{96822}{0,9 \cdot 41,369 \cdot \left(66 - \frac{8,84}{2}\right)} = 36,33 \text{ cm}^2$$

Reinforcement applied:

$$5\phi 26 + 4\phi 26$$

$$A_{s,top} = \frac{9 \cdot \pi \cdot 2,6^2}{4} = 47,78 \text{ cm}^2$$

$$4\phi 26 + 3\phi 26$$

$$A_{s,bottom} = \frac{7 \cdot \pi \cdot 2,6^2}{4} = 37,17 \text{ cm}^2$$

The minimum required area of flexural reinforcing:

The minimum flexural tensile steel required in a beam section is given by the minimum of the following two limits:

$$A_s \geq \min \left\{ \begin{array}{l} \max \left\{ \begin{array}{l} \frac{3\sqrt{f'_c}}{f_y} \cdot b \cdot d \\ \frac{200}{f_y} \cdot b \cdot d \end{array} \right. \\ \frac{4}{3} A_{s(required)} \end{array} \right. \quad (4.42)$$

$$A_s \geq \min \left\{ \begin{array}{l} \max \left\{ \begin{array}{l} \frac{3 \cdot \sqrt{5801,5}}{60000} \cdot 19,685 \cdot 29,92 = 2,24 \text{ in}^2 = 14,45 \text{ cm}^2 \\ \frac{200}{60000} \cdot 19,685 \cdot 29,92 = 1,96 \text{ in}^2 = 12,64 \text{ cm}^2 \end{array} \right. \\ \frac{4}{3} A_{s(required)} \end{array} \right.$$

For section 1:

$$\frac{4}{3} A_{s,top(required)} = \frac{4}{3} \cdot 46,52 = 62,03 \text{ cm}^2 \quad (4.43)$$

$$\frac{4}{3} A_{s,bottom(required)} = \frac{4}{3} \cdot 42,18 = 56,24 \text{ cm}^2$$

$$\bullet \quad A_{s,top} \geq \min \left\{ \begin{array}{l} \max \left\{ \begin{array}{l} 14,45 \text{ cm}^2 \\ 12,64 \text{ cm}^2 \end{array} \right. \\ 62,03 \text{ cm}^2 \end{array} \right. \quad (4.44)$$

$$\bullet \quad A_{s,bottom} \geq \min \left\{ \begin{array}{l} \max \left\{ \begin{array}{l} 14,45 \text{ cm}^2 \\ 12,64 \text{ cm}^2 \end{array} \right. \\ 56,24 \text{ cm}^2 \end{array} \right.$$

For section 2:

$$\frac{4}{3}A_{s(required)} = \frac{4}{3} \cdot 10,96 = 14,61 \text{ cm}^2$$

$$\bullet \quad A_s \geq \min \begin{cases} \max \begin{cases} 14,45 \text{ cm}^2 \\ 12,64 \text{ cm}^2 = 14,45 \text{ cm}^2 \end{cases} \\ 14,61 \text{ cm}^2 \end{cases}$$

For section 3:

$$\frac{4}{3}A_{s,top(required)} = \frac{4}{3} \cdot 41,87 = 55,83 \text{ cm}^2$$

$$\frac{4}{3}A_{s,bottom(required)} = \frac{4}{3} \cdot 36,33 = 48,44 \text{ cm}^2$$

$$\bullet \quad A_{s,top} \geq \min \begin{cases} \max \begin{cases} 14,45 \text{ cm}^2 \\ 12,64 \text{ cm}^2 = 14,45 \text{ cm}^2 \end{cases} \\ 55,83 \text{ cm}^2 \end{cases}$$

$$\bullet \quad A_{s,bottom} \geq \min \begin{cases} \max \begin{cases} 14,45 \text{ cm}^2 \\ 12,64 \text{ cm}^2 = 14,45 \text{ cm}^2 \end{cases} \\ 48,44 \text{ cm}^2 \end{cases}$$

The maksimum required area of flexural reinforcing:

The reinforcing ratio is the ratio producing balanced strain conditions and it is calculated as follow:

$$\rho_b = \frac{0,85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87000}{87000 + f_y} \right) \quad (4.45)$$

$$\rho_b = \frac{0,85 \cdot 0,75 \cdot 5801,3}{60000} \cdot \left(\frac{87000}{87000 + 60000} \right) = 0,0364$$

$$f'_c = 40 \text{ MPa} = 5801,5 \text{ psi}$$

$$f_y = 413,69 \text{ MPa} = 60000 \text{ psi}$$

The reinforcing ratio p_{max} :

This is the maximum allowable reinforcing ratio and is calculated as follows:

$$p_{max} = 0,85 \cdot \beta_1 \cdot \frac{f'_c}{f_y} \cdot \left(\frac{\epsilon_u}{\epsilon_u + 0,004} \right) \quad (4.46)$$

$$p_{max} = 0,85 \cdot 0,75 \cdot \frac{5801,3}{60000} \cdot \frac{0,003}{0,003 + 0,004} = 0,026$$

Maximum allowable area of reinforcing:

$$A_{s_{max}} = p_{max} \cdot b \cdot d \quad (4.47)$$

$$A_{s_{max}} = 0,026 \cdot 19,685 \cdot 29,92 = 15,31 \text{ in}^2 = 98,77 \text{ cm}^2$$

upper limit of 0.04 times the gross web area on the tension reinforcement:

$$A_s \leq 0,04 \cdot b \cdot d = 0,04 \cdot 50 \cdot 76 = 152 \text{ cm}^2$$

- The beam flexural steel is limited to a maximum given by:

$$A_s \leq 0,025 \cdot b \cdot d = 0,025 \cdot 50 \cdot 76 = 95 \text{ cm}^2 \quad (4.48)$$

- Flexure reinforcement cover : (ACI 318-11, 7.7.1)

For principle reinforcement , for dry conditions according to Table 3.3.2.3 :

$$cover_{min} = 2 \text{ in} \cong 50 \text{ mm}$$

Placement of longitudinal reinforcement was due to the results obtained with the help of ETABS.

Top reinforcement in the middle part of the beam ($3\phi 32$ – Area: $24,13 \text{ cm}^2$) is provided between station point 1,2 m to 2,8 m.)

Bottom reinforcement in the middle part of the beam ($2\phi 32$ – Area: $16,08 \text{ cm}^2$) is provided between station point 1,6 m to 2,8 m

COMBO ID	STATION LOC	TOP STEEL	BOTTOM STEEL	SHEAR STEEL
Comb15	0,0000	4652	4218	3509,18
Comb15	0,4000	3577	3379	3486,43
Comb15	0,8000	2550	2559	3463,68
Comb15	1,2000	1564	1755	3440,93
Comb15	1,2000	1530	1725	3042,86
Comb15	1,6000	1447	1447	3024,66
Comb15	2,0000	1447	1447	3014,76
Comb15	2,4000	1447	1447	3032,97
Comb15	2,8000	1447	1447	3051,17
Comb15	2,8000	1447	1447	3219,13
Comb15	3,2000	1794	1819	3237,33
Comb15	3,6000	2563	2417	3255,54
Comb15	4,0000	3360	3022	3273,74
Comb15	4,4000	4187	3633	3291,94
Comb15	4.4000	4187	3633	3291.94

Figure 4.24: Beam sectioning

According to these results , length of the end reinforcement was taken as $1,5 (l_b + h)$

4.2.1.2 Shear design

The design shear force (ACI 21.5.4.1)

$$V_u = \max\{V_{e1}, V_{e2}\} \quad (4.49)$$

$$V_{e1} = V_{p1} + \frac{V_{D+L}}{2} \quad (4.50)$$

$$V_{e2} = V_{p2} + V_{D+L} \quad (4.51)$$

$$V_{p1} = \frac{M_i^- + M_j^+}{L} \quad (4.52)$$

$$V_{p2} = \frac{M_i^+ + M_j^-}{L} \quad (4.53)$$

M_i^- – Moment capacity at end I, with top steel in tension,

M_j^+ – Moment capacity at end J, with bottom steel in tension

M_i^+ – Moment capacity at end I, with bottom steel in tension

M_j^- – Moment capacity at end J, with top steel in tension

V_{D+L} – Contribution of shear force from the in-span distribution of gravity loads with the assumption that the ends are simply supported

L – Clear span of beam.

Capacity moments:

$$\text{Left: } M_{pos} = 1615,81 \text{ kNm}$$

$$M_{neg} = 1657,94 \text{ kNm}$$

$$\text{Right: } M_{pos} = 1412,35 \text{ kNm}$$

$$M_{neg} = 1507,77 \text{ kNm}$$

$$V_{p1} = \frac{1657,94 + 1412,35}{4,4} = 697,79 \text{ kN}$$

$$V_{p2} = \frac{1615,81 + 1507,77}{4,4} = 709,9 \text{ kN}$$

$$V_{D+L} = -71,62 \text{ kN}$$

$$V_{e1} = 697,79 - \frac{71,63}{2} = 661,97 \text{ kN}$$

$$V_{e2} = 709,9 - 71,63 = 638,27 \text{ kN}$$

$$V_u = \max\{661,97; 638,27\} = 661,97 \text{ kN}$$

The nominal shear strength:

$$V_n = V_c + V_s \quad (4.54)$$

V_c – contribution of concrete, concrete capacity

V_s – contribution of steel

$$V_s = \frac{A_v \cdot f_y \cdot d}{s} \quad (4.55)$$

$$V_c = 0,17\lambda \sqrt{f'_c} \cdot b \cdot d \quad (4.56)$$

$$V_c = 0,17\sqrt{4} \cdot 50 \cdot 76 = 1292 \text{ kN}$$

Maximum shear force is limited:

$$V_{max} = 0,66\lambda \sqrt{f'_c} \cdot b \cdot d \quad (4.57)$$

$$V_{max} = 0,66\sqrt{4} \cdot 50 \cdot 76 = 5016 \text{ kN}$$

$$\frac{A_v}{s} = \frac{V_u}{f_{ys} \cdot d \cdot \phi_s} \quad (4.58)$$

$$\frac{A_v}{s} = \frac{661,97}{41,369 \cdot 76 \cdot 0,75 \cdot 0,6} = 3509,18 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 10 \text{ cm}, 20 \text{ cm}$$

$$A_v = 2 \cdot \frac{\pi \cdot 1,2^2}{4} = 2,26 \text{ cm}^2$$

$$A_v = 4 \cdot \frac{\pi \cdot 1,2^2}{4} = 4,52 \text{ cm}^2$$

For total length: 4,4 m there is 185,32 cm² (39 ties · 4,52 cm² + 4 ties · 2,26 cm²) of transversional reinforcement. It is greater than required amount 154,4 cm² (4,4 · 35,09).

The required spacing has to govern as well conditions:

$$s \leq \begin{cases} \frac{d}{2} \\ \frac{A_v \cdot f_{yt}}{0,75 \sqrt{f'_c} \cdot b} \\ \frac{A_v \cdot f_{yt}}{50 \cdot b} \end{cases} \quad (4.59)$$

$$s \leq \begin{cases} \frac{0,243 \cdot 60000}{0,75 \cdot \sqrt{5801} \cdot 15,7} = 16,25 \text{ in} = 41,3 \text{ cm} \\ \frac{0,243 \cdot 60000}{50 \cdot 15,7} = 18,57 \text{ in} = 47,2 \text{ cm} \end{cases}$$

Reinforcement details

- Spacement of transverse bars

$$s \leq \begin{cases} \frac{d}{4} = \frac{760}{4} = 190 \text{ mm} \approx 200 \text{ mm} \\ 8d_b = 8 \cdot 32 = 256 \text{ mm} \\ 24 \text{ hoop bar diameter} = 384 \text{ mm} \\ 12'' = 30,48 \text{ cm} \end{cases} \quad (4.60)$$

- Transverse reinforcement cover : (ACI 318-11, 7.7.1)

For principle reinforcement , for dry conditions according to Table 3.3.2.3 :

$$cover_{min} = 2 \text{ in} \cong 50 \text{ mm}$$

- Anchorage length for exterior joint:

$$l_d = \frac{f_y \cdot d_b}{5,4 \cdot \sqrt{f'_c}} = \frac{41,369 \cdot 2,6}{5,4 \cdot \sqrt{4}} = 9,69 \text{ cm} \cong 10 \text{ cm} \quad (4.61)$$

$$l_{anchorage} \geq \max \begin{cases} l_d = 10 \text{ cm} \\ 8d_b = 20,8 \text{ cm} \\ 6 \text{ in} = 15,24 \text{ cm} \end{cases}$$

4.2.2 Column Design, C2

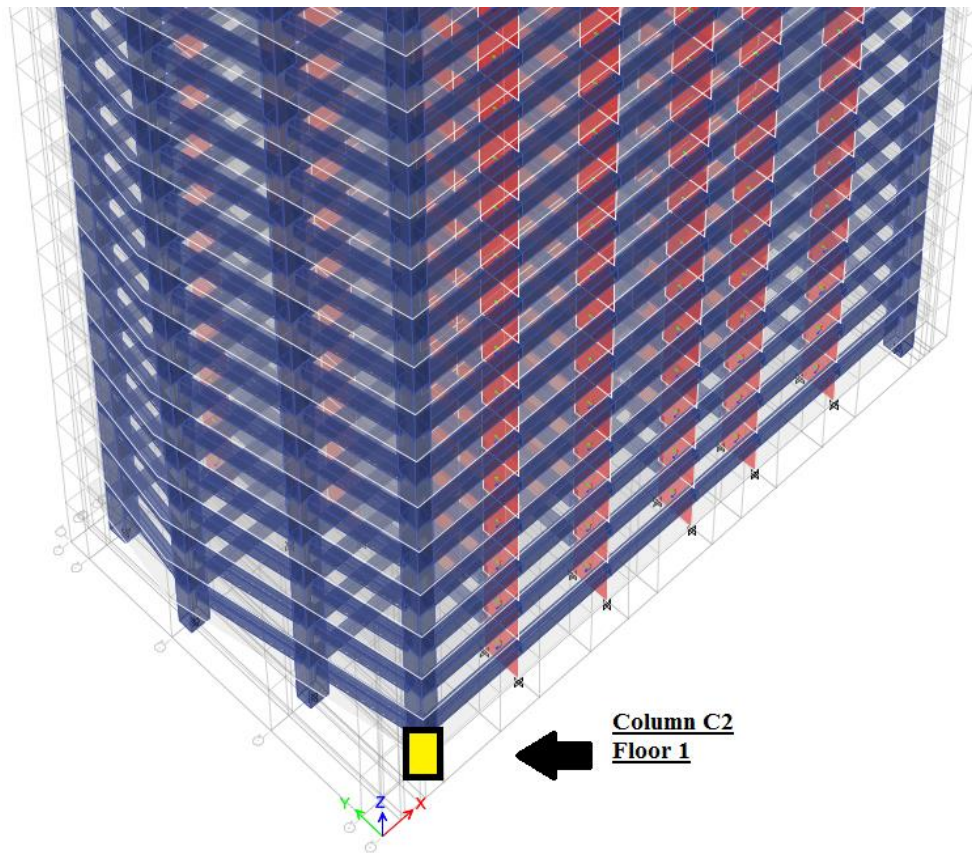


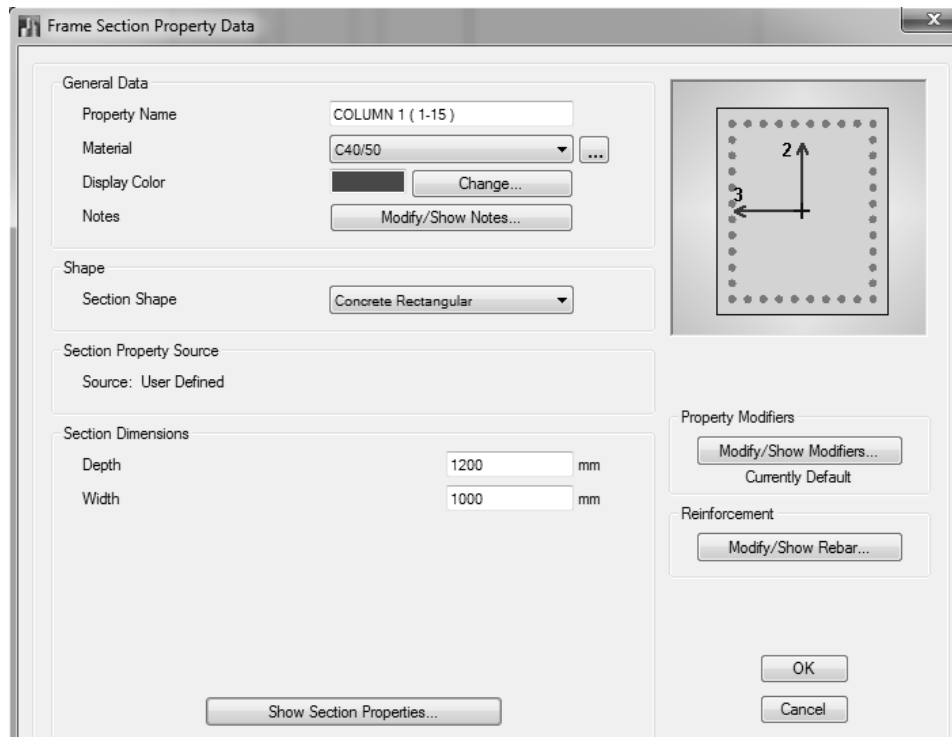
Figure 4.25 : Location of column C2.

With the help of ETABS the required amount of reinforcement to design a column was obtained. Based on provided reinforcing bar configuration assumed in frame profile section menu as visible in the figure below , reinforcement bars are calculated along the stations of the column span.

The design procedure for the reinforced concrete columns of the structure involve generation of the axial force-biaxial moment interaction surfaces for all of the different concrete section

Program generated the interaction surfaces for the range of allowable reinforcement. After that the capacity ratio or the required reinforcing area for the factored axial force and biaxial bending moments is obtained from each loading combination at each station of the column. The target capacity ratio is taken as the Utilization Factor Limit , which is set to 0.95 by default, when calculating the required reinforcing area.

4.2.2.1 Flexure design

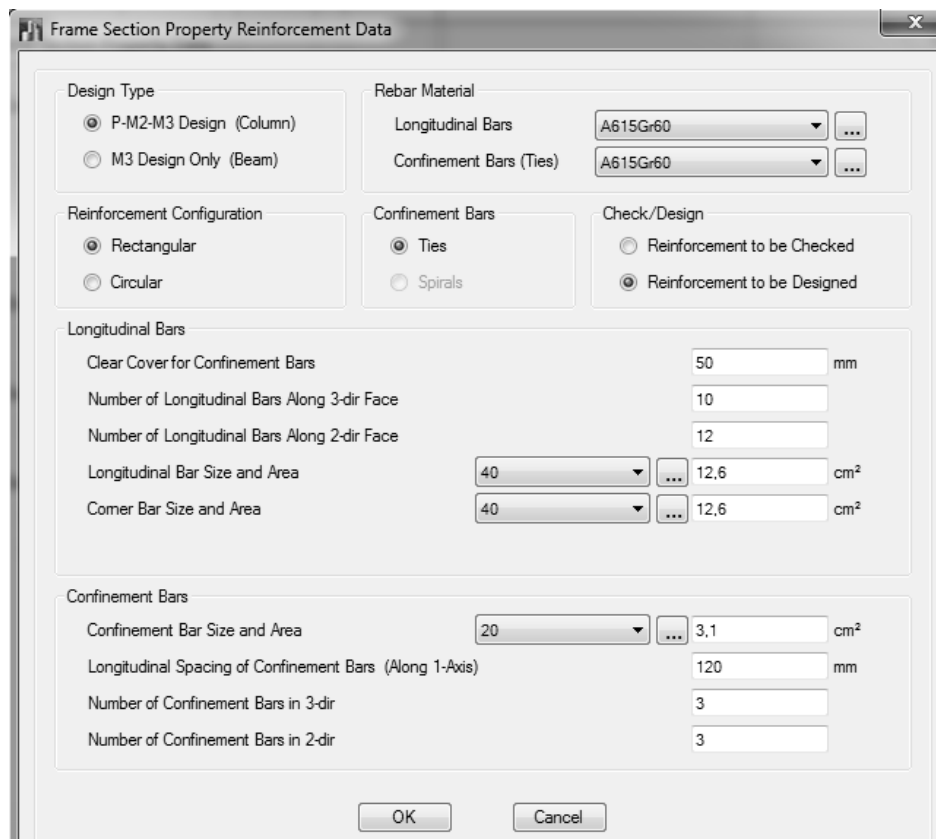


The 'Frame Section Property Data' dialog box is used to define the properties of a frame section. It includes the following sections:

- General Data:** Property Name (COLUMN 1 (1-15)), Material (C40/50), Display Color (black), and Notes (Modify/Show Notes...).
- Shape:** Section Shape (Concrete Rectangular).
- Section Property Source:** Source: User Defined.
- Section Dimensions:** Depth (1200 mm) and Width (1000 mm).
- Property Modifiers:** Modify/Show Modifiers... (Currently Default).
- Reinforcement:** Modify/Show Rebar....

Buttons: OK, Cancel, Show Section Properties...

Figure 4.26: Tensile reinforcement assumptions.



The 'Frame Section Property Reinforcement Data' dialog box is used to define the reinforcement properties of a frame section. It includes the following sections:

- Design Type:** P-M2-M3 Design (Column) (selected), M3 Design Only (Beam).
- Rebar Material:** Longitudinal Bars (A615Gr60), Confinement Bars (Ties) (A615Gr60).
- Reinforcement Configuration:** Rectangular (selected), Circular.
- Confinement Bars:** Ties (selected), Spirals.
- Check/Design:** Reinforcement to be Checked, Reinforcement to be Designed (selected).
- Longitudinal Bars:** Clear Cover for Confinement Bars (50 mm), Number of Longitudinal Bars Along 3-dir Face (10), Number of Longitudinal Bars Along 2-dir Face (12), Longitudinal Bar Size and Area (40 mm, 12.6 cm²), Corner Bar Size and Area (40 mm, 12.6 cm²).
- Confinement Bars:** Confinement Bar Size and Area (20 mm, 3.1 cm²), Longitudinal Spacing of Confinement Bars (Along 1-Axis) (120 mm), Number of Confinement Bars in 3-dir (3), Number of Confinement Bars in 2-dir (3).

Buttons: OK, Cancel.

Figure 4.27: Tensile reinforcement assumptions.

Design internal forces:

Combination 15: $1,2 D + [1,3 (0,3E_x - E_y) + 0,2S_{DS} \cdot D] + 1,0 L + 0,5 S$

$$P_u = 29922,11 \text{ kN}$$

$$M_{u2} = 679,7 \text{ kNm}$$

$$M_{u3} = -2347,98 \text{ kNm}$$

Minimum and maximum longitudinal reinforcement limits: (ACI 10.9.1)

$$A_{st,min} = 0,01 \cdot A_g = 0,01 \cdot 120 \cdot 100 = 120 \text{ cm}^2 \quad (4.62)$$

$$A_{st,max} = 0,06 \cdot A_g = 0,08 \cdot 120 \cdot 100 = 720 \text{ cm}^2 \quad (4.63)$$

40 bars $d = 40 \text{ mm}$ were chosen

$$A_{s,provided} = 502,65 \text{ cm}^2$$

The provided area of longitudinal reinforcement falls between the minimum and maximum limits.

It is convenient to construct interaction diagrams when concrete member is subjected to combined flexure and axial load. This diagram shows the relationship between axial load and bending moments at failure.

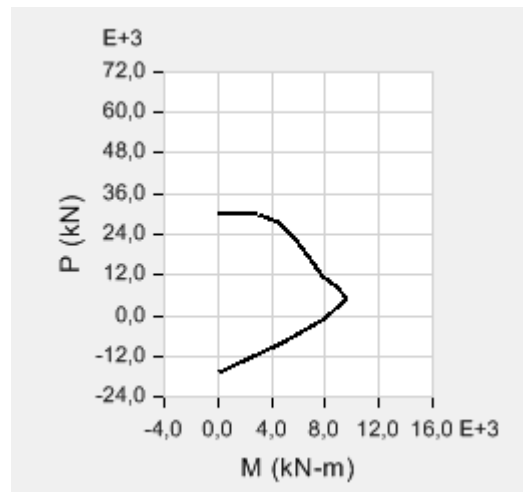


Figure 4.28: Interaction surface for column C2.

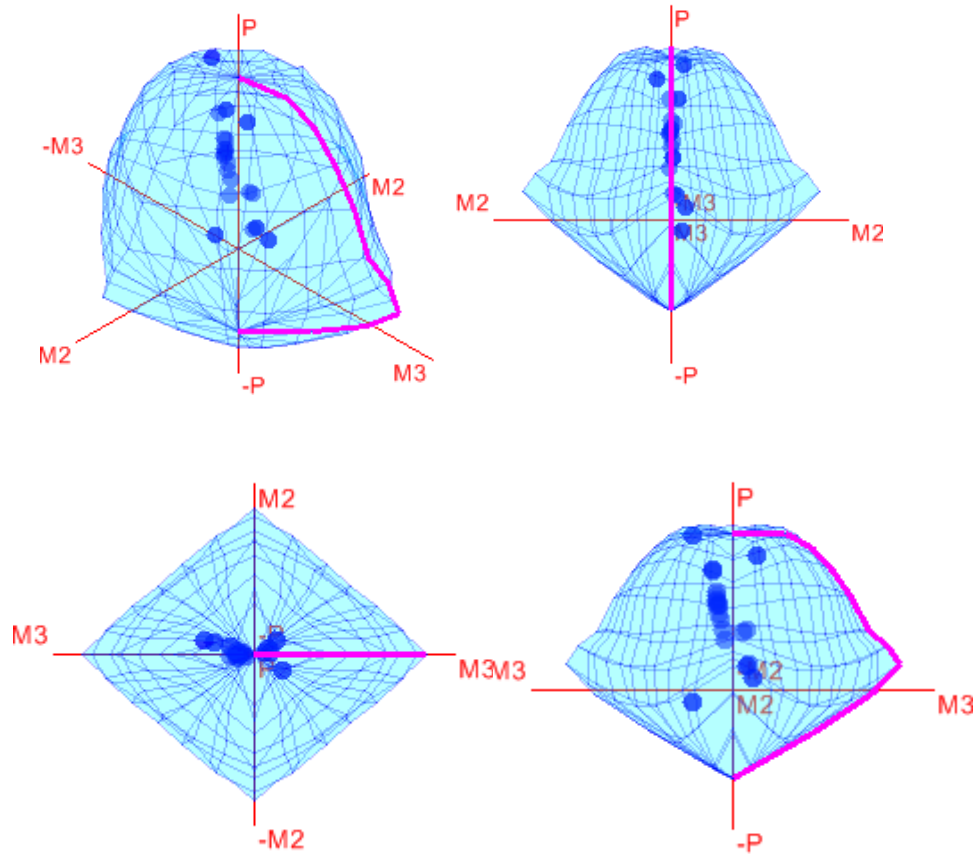


Figure 4.29: Interaction surface for column C2.

All points which represent the internal forces in each combination are inside the volume limited by critical curvature. The most critical is combination 15, as shown below demand/ capacity ratio for column then is equal to 0,997. It means that design of reinforcement fulfil the requirements. However, dimensions of the column can be increased to reduce this ratio (120 cm x 120 cm).

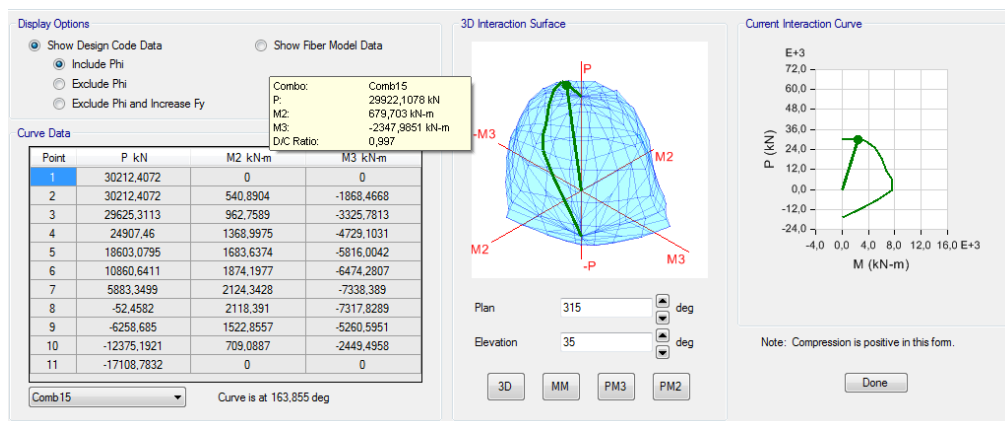


Figure 4.30: Interaction surface for column C2.

4.2.2.2 Shear design:

$$V_u \leq \phi V_n$$

V_u - total shear force applied at a given section

V_n - nominal shear strength

$$V_n = V_c + V_s$$

V_c – nominal shear strength provided by concrete

V_s – nominal shear strength provided by shear reinforcement

Design for V_{u2}, V_{u3} :

Combination 24: (0,724D +0,39 Ex – 1,3Ey)

Major shear V_2 :

$$\text{Design } V_u = 435,90 \text{ kN}$$

$$\text{Design } P_u = -2149,11 \text{ kN}$$

$$\text{Design } M_u = -2278,1 \text{ kNm}$$

$$A_{cv} = (120 - 9) \cdot 100 = 11100 \text{ cm}^2$$

$$A_g = 100 \cdot 120 = 12000 \text{ cm}^2$$

Shear force carried by the concrete:

For a special frame design,

a) if the factored axial compressive force P_u including earthquake effect is small

$$(P_u < \frac{f_c \cdot A_g}{20}) :$$

$$2149,11 \text{ kN} < \frac{4 \cdot 12000}{20} = 2400 \text{ kN}$$

b) if the contribution of shear force from earthquake is more than half of the total factored maximum shear force ,

$$(V_E \geq 0,5 \cdot V_u)$$

$$293,845 \geq 0,5 \cdot 435,9 = 217,95 \text{ kN}$$

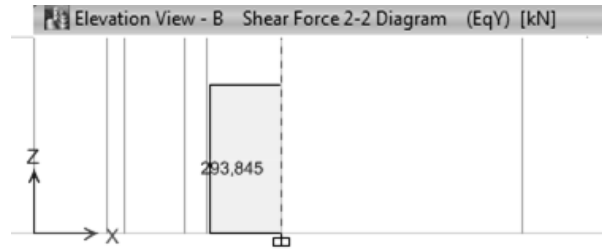


Figure 4.31: Shear force 2-2 diagram.

Then the concrete capacity is equal to 0 (ACI 21.6.5.2)

It is assumed by the most critical case by ETABS, that only the transverse reinforcement will carry the shear stress

$$\phi V_c = 0$$

Then,

$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_{ys} d}$$

$$\frac{A_v}{s} = \frac{(435,9 - 0) \cdot 10^4}{0,6 \cdot 41,369 \cdot 111} = 1582 \frac{mm^2}{m}$$

Provided:

#2 bars (d=6 mm) amount of applied reinforcement smaller than required.

#3 bars were chosen (d=10 mm)

$$s = 12 \text{ cm} , 15 \text{ cm}$$

$$A_v = 5 \cdot \frac{\pi \cdot 1^2}{4} = 3,93 \text{ cm}^2$$

For total height: 3,25 m there is 106,11 cm^2 (27 ties $\cdot$ 3,93 cm^2) of transversional reinforcement. It is greater than required amount 51,42 cm^2 (3,25 $\cdot$ 15,82 cm^2). Condition 21.4.4.1 (b) governs the amount required.

Minor shear V_3 :

$$\text{Design } V_u = 164,46 \text{ kN}$$

$$\text{Design } P_u = -2149,11 \text{ kN}$$

$$\text{Design } M_u = 644,49 \text{ kNm}$$

$$A_{cv} = (100 - 9) \cdot 120 = 10920 \text{ cm}^2$$

$$A_g = 100 \cdot 120 = 12000 \text{ cm}^2$$

Shear force carried by the concrete:

$$a) \quad P_u < \frac{f_c \cdot A_g}{20}$$

$$2149,11 \text{ kN} < \frac{4 \cdot 12000}{20} = 2400 \text{ kN}$$

$$b) \quad V_E \geq 0,5 \cdot V_u$$

$$130,412 \text{ kN} \geq 0,5 \cdot 164,46 = 82,23 \text{ kN}$$

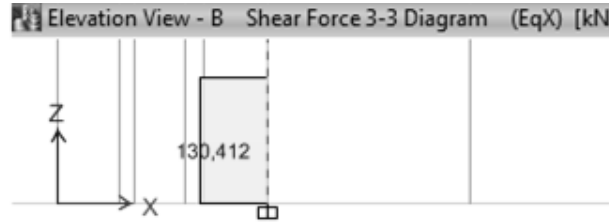


Figure 4.32: Shear force 3-3 diagram.

$$\phi V_c = 0$$

$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_{ys} d}$$

$$\frac{A_v}{s} = \frac{(164,46 - 0) \cdot 10^4}{0,6 \cdot 41,369 \cdot 91} = 728 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 12 \text{ cm}, 15 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

For total height: 3,25 m there is $84,78 \text{ cm}^2$ ($27 \text{ ties} \cdot 3,14 \text{ cm}^2$) of transversional reinforcement. It is greater than required amount $23,66 \text{ cm}^2$ ($3,25 \cdot 7,28$). Condition 21.4.4.1 (b) governs the amount required.

According to confinement requirements the total cross-sectional area of rectangular hoop reinforcement can not be less than required amount by equations:

$$A_{sh} \geq \begin{cases} 0,3 \cdot s \cdot b_c \cdot \left[\left(\frac{A_g}{A_{ch}} \right) - 1 \right] \cdot \left(\frac{f'_c}{f_{yt}} \right) \\ 0,09 \cdot s \cdot b_c \cdot \frac{f'_c}{f_{yt}} \end{cases} \quad (4.64)$$

Transverse reinforcement spacing:

$$s_1 \leq \begin{cases} \frac{b}{4} = \frac{100}{4} = 25 \text{ cm} \\ \frac{h}{4} = \frac{120}{4} = 30 \text{ cm} \\ 6d_b = 6 \cdot 4 = 24 \text{ cm} \\ s_0 = 12 \text{ cm} \end{cases} \quad (4.65)$$

s_1 was chosen as 12 cm.

Center-to-center spacing of transverse reinforcement:

$$100 \text{ mm} \leq s_0 = 100 + \left[\frac{(350 - h_x)}{3} \right] \leq 150 \text{ mm} \quad (4.66)$$

$$h_x = 31 \text{ cm}$$

$$s_0 = 100 + \left[\frac{(350 - 310)}{3} \right] = 120 \text{ mm}$$

$$A_{sh2} \geq \begin{cases} 0,3 \cdot 12 \cdot 110 \cdot \left[\left(\frac{100 \cdot 120}{110 \cdot 90} \right) - 1 \right] \cdot \left(\frac{4}{41,369} \right) = 12,76 \text{ cm}^2 \\ 0,09 \cdot 12 \cdot 110 \cdot \left(\frac{4}{41,369} \right) = 11,49 \text{ cm}^2 \end{cases}$$

$$A_{sh2} = 5 \cdot \frac{\pi \cdot 2^2}{4} = 15,7 \text{ cm}^2$$

$$A_{sh3} \geq \begin{cases} 0,3 \cdot 12 \cdot 90 \cdot \left[\left(\frac{100 \cdot 120}{110 \cdot 91} \right) - 1 \right] \cdot \left(\frac{4}{41,369} \right) = 6,65 \text{ cm}^2 \\ 0,09 \cdot 12 \cdot 91 \cdot \left(\frac{4}{41,369} \right) = 9,50 \text{ cm}^2 \end{cases}$$

$$A_{sh3} = 4 \cdot \frac{\pi \cdot 2}{4} = 12,56 \text{ cm}^2$$

Length, measured from joint face along axis of column over which special transverse reinforcement must be provided: (distance s_1 times number of horizontal ties)

$$l_o \geq \max \begin{cases} \text{larger of } b \text{ or } h = 1200 \text{ mm} \\ \frac{l}{6} = \frac{3250}{6} = 541,7 \text{ mm} \\ 18'' = 457,2 \text{ mm} \cong 460 \text{ mm} \end{cases} \quad (4.67)$$

Center-to-center spacing of longitudinal shear reinforcement:

$$s_2 \leq \begin{cases} 6d_b = 6 \cdot 40 = 240 \text{ mm} \\ 6'' = 152,4 \text{ mm} \end{cases} \quad (4.68)$$

Distance s_2 was chosen as 150 mm

Minimum concrete cover for reinforcement according to table 3.3.2.3 of ACI 318-11.

For beams and column : principal reinforcement : 2 in = 50 mm

Concrete cover provided: 70 mm

4.2.2.3 Splice length of longitudinal reinforcement

ACI 12.17 contains special provisions for splices in columns. A Class A tension lap splice was used.

$$l_d = \left(\frac{3}{40} \cdot \frac{f_y}{\sqrt{f'_c}} \cdot \frac{\Psi_t \cdot \Psi_e \cdot \Psi_s \cdot \lambda}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) \quad (4.69)$$

Ψ_t – reinforcement location factor = 1.0 for other than top bars

Ψ_e – coating factor = 1.0 for uncoated reinforcement

Ψ_s – reinforcement size factor = 1.0 for No.7 and larger bars

λ – lightweight aggregate concrete factor = 1.0 for normal weight concrete

c_b – spacing or cover dimension

K_{tr} – transverse reinforcement index = 0 (conservative)

$$c_b = 6,3 + \frac{2,6}{2} = 7,6 \text{ cm} = 2,98 \text{ in} \quad (\text{governs})$$

$$= \frac{100 - 2 \left(4 + \frac{2,6}{2} \right)}{7} = 12,77 \text{ cm}$$

$$l_d = \left(\frac{3}{40} \cdot \frac{60000}{\sqrt{5801,5}} \cdot \frac{1 \cdot 1 \cdot 1 \cdot 1}{\left(\frac{2,98 + 0}{1,03} \right)} \right) = 20,42 \text{ in} = 51,87 \text{ cm}$$

Class A splice length = $1,0 \cdot l_d \cong 52 \text{ cm}$

Splice located just above the floor level

4.2.3 Shear Wall Design, Core 4- floor 1

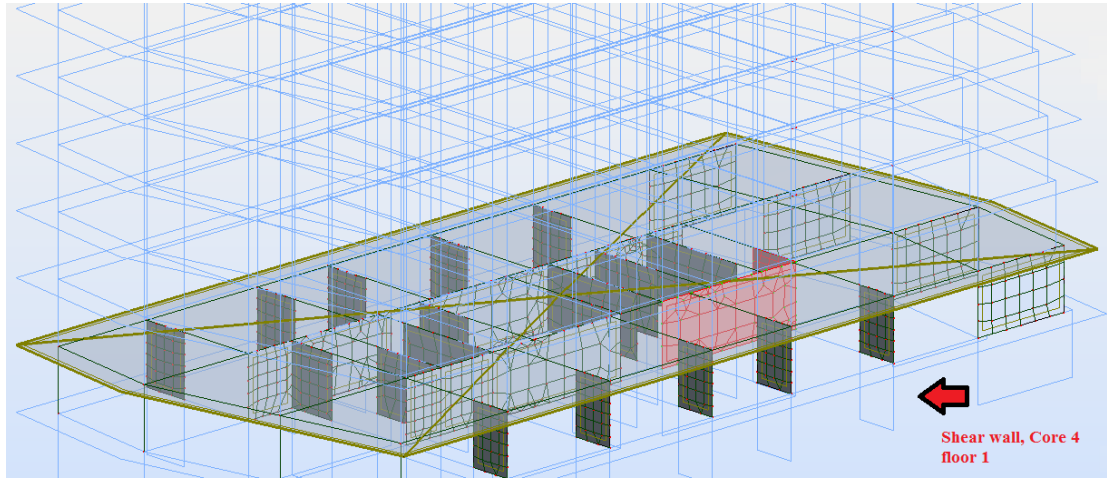


Figure 4.33: Location of Core 4.

4.2.3.1 Arrangement 1

Details for reinforcement were assumed according to arrangement 1 shown on the Figure 4.15

4.2.3.1.1 Design for flexure:

Table 4.7: Flexure design forces for Core 4,ACI 318-11.

Station	Flexural	P_u	M_{u2}	M_{u3}
Location	Combo	kN	kN-m	kN-m
Top	DWal41	8261,662	-14441,982	37120,357
Bottom	DWal41	8666,8173	-19419,541	41226,878

Interaction surface for shear wall, CORE 4 , floor 1:

In program, a three-dimensional interaction surface is defined with reference to the P , M_2 and M_3 axes. The surface is developed using a series of interaction curves that are created by rotating the direction of the pier neutral axis in equally spaced increments around a 360-degree circle.

The program creates a series of interaction surfaces for the pier, based on the size of the pier as specified in Section Designer tool, the location of the specified and size of each reinforcing bar relative to the size of the other bars.

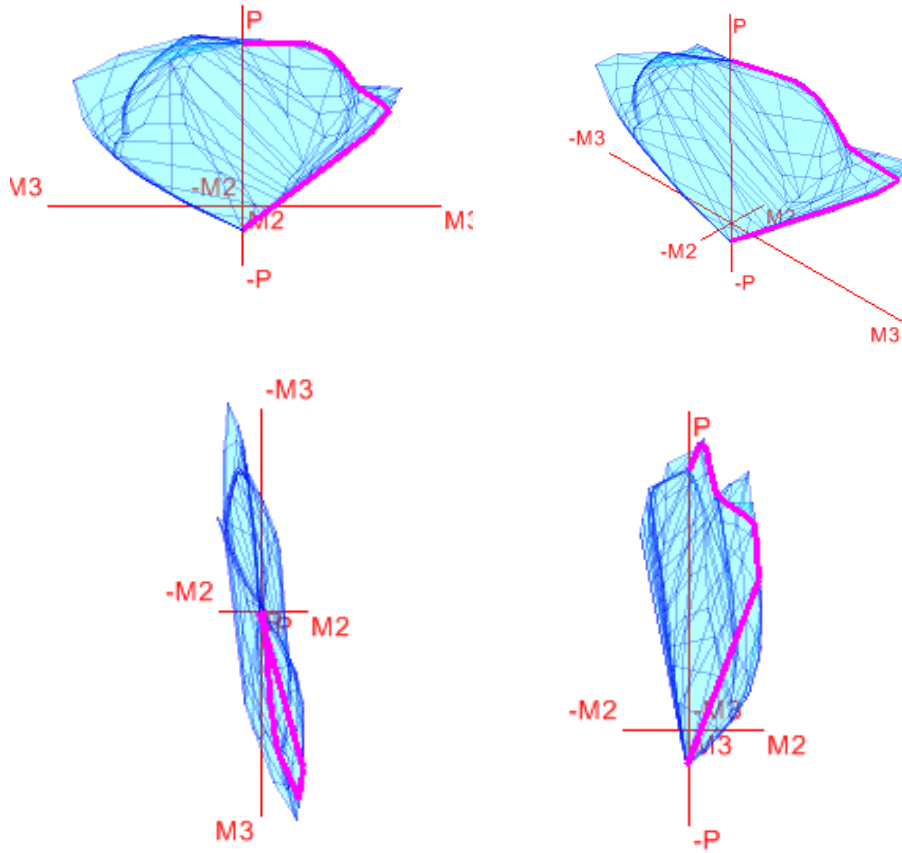


Figure 4.34: Interaction surface for Core 4.

Longitudinal reinforcement limits: (ACI 10.9.1)

$$A_{st,min} = 0,0025 \cdot A_g = 0,0025 \cdot 65912,86 = 164,78 \text{ cm}^2 \quad (4.70)$$

$$A_{st,max} = 0,06 \cdot A_g = 3954,77 \text{ cm}^2 \quad (4.71)$$

$$A_g = 65912,86 \text{ cm}^2$$

$$A_{s,provided} = 1051,24 \text{ cm}^2$$

$$\rho_{top,required} = 0,0093$$

$$\rho_{top,current} = 0,0157$$

$$\rho_{bottom,required} = 0,0138$$

$$\rho_{bottom,current} = 0,0157$$

Reinforcement details and limits:

- Wall cover: (ACI 318-11, 7.7.1)
 $cover_{min} = 19,05 \text{ mm}$ (#26 or smaller bar, interior exposure)
 $cover = 40 \text{ mm} \geq cover_{min} = 19,05 \text{ mm}$

Wall thickness is greater than 10 in. then 2 curtains are required.

- Bar spacing: (ACI 318-11 11.9.9):

$$\frac{\phi V_c}{2} = \frac{4381,39}{2} = 2190,7 \text{ kN}$$

$$V_u = 3613,37 \text{ kN}$$

$$V_u \geq \frac{\phi V_c}{2}$$

Provisions from chapter 14 are applied.

$$s_{1max}; s_{2max} = \min \begin{cases} \frac{l_w}{5} = \frac{9,27}{5} = 1,854 \text{ m} \\ 3h = 9,75 \text{ m} \\ 18 \text{ in} = 0,4572 \text{ m} \end{cases} \quad (4.72)$$

$$s_{1max} = s_{2max} = 457,2 \text{ mm}$$

4.2.3.1.2 Boundary check elements:

- Distance from the extreme compressive fibers to N.A

When the neutral axial depth calculated from the factored axial force and nominal moment strength are consistent with design displacement and exceed the limit Boundary elements are required in the cross section.

$$c \geq \frac{L_w}{600 \left(\frac{\delta_u}{h_w} \right)} \quad (4.73)$$

$$\delta_u = \delta_{u_{elastic}} \cdot \frac{C_d}{I} \quad (4.74)$$

$$\frac{\delta_u}{h_w} \geq 0,007 \quad (4.75)$$

C_d – The deflection amplification factor

I - The Importance factor determined in accordance with Section 11.5.1

FLOOR 1: core 4

For Leg 1:

$$\delta_{u_{elastic}} = 0,098 \text{ m}$$

$$\delta_u = 0,098 \cdot 5 = 0,49$$

$$\frac{\delta_u}{h_w} = \frac{0,49}{145} = 0,003 \rightarrow \frac{\delta_u}{h_w} = 0,007$$

$$c_{limit_1} = \frac{3,2}{600 \cdot 0,007} = 0,7619 \text{ m}$$

$$c = 1,438 \text{ m}$$

For Leg 2:

$$\delta_{u_{elastic}} = 0,01 \text{ m}$$

$$\delta_u = 0,01 \cdot 5 = 0,5$$

$$\frac{\delta_u}{h_w} = \frac{0,5}{145} = 0,006 \rightarrow \frac{\delta_u}{h_w} = 0,007$$

$$c_{limit_2} = \frac{9,27}{600 \cdot 0,007} = 2,207 \text{ m}$$

$$c = 3,682 \text{ m}$$

- Extreme fibers compressive stress limit:

When the extreme fiber compressive stress exceed limit boundary elements are required

$$0,2 \cdot f'_c = 0,2 \cdot 40 = 8 \text{ MPa} \quad (4.76)$$

- Vertical extent (ACI 318-11 21.9.6.2b)

Leg 1:

$$\frac{M_u}{4V_u} = \frac{1999,34}{4 \cdot 30235,55} = 0,017 \text{ m} \quad (4.77)$$

$$h_{BE,min} = \max \begin{cases} \frac{M_u}{4V_u} = 0,017 \text{ m} \\ \leq h_w = 3,25 \text{ m} \end{cases} \quad (4.78)$$

$$h_{BE} = 3,25 \geq h_{BE,min} = 3,25 \text{ m}$$

Leg 2:

$$\frac{M_u}{4V_u} = \frac{-15504,16}{4 \cdot 77200,19} = 0,05$$

$$h_{BE,min} = \max \left\{ \begin{array}{l} \frac{M_u}{4V_u} = 0,05 \text{ m} \\ \leq h_w = 3,25 \text{ m} \end{array} \right.$$

$$h_{BE} = 3,25 \geq h_{BE,min} = 3,25 \text{ m}$$

- Horizontal extent (ACI 318-11 21.9.6.4a)

$$l_{BE,min} = \max \left\{ \begin{array}{l} c - 0,1 \cdot l_w \\ \frac{c}{2} \end{array} \right. \quad (4.79)$$

Leg 1:

$$l_{BE,min} = \max \left\{ \begin{array}{l} 1,438 - 0,1 \cdot 3,2 = 1,118 \text{ m} \cong 1120 \text{ mm} \\ \frac{1,438}{2} = 0,719 \text{ m} \end{array} \right.$$

Leg 2:

$$l_{BE,min} = \max \left\{ \begin{array}{l} 3,682 - 0,1 \cdot 9,27 = 2755 \text{ m} \cong 2800 \text{ mm} \\ \frac{3,682}{2} = 1,841 \text{ m} \end{array} \right.$$

Table 4.8: Extreme fibers compressive stress limmit- ETABS results.

Station	ID	Edge	Governing	P _u	M _u	Stress Comp	Stress Limit	C Depth	C Limit
Location		Length (mm)	Combo	kN	kN-m	MPa	MPa	mm	mm
Top-Left	Leg 1	1068,1	DWal36	30066,601	-781,005	19,71	8	1388,1	761,9
Top-Right	Leg 1	1071,3	DWal36	30066,601	870,994	19,81	8	1391,3	761,9
Top-Left	Leg 2	2651,5	DWal37	76710,762	-8801,4541	17,78	8	3578,5	2207,1
Top-Right	Leg 2	2607,3	DWal37	76710,762	5183,6863	17,27	8	3534,3	2207,1
Bottom- Left	Leg 1	1116,4	DWal36	30235,551	1930,1109	21,16	8	1436,4	761,9
Bottom- Right	Leg 1	1118,8	DWal36	30235,551	1999,3443	21,24	8	1438,8	761,9
Bottom- Left	Leg 2	2755,4	DWal37	77200,189	15504,161	18,82	8	3682,4	2207,1
Bottom- Right	Leg 2	2705,5	DWal37	77200,189	11410,342	18,25	8	3632,5	2207,1

According to the result shown above Boundary Element were introduced into a wall cross section. Extreme values in both of legs were taken as a its edge length: 1120 mm and 2800 mm respectively.

4.2.3.1.3 Design for shear

The wall pier shear reinforcing is designed leg by leg (panel by panel) for each of the design load combinations. The following steps are involved in designing the shear reinforcing for a particular wall pier section for a particular design loading combination.

Table 4.9: The factored forces that are acting on a leg of the wall pier section.

Station Location	ID	Shear Combo	P _u kN	M _u kN-m	V _u kN
Top	Leg 1	DWal30	-6599,2127	-14247,831	2170,3957
Top	Leg 2	DWal36	35997,251	28906,184	5947,166
Bottom	Leg 1	DWal32	-6482,2473	-21301,617	2170,3957
Bottom	Leg 2	DWal36	36486,678	47237,671	5947,166

- The shear force that can be carried by the concrete of the leg (panel)
- Given the design force set P_u , M_u and V_u acting on a wall pier section, the shear force carried by the concrete, V_c , is calculated using the minimum from the following two equations (ACI 11.9.6).

$$V_c = 0,27 \sqrt{f'_c} t_p \cdot (0,8 L_p) + \frac{N_u \cdot d}{4L_p} \quad (4.80)$$

$$V_c \geq \left[0,05 \lambda \sqrt{f'_c} + \frac{L_p \left(0,1 \lambda \sqrt{f'_c} + 0,2 \frac{N_u}{L_p t_p} \right)}{\left(\frac{M_u}{V_u} \right) - \frac{L_p}{2}} \right] t_p d \quad (4.81)$$

N_u is the axial force, and N_u is positive for compression and negative for tension.

$$d = 0,8 \cdot L_p \quad (4.82)$$

$$LEG 1 : TOP \quad \phi V_c = 198,25 \text{ kN}$$

$$LEG 2 : TOP \quad \phi V_c = 4381,39 \text{ kN}$$

$$LEG 1 : BOTTOM \quad \phi V_c = 244,96 \text{ kN}$$

$$LEG 2 : BOTTOM \quad \phi V_c = 4381,39 \text{ kN}$$

- The required shear reinforcing to carry the balance of the shear :

$$\text{If: } \phi V_c < V_u \leq \phi V_{max}$$

Then:

$$\frac{A_v}{s} = \frac{(V_u - \phi V_c)}{\phi f_{ys} d} \quad (4.83)$$

- LEG 1 : TOP

$$\phi V_c = 198,25 \text{ kN}$$

$$\phi V_n = 2170,4 \text{ kN}$$

$$\frac{A_v}{s} = \frac{(2170,4 - 198,25)}{0,75 \cdot 41,369 \cdot 0,8 \cdot 320} = 2482,9 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 12 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

For total height: 3,25 m there is 81,64 cm^2 ($26 \cdot 3,14 \text{ cm}^2$) of transversional reinforcement. It is greater than required amount 80,7 cm^2 ($3,25 \cdot 24,83 \text{ cm}^2$).

- LEG 2 : TOP

$$\phi V_c = 4381,39 \text{ kN}$$

$$\phi V_n = 7257,54 \text{ kN}$$

$$\frac{A_v}{s} = \frac{(7257,54 - 4381,39)}{0,75 \cdot 41,369 \cdot 0,8 \cdot 927} = 1250 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 20 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

For total height: 3,25 m there is 50,24 cm^2 ($16 \cdot 3,14 \text{ cm}^2$) of transversional reinforcement. It is greater than required amount 40,63 cm^2 ($3,25 \cdot 12,5 \text{ cm}^2$).

- LEG 1 : BOTTOM

$$\phi V_c = 244,96 \text{ kN}$$

$$\phi V_n = 2210,74 \text{ kN}$$

$$\frac{A_v}{s} = \frac{(2210,74 - 244,96)}{0,75 \cdot 41,369 \cdot 0,8 \cdot 320} = 2424,14 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 12 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

For total height: 3,25 m there is $81,64 \text{ cm}^2$ ($26 \cdot 3,14 \text{ cm}^2$) of transversional reinforcement. It is greater than required amount $78,78 \text{ cm}^2$ ($3,25 \cdot 24,24 \text{ cm}^2$).

- LEG 2 : BOTTOM

$$\phi V_c = 4381,39 \text{ kN}$$

$$\phi V_n = 7257,54 \text{ kN}$$

$$\frac{A_v}{s} = \frac{(7257,54 - 4381,39)}{0,75 \cdot 41,369 \cdot 0,8 \cdot 927} = 1250 \frac{\text{mm}^2}{\text{m}}$$

Provided:

$$s = 20 \text{ cm}$$

$$A_v = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

For total height: 3,25 m there is $50,24 \text{ cm}^2$ ($16 \cdot 3,14 \text{ cm}^2$) of transversional reinforcement. It is greater than required amount $40,63 \text{ cm}^2$ ($3,25 \cdot 12,5 \text{ cm}^2$).

- Required development length- vertical wall bars:

(Chapter 12 ACI 318-11)

$$\Psi_t = 1 \text{ (bars are not horizontal)}$$

$$\Psi_e = 1 \text{ (bars not epoxy coated)}$$

$$\Psi_s = 0,8 \text{ (bars are \#6 or smaller)}$$

$$\lambda = 1 \text{ (normal weight concrete)}$$

$$\text{cover} + \frac{d_b}{2} = 4 + \frac{1,6}{2} = 4,8 \text{ cm}$$

$$c_b = 4,8 \text{ cm}$$

$K_r = 0$ (no transverse reinforcement)

$$\frac{c_b + K_r}{d_b} = \frac{4,8}{1,6} = 3 \leq 2,5 \quad (4.84)$$

$$l_d = \left(\frac{f_y}{1,1\lambda\sqrt{f'_c}} \cdot \frac{\Psi_t \cdot \Psi_e \cdot \Psi_s}{2,5} \right) d_b = \left(\frac{41,369}{1,1\sqrt{4}} \cdot \frac{1 \cdot 1 \cdot 0,8}{2,5} \right) \cdot 1,6 \quad (4.85)$$

$$= 9,63 \text{ cm}$$

(Chapter 21 ACI 318-11)

$$l_d = \frac{f_y \cdot d_b}{5,4 \cdot \sqrt{f'_c}} = \frac{41,369 \cdot 1,6}{5,4 \cdot \sqrt{4}} = 6,13 \text{ cm}$$

- Minimum Tie area (ACI 318-11 21.9.6.4c) –Web: Ties parallel to wall

Leg 1:

$$A_{sh} = 0,09 \cdot s \cdot h_x \cdot \frac{f'_c}{f_{yh}} = 0,09 \cdot 12 \cdot 12,5 \cdot \frac{4}{41,369} = 1,31 \text{ cm}^2 \quad (4.86)$$

$$A_{provided} = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

Leg 2:

$$A_{sh} = 0,09 \cdot s \cdot h_x \cdot \frac{f'_c}{f_{yh}} = 0,09 \cdot 20 \cdot 12,5 \cdot \frac{4}{41,369} = 2,18 \text{ cm}^2$$

$$A_{provided} = 4 \cdot \frac{\pi \cdot 1^2}{4} = 3,14 \text{ cm}^2$$

- Minimum Tie area (ACI 318-11 21.9.6.4c) –Web: Ties perpendicular to wall

Leg 1:

$$A_{sh} = 0,09 \cdot s \cdot h_x \cdot \frac{f'_c}{f_{yh}} = 0,09 \cdot 12 \cdot 35,8 \cdot \frac{4}{41,369} = 3,74 \text{ cm}^2$$

$$A_{provided} = 5 \cdot \frac{\pi \cdot 1^2}{4} = 3,93 \text{ cm}^2$$

Leg 2:

$$A_{sh} = 0,09 \cdot s \cdot h_x \cdot \frac{f'_c}{f_{yh}} = 0,09 \cdot 20 \cdot 38,8 \cdot \frac{4}{41,369} = 6,75 \text{ cm}^2$$

$$A_{provided} = 11 \cdot \frac{\pi \cdot 1^2}{4} = 8,64 \text{ cm}^2$$

- Wall Web Horizontal Reinforcement Anchoring (ACI 318-11 21.9.6.4e)

Leg 1, Leg 2:

Horizontal reinforcement in the wall web should not extend to within 150 mm of wall end.

Requirement for bars to be anchored to develop:

$\psi_e = 1$ (uncoated hooked bars)

$\lambda = 1$ (normal weight concrete)

$$l_{ah} = \psi_e \frac{f_y}{2,1\lambda\sqrt{f'_c}} \cdot d_b = 1 \cdot \frac{41,369}{2,1 \cdot 1 \cdot \sqrt{4}} \cdot 2 = 19,7 \text{ cm} \quad (4.87)$$

$$l_{ah} = 19,7 \text{ cm} \geq \begin{cases} 8d_b = 8 \cdot 1 = 8 \text{ cm} \\ 6 \text{ in} = 15,24 \text{ cm} \end{cases}$$

Bars are hooked, so the requirement for anchorage with hooks or heads is satisfied.

4.2.4 Story drift

Similar to calculations made with help of Eurocode about SLS, drift control checking was conducted.

Drift in building frames is a result of flexural and shear mode contributions, due to the column axial deformations and to the diagonal and girder deformations. The intent is to limit the interstory drift to a reasonable value, beyond which it will be thought that the structure may experience loss of vertical stability.

According to Table 12.12-1 of ASCE 7-10, allowable drift limits were obtained as follow:

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less (etc.)	$0.025h_{sx}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Figure 4.35: Allowable drift limmits for ACI 318-11

Table 4.10: Allowable drift limit check in X direction- ASCE 7-10.

Story	Load Case/Combo	Item	Drift ratio	Δs	Max inelastic response displacement	Allowable limmit
Story45	EqX Max	Max Drift X	0,000713	0,0023173	0,0139	0,065
Story44			0,000729	0,0023693	0,0142	0,065
Story43			0,000741	0,0024083	0,0144	0,065
Story42			0,000752	0,002444	0,0147	0,065
Story41			0,000764	0,002483	0,0149	0,065
Story40			0,000774	0,0025155	0,0151	0,065
Story39			0,000783	0,0025448	0,0153	0,065
Story38			0,00079	0,0025675	0,0154	0,065
Story37			0,000796	0,002587	0,0155	0,065
Story36			0,000806	0,0026195	0,0157	0,065
Story35			0,000816	0,002652	0,0159	0,065
Story34			0,000824	0,002678	0,0161	0,065
Story33			0,000831	0,0027008	0,0162	0,065
Story32			0,000837	0,0027203	0,0163	0,065
Story31			0,000839	0,0027268	0,0164	0,065
Story30			0,000842	0,0027365	0,0164	0,065
Story29			0,000844	0,002743	0,0165	0,065
Story28			0,000844	0,002743	0,0165	0,065
Story27			0,000844	0,002743	0,0165	0,065
Story26			0,000842	0,0027365	0,0164	0,065
Story25			0,00084	0,00273	0,0164	0,065
Story24			0,000837	0,0027203	0,0163	0,065
Story23			0,000834	0,0027105	0,0163	0,065
Story22			0,00083	0,0026975	0,0162	0,065
Story21			0,000825	0,0026813	0,0161	0,065
Story20			0,00082	0,002665	0,0160	0,065
Story19			0,000814	0,0026455	0,0159	0,065
Story18			0,000808	0,002626	0,0158	0,065
Story17			0,000801	0,0026033	0,0156	0,065
Story16			0,000792	0,002574	0,0154	0,065
Story15			0,000784	0,002548	0,0153	0,065
Story14			0,000774	0,0025155	0,0151	0,065
Story13			0,000763	0,0024798	0,0149	0,065
Story12			0,00075	0,0024375	0,0146	0,065
Story11			0,000735	0,0023888	0,0143	0,065
Story10			0,000716	0,002327	0,0140	0,065
Story9			0,000694	0,0022555	0,0135	0,065
Story8			0,000668	0,002171	0,0130	0,065
Story7			0,000635	0,0020638	0,0124	0,065
Story6			0,000596	0,001937	0,0116	0,065
Story5			0,000547	0,0017778	0,0107	0,065
Story4			0,000487	0,0015828	0,0095	0,065
Story3			0,000412	0,001339	0,0080	0,065
Story2			0,000317	0,0010303	0,0062	0,065
Story1			0,000169	0,0005493	0,0033	0,065

Table 4.11: Allowable drift limit check in Y direction- ASCE 7-10.

Story	Load Case/Combo	Item	Drift ratio	Δs	Max inelastic response displacement	Allowable limit
Story45	EqY Max	Max Drift Y	0,00212	0,00688	0,041	0,065
Story44			0,00219	0,00710	0,043	0,065
Story43			0,00226	0,00734	0,044	0,065
Story42			0,00233	0,00758	0,045	0,065
Story41			0,00241	0,00783	0,047	0,065
Story40			0,00248	0,00807	0,048	0,065
Story39			0,00255	0,00829	0,050	0,065
Story38			0,00261	0,00849	0,051	0,065
Story37			0,00267	0,00866	0,052	0,065
Story36			0,00271	0,00881	0,053	0,065
Story35			0,00275	0,00893	0,054	0,065
Story34			0,00278	0,00903	0,054	0,065
Story33			0,00280	0,00910	0,055	0,065
Story32			0,00282	0,00916	0,055	0,065
Story31			0,00282	0,00917	0,055	0,065
Story30			0,00283	0,00919	0,055	0,065
Story29			0,00283	0,00921	0,055	0,065
Story28			0,00284	0,00921	0,055	0,065
Story27			0,00283	0,00921	0,055	0,065
Story26			0,00283	0,00920	0,055	0,065
Story25			0,00283	0,00918	0,055	0,065
Story24			0,00282	0,00916	0,055	0,065
Story23			0,00281	0,00913	0,055	0,065
Story22			0,00280	0,00910	0,055	0,065
Story21			0,00279	0,00905	0,054	0,065
Story20			0,00277	0,00901	0,054	0,065
Story19			0,00276	0,00895	0,054	0,065
Story18			0,00274	0,00889	0,053	0,065
Story17			0,00272	0,00883	0,053	0,065
Story16			0,00269	0,00874	0,052	0,065
Story15			0,00267	0,00867	0,052	0,065
Story14			0,00264	0,00858	0,051	0,065
Story13			0,00261	0,00849	0,051	0,065
Story12			0,00258	0,00838	0,050	0,065
Story11			0,00254	0,00824	0,049	0,065
Story10			0,00249	0,00808	0,048	0,065
Story9			0,00243	0,00788	0,047	0,065
Story8			0,00235	0,00763	0,046	0,065
Story7			0,00225	0,00730	0,044	0,065
Story6			0,00212	0,00688	0,041	0,065
Story5			0,00195	0,00634	0,038	0,065
Story4			0,00173	0,00563	0,034	0,065
Story3			0,00145	0,00470	0,028	0,065
Story2			0,00106	0,00345	0,021	0,065
Story1			0,00046	0,00151	0,009	0,065

For occupancy category I and type of structure drift limit is the 20% of the storey height.

- Design story drift ratio — Relative difference of design displacement between the top and bottom of a story, divided by the story height.

$$\text{Drift ratio} = \frac{\Delta s}{h_{st}} \quad (4.88)$$

- The interstory drift Δs : for this structure that does not have plan irregularity, the drift at story level x is determined by subtracting the design earthquake displacement at the bottom of the story from the design earthquake displacement at the top of the story.

$$\Delta s = s_h - s_{h-1} \quad (4.89)$$

- Max inelastic response displacement: the interstory drift multiplied by seismic load reduction factor.

$$\Delta m = \Delta s \cdot R \quad (4.90)$$

- Allowable limit: Based on figure 4.35, limitation for drifts were assumed as 20 % of the story height below considered level.

$$\Delta s_{Rd} = 0,02 \cdot h_{sh} \quad (4.91)$$

If $\Delta s \cdot R \leq \Delta s_{Rd}$, then drift requirements fulfill conditions of ASCE 7-10.

It can be seen from tables and figures below, that the displacement shape is relatively linear . Lateral drifts obtained from the prescribed lateral forces are less than the limiting values.

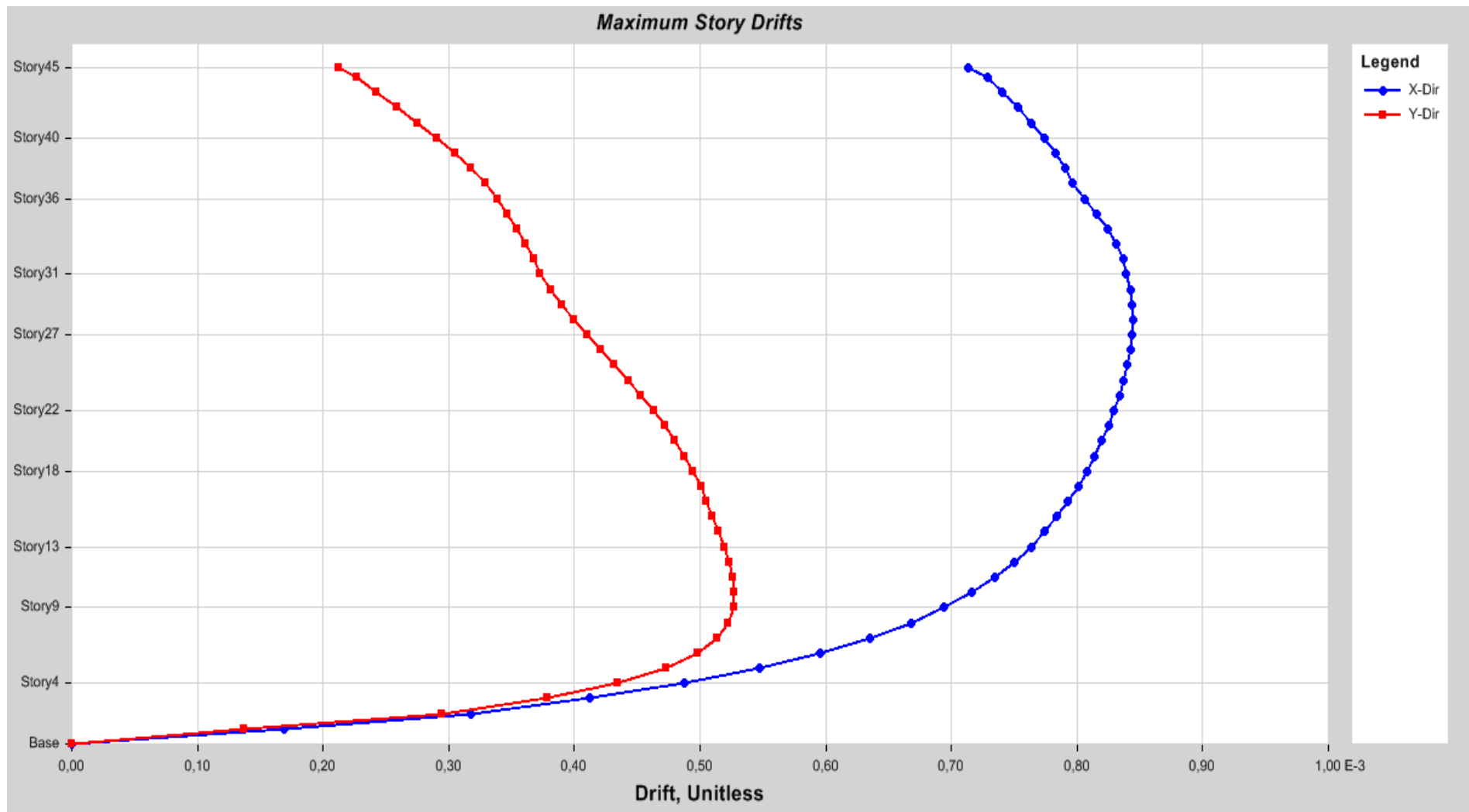


Figure 4.36:Critical drifts for storeys due to EqX load case– ACI 318-11.

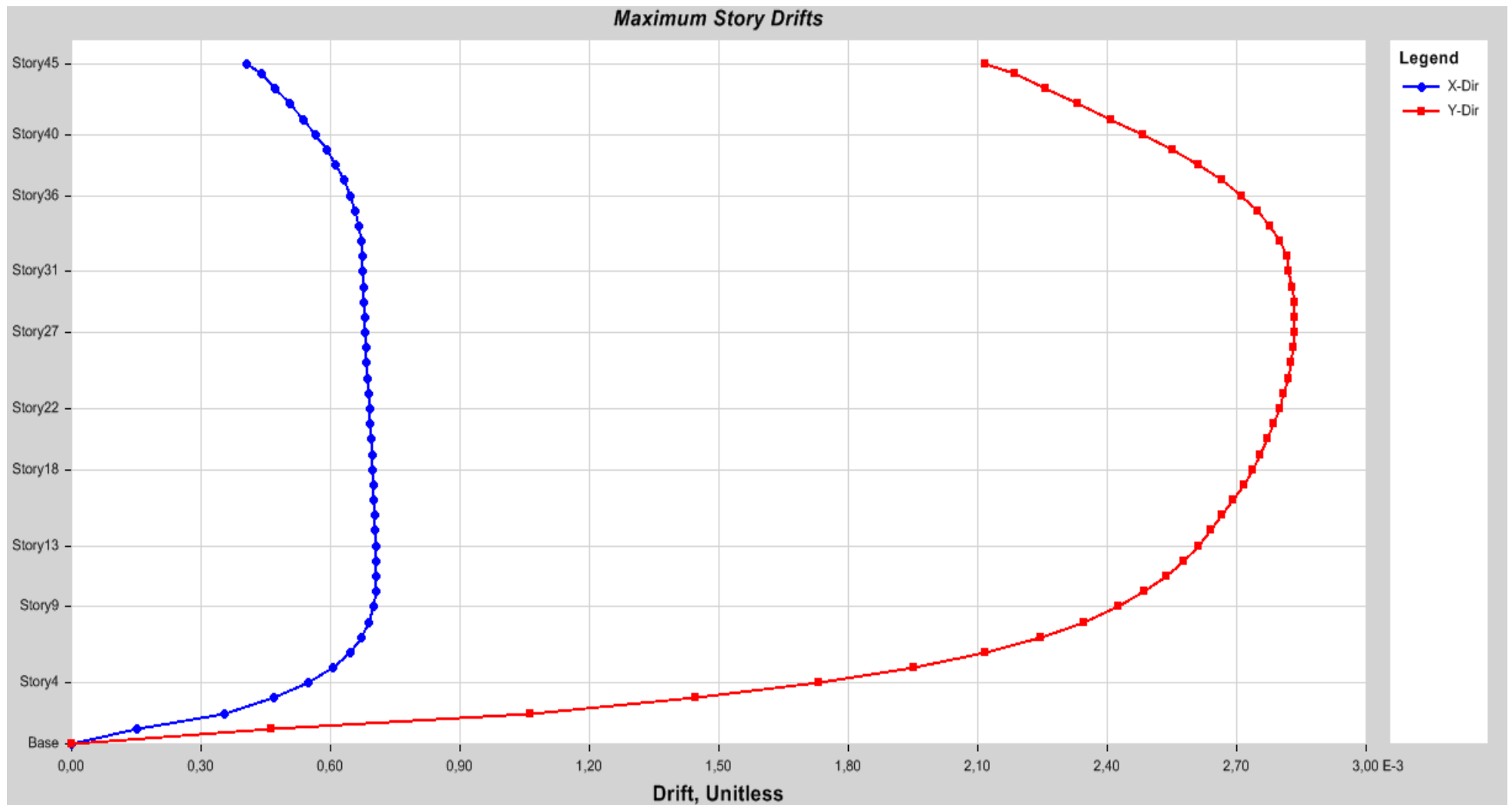


Figure 4.37:Critical drifts for storeys due to EqY load case – ACI 318-11.

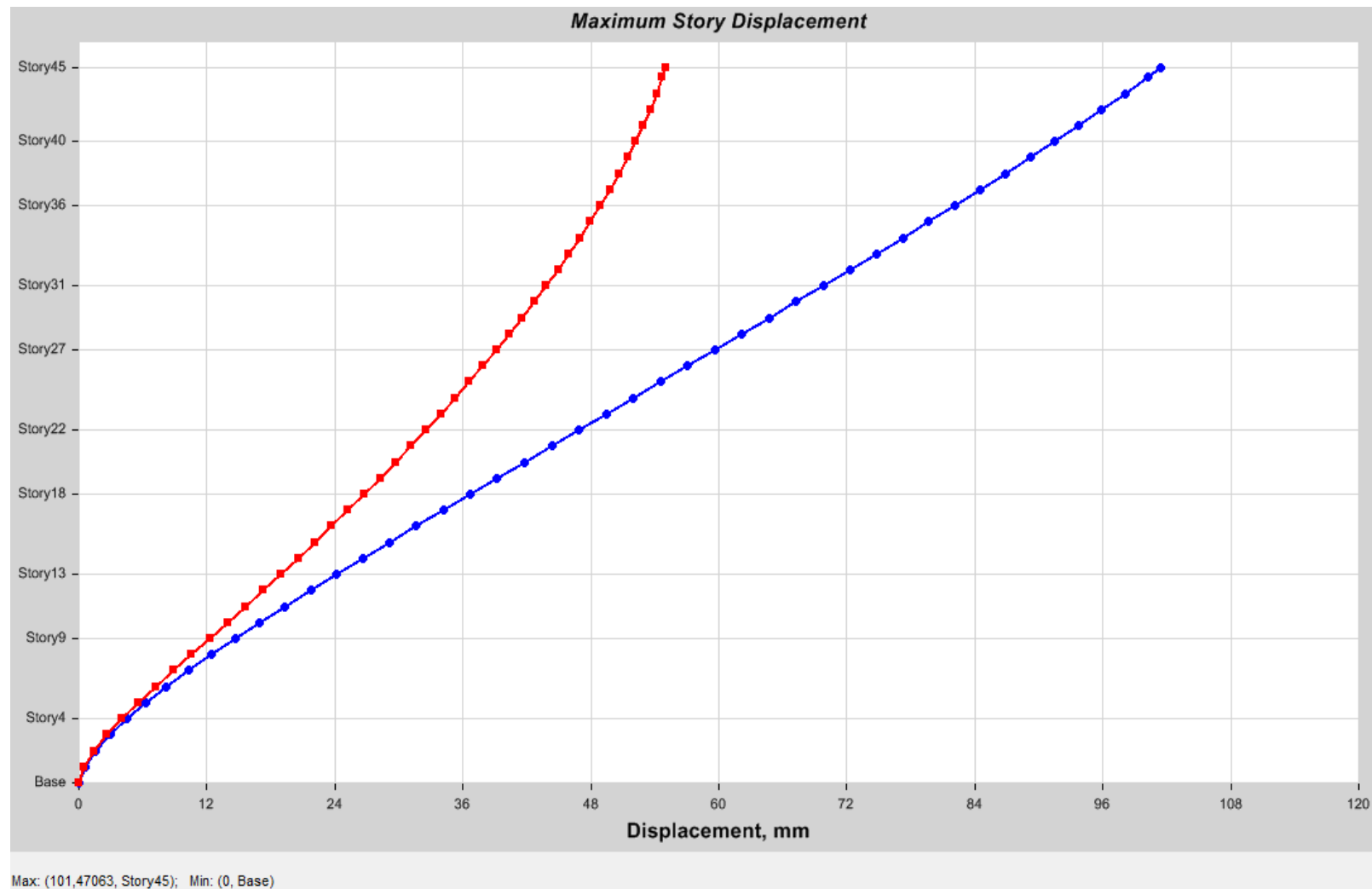


Figure 4.38: Maximum story displacement due to earthquake load in X direction- ACI 318-11.

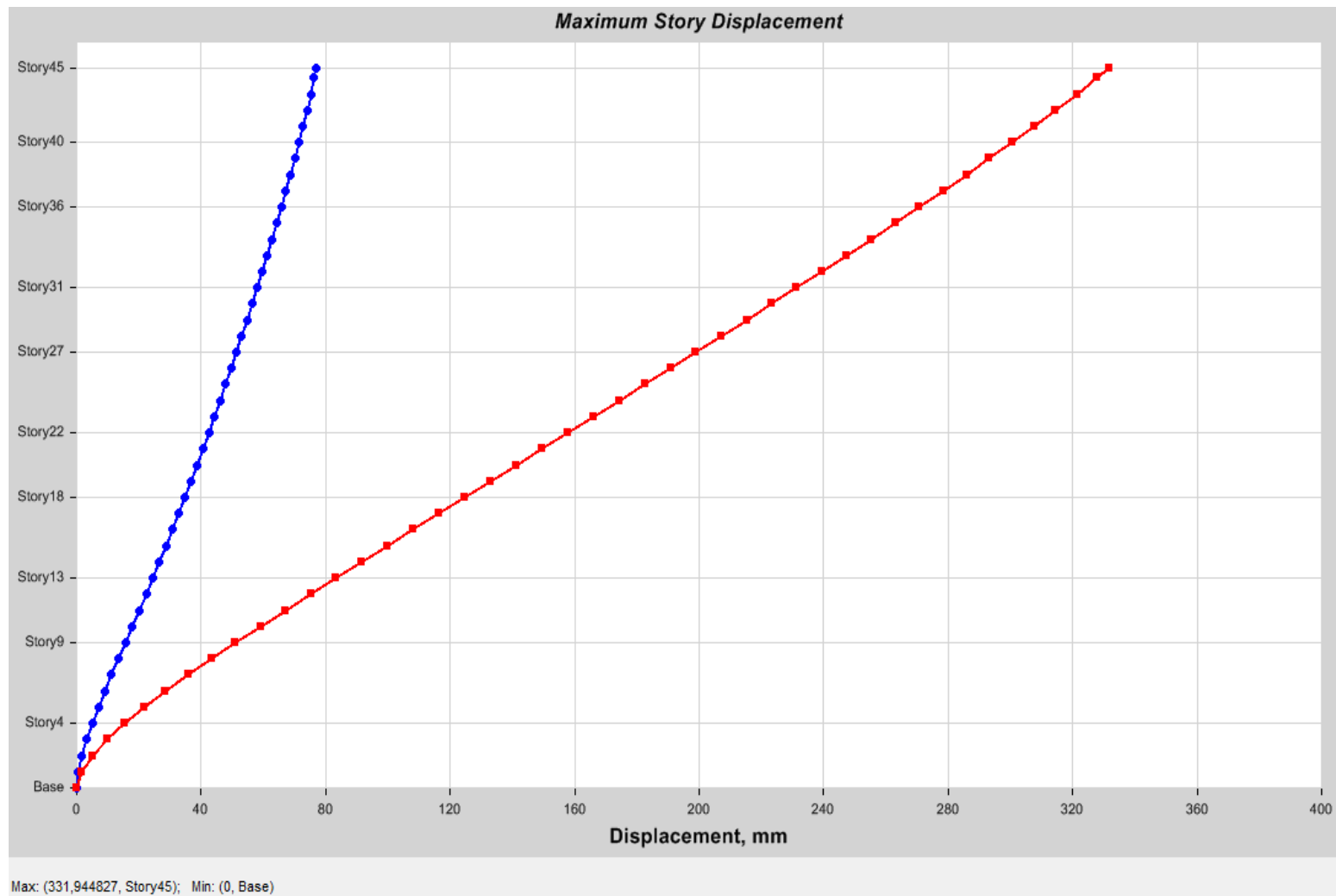


Figure 4.39: Maximum story displacement due to earthquake load in Y direction- ACI 318-11.

5. COMPARISON OF RESULTS

5.1 Dead load

The same value of the dead load for a building were taken into analysis conducted in both regulations. The characteristic values of loading for floor, roof were defined on the cross section with description of the lawers thicknes and calculated in tabels in previous chapters. As a result value $5,4 \frac{kN}{m^2}$ was defined.

5.2 Live Load

Table 5.1: Live load comparison.

Category	Eurocode	ASCE
	Rooms in residential buildings and houses, bedrooms in hotel ect	Residential: Hotels and multifamily houses
Live load	2 kN/m ²	1,92 kN/m ²
Partitions	0,8 kN/m ²	0,72 kN/m ²
Total	2,8 kN/m ²	2,64 kN/m ²

In Eurocode 1991-1-1 live load $2 \frac{kN}{m^2}$ was taken as for category: Areas for domestic and residential activities (floors). Partitions load with self weight not exceeding 2 kN/m contribute in total live loading with value $0,8 \frac{kN}{m^2}$. In ASCE 7-10 ,according to the occupancy and use category of design building defined as residential live load was taken as $1,92 \frac{kN}{m^2}$. Provision for partitions weight were made and their loading defined as $0,72 \frac{kN}{m^2}$.

As a result the difference between live load taken to analysis is approximately 5,7

5.3 Wind load

Comparisson of some coefficients used in calculation, divided into few categories is shown in the table below.

Table 5.2: Wind load comparison.

Category	EUROCODE			ASCE		
	Type	Value		Type	Value	
		N-S direction	W-E direction		N-S direction	W-E direction
Main characteristic:	Vb,0	Fundamental value of basic wind velocity	42 m/s	V	Basic wind speed	42 m/s
	Averaging time for basic wind velocity		10 min	Averaging time for basic wind velocity		3 s
	Averaging time for design velocity at reference height		1 hour	Averaging time for design velocity at reference height		1 hour
	Reference height for gust factor		0,6 h	Reference height for gust factor		0,6 h
	Cdir		1	Kd		1
Topography	Kr		0,23	Kz		1
	Cv(zs)		1,03			
	Co(zs)		1			
Serviceability			Category of building		II	
			I		1	
Peak and gust effect	CsCd		0,92	Gf		2,18
	B²		0,51	Q		0,814
	R²		0,45	R		1,2
	Iv		0,22	Izt		0,14
Aerodynamic coefficients	Cpe		0,8	Cp		0,8
			-0,7			-0,5
			-1,2 ; -0,8			-0,7
Results	Design wind force		24903,12 kN	Design wind force		25382,78 kN

The same values of basic wind speed in analysis were assumed based on the TS 498.

Averaging times for wind velocity vary between the standards.

These differences are between averaging time affect the intermediary parameters and resulting responses. Wind pressure in both standards is calculated using the

multiplication of wind velocity and air density. The used wind velocity differs. ASCE 7-10 uses a gust wind velocity averaged over a period of 3s and Eurocode uses fundamental wind velocity averaged over 10 minutes, increased with factor to 3s mean wind velocity. The effects of topography are expressed in Eurocode mainly by roughness factor, terrain factor and orography factor which control situations in which surrounding area don't increase wind velocity. However, ASCE uses just one coefficient, namely topographic factor. Because building is situated on level ground and not on hill its value is 1. Coefficients categorized in group serviceability, not mentioned in Eurocode, deal much more with probability of hazard on human life in the event of failure.

Because of complicated character of the structure's response to the dynamic loading, gust and turbulence, two codes define factors based on the background and resonant response. Eurocode proposes structural factor which takes into account simultaneous occurrence of peak wind pressure on the surface and vibrations due to turbulence. In ASCE, the dynamics of wind flow are expressed by the gust factor which is function of the turbulence intensity. Gust factor reduces the maximum expected wind effects with given probability. Atmospheric turbulence and dynamic sensitivity of the structure is then simplified to obtain equivalent static load.

Aerodynamic coefficients which describe the relative pressure throughout a flow of the wind varies slightly between codes. In case of Eurocode calculations leeward pressure coefficients are higher around 40% in N-S direction and around 100 % in W-E direction. Moreover, Eurocode defined two factors on side part of the building compare to ASCE which use uniform value within sides length. Wind loads are determined by multiplying the wind pressure by the tributary area of the building.

Exposure factor takes into account gust factor, orography and roughness and the parameters like turbulence intensity, terrain factor need to be specified. Basic wind pressure needed to calculate wind pressure on surface include such parameters as air density, seasonal factors. As a result, the total wind force in N-S direction obtained with help of Eurocode 1 (24903,12 kN) is smaller around 1,9 % than result obtained by ASCE 7-10 (25382,78 kN). Although in W-E direction is a higher. Difference is

6,9 %. It can be said that those discrepancies are relatively small. Different coefficients taken to the analysis impact directly or indirectly other factors.

Dynamic behaviour should be investigated by using wind tunnel method. The aerodynamic data used in above ways are simplified and can result in severely distorted representations of actual wind effects. For the sake of safety, when more reliable and detailed data is needed this method should be used.

5.4 Seismic load

Code provisions regarding the the specification of the hazard , site classification , design response spectrum, ductility classification ,response reduction factors and minimum design base shear force are compared and their cumulative effect on the design practice is shown below. The presented points highlights the major areas of differences . The Eurocode 8 recommends the consideration of a recurrence period for the non- collapse state as 475 years. This corresponds to a probability of 10 % of the seismic data which is exceeded in 50 years. In ASCE , that period is stated as 2475 years , which correspond to a probability of 2% of the seismic date exceeded in 50 years. Design seismic hazard is expressed by spectral ordinates. Eurocode specify hazard in the term of peak ground acceleration, which has to be defined by each of the National Authorities in Attachements. However, ASCE 7 express it in the term of the spectral accelerations and coefficients S_s S_1 at 0,2 and 1 s periods. These defines the displacement governed region of the spectrum.

In both codes, the elastic response spectra and acceleration are given as a function of the structural periods. The spectra unsurprisingly vary proportionally to the peak ground acceleration and soil coefficients. It is seen that that the nominal structural damping in both cases is assumed as 5 %. Because a_{vg} was greater than 0,25 g vertical component of the seismic action was taken into account in calculation according to Eurocode 8. The vertical response spectrum is defined as a separate case, thats why the frequency content of the vertical response is different than the horizontal one.

ASCE dont require to use vertical response spectrum in calculations.

Table 5.3: Seismic load comparison.

Category	EUROCODE			ASCE			
	Type		Value	Type		Value	
Design seismic hazard description	agr	Peak ground acceleration	0,25	Ss	0,2 s spectral acceleration	1,65	
	ag	Design grond acceleration	0,3	S1	1 s spectral acceleration	0,75	
	Response spectrum type		I				
	Direction		Horizontal ; Vertical	Direction		Horizontal	
Mass	Seismic weight		559097,74 kN	Seismic weight		540973,71 kN	
	Mass source	Dead load	1	Mass source	Dead load	1	
		Live load	0,15		Live load	0	
Modal participating mass ratio	T1x	Period of the mode	2,23 s	T1x	Period of the mode	2,47 s	
	UX	Ratio in the global X direction	0,6584	UX	Ratio in the global X direction	0,6593	
	T2y	Period of the mode	4,01 s	T2y	Period of the mode	4,47 s	
	UY	Ratio in the global X direction	0,647	UY	Ratio in the global X direction	0,646	
Site and building category	Ground Type		A (Rock)	Site class		A (Hard rock)	
	Importance class		III (Importance in view of the consequences associated with collapse)	I	Importance factor	1	
				Category of building		II (Low hazard of human life in the event of failure)	
Response spectrum parameters	Horizontal	S	Soil factor	Horizontal	Fa	Site coefficient	0,8
		Tb	Spectrum period		Fv	Site coefficient	0,8
		Tc	Spectrum period		Sd1	Spectrum curve	0,4
		Td	Spectrum period		Sds	Spectrum curve	0,88
	Vertical				Tl	Long-period transition period	12 s
		Tb	Spectrum period				
		Tc	Spectrum period				
		Td	Spectrum period				
		Avg/Ag	Acc. Ratio				
	Limits	β	Lower bound factor	0,2	Vmin	Minimum design base shear	0,044 Sd
Reduction factors	q	Behaviour factor	5,85	R	Seismic load reduction factor	6	
				Cd	Deflection amplification factor	5	
Base shear force	Vx	X direction	25177 kN	Vx	X direction	27208 kN	
	Vy	Y direction	22719 kN	Vy	Y direction	27295 kN	

Mass source definition played a key role in obtaining seismic weight of the structure. Due to higher coefficients in case of live load, seismic weight calculated with the help of Eurocode has 1% higher value. Periods of the modes and participating mass ratios in global directions X and Y has similar values.

Comparison of soil classification can be shown in table below. Both standards classify the ground conditions due to the shear wave propagation velocities.

Table 5.4: Shear wave velocities.

ASCE 7	Shear wave velocity	Eurocode 8	Shear wave velocity
Type A (Hard rock)	>1524	Type A	>800
TypeB (Rock)	762-1524		

Design spectrum depends on the level of ground motion expected at site.

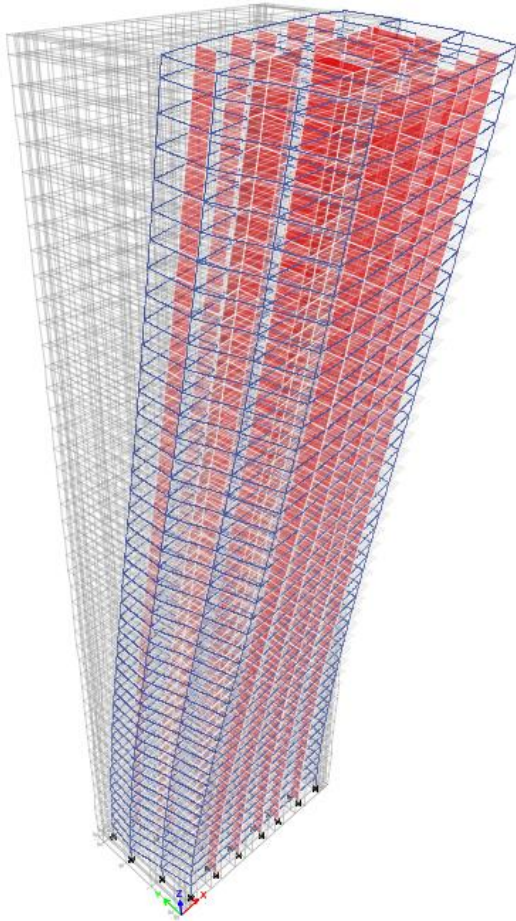
ASCE 7 specifies amplification factors for various soil type but Eurocode specify two different spectra, based on expected surface-magnitude at site. The seismic amplification at ground layers changes according to the shape of the response spectra. The less stiff layer, the higher ground acceleration and in result the higher value of the soil coefficient. Few differences can be observed while comparing base shear force requirements. The need of expressing differences in importance of the structures is underlined by importance factors which consider reliability aspects due to the estimated risk and failure. Irregularity in plan or in the elevation require more advanced method of analysis and its criterias are more strict. For regular and simple structure, the equivalent static method is permitted. The number of modes in modal analysis in Eurocode and ASCE is required at least as the number which assure 90 % of the total mass of the structure in each orthogonal horizontal directions. In Eurocode 8 the response spectrum used in modal analysis is obtained by factoring the elastic response spectrum. For different period the scale factor changes. Minimum spectra ordinate of 0,2 peak ground acceleration is defined as the minimum design base shear. The period at which this control the base shear increases with the behaviour factor q . Under seismic actions, structures are expected to behave in a non-linear range, showing large deformation and dissipating large amount of the energy.

Internal forces due to the seismic actions, designed in the force based method have to take into account that the system dissipate seismic energy. That's why the seismic forces has to be reduce by the factor as a function of the structural materials.

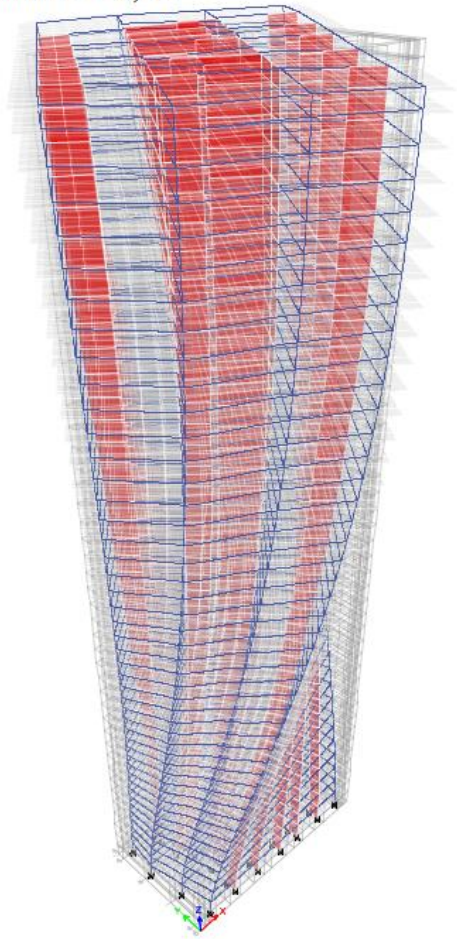
In Eurocode it is called behaviour factor. This factor is related to the ductility demand of the structure. The reduced load is critical for describing the demand for the structural type and its structural members. That demand need to be covered by the total capacity of the structure. The upper limit value of behaviour factor in each of the design direction needs to take into account the energy dissipation, number of regions where energy can be dissipated, failure modes, regularity or irregularity in plane and elevation aspect. However, in ASCE 7-10 for modal analysis the elastic response spectrum is used. To obtain design spectrum and to take into account ductile behaviour elastic spectrum is reduced by strength reduction factor R . The minimum design base shear is controlled by relation between design base shear with the equivalent static value:

$V_{min} = 0,044 S_{DS}$. Base shear force results show that because of much more rigorous restriction about minimum value of design response spectrum values obtained by Eurocode 8 are greater. The opposite values are seen when ASCE results are scaled according to equivalent lateral force procedure. To fulfil the requirements of the code, results obtained by response spectra method were compared with results obtained by Equivalent Lateral Static load method. According to clause 12.9.4.1 ASCE 7-10: if the modal base shear is less than 85 % of the calculated base shear using second method, the forces should be scaled. Based on the result after response spectrum and equivalent lateral force procedure, scale factor in load case definition of response spectrum method was changed in both direction X and Y, respectively to 1,68 and 3,13. EC8 is unique amongst seismic design codes. It is a template for a code rather than a complete set of definitions of earthquake actions for engineering design. Each member state of the European Union has to produce its own National Application Document, including a seismic hazard map showing PGA values for the 475-year return period, select either the Type 1 or Type 2 spectrum and, if considered appropriate, adapt details of the specification of site classes and spectral parameters. Interestingly, although the stated purpose of EC8 is harmonisation of seismic design across Europe, there could well be jumps in the level of seismic design loads across national borders as currently there is no official project for a community-wide hazard zonation map. Although there are a number of innovative features in EC8 with regards to the ASCE 7-10, such as the separate definition of the vertical response spectrum, the basic mechanism for defining the horizontal elastic design spectrum is outdated and significantly behind innovations in recent codes from other parts of the world.

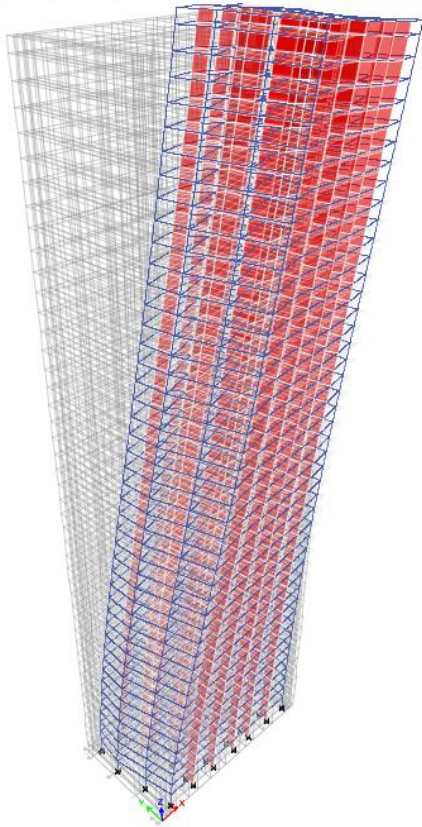
Mode 1: $T=3,68s$



Mode2 : $T=2,42s$



Mode 3: $T=2,08 s$



Mode 4: $T=0,94s$

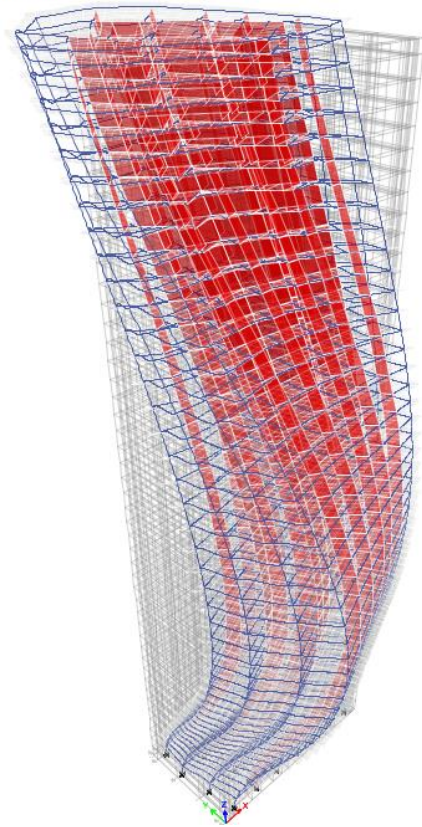
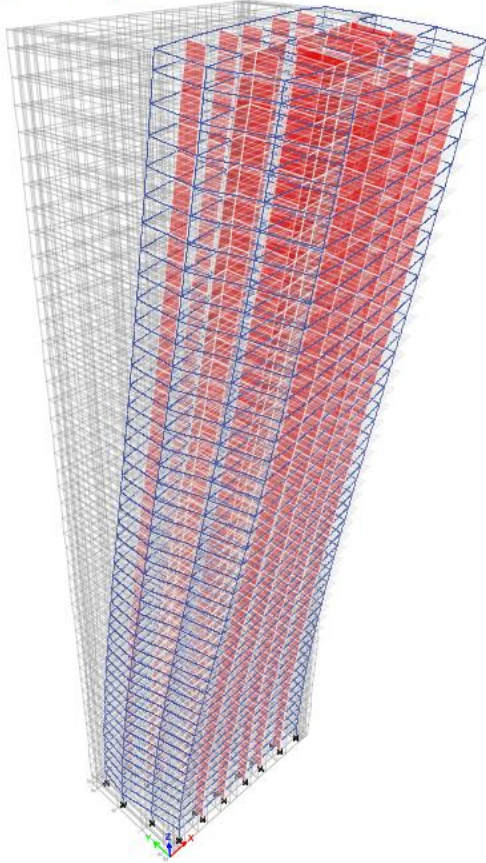
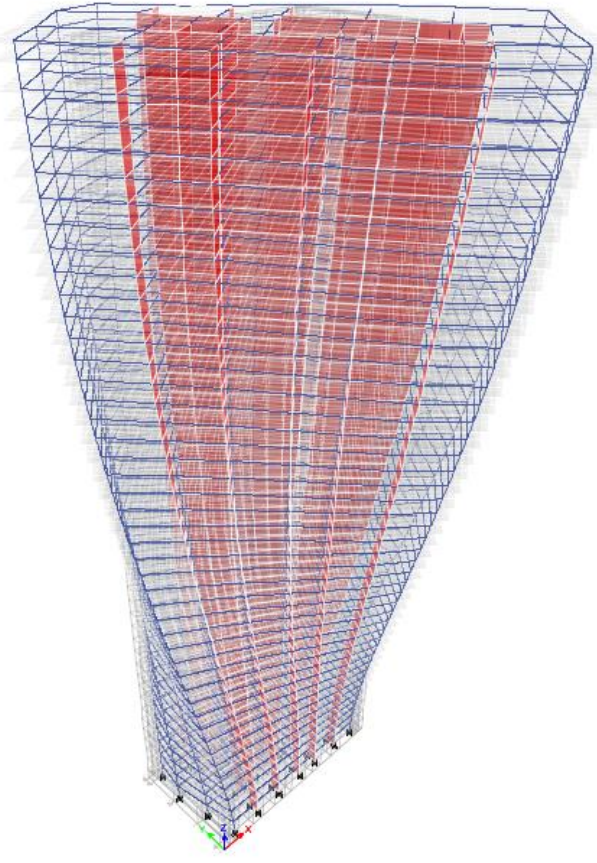


Figure 5.1: Mode shapes – Eurocode analysis.

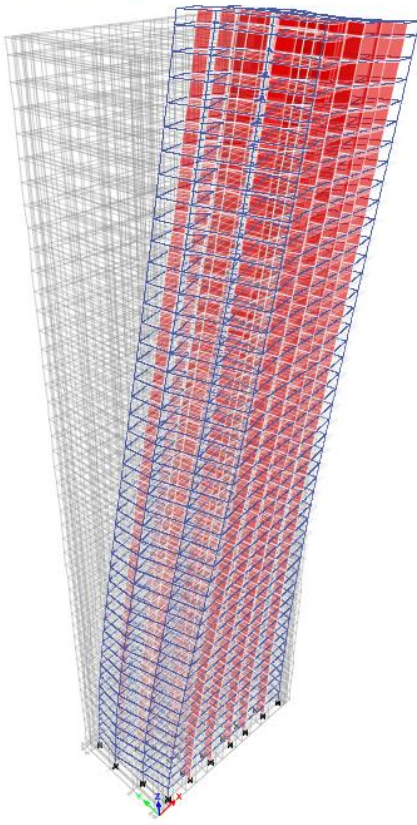
Mode 1: $T=3,64s$



Mode 2 : $T=2,4 s$



Mode 3: $T=2,05s$



Mode 4 : $T=0,93 s$

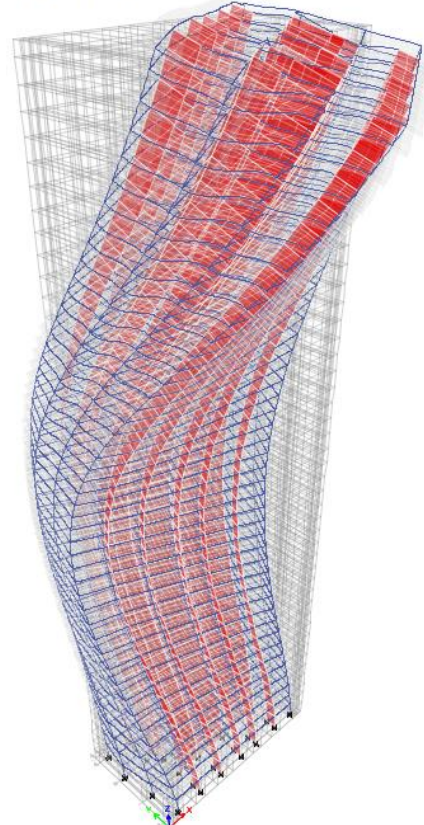


Figure 5.2: Mode shapes –ASCE 7-10 analysis.

5.5 Snow load

Table 5.5: Snow load comparison.

		Eurocode		ASCE	
Load paramet ers	μ	Roof shape coefficient	0,8	I	Importance factor
	Ce	Exposure coefficient	1	Ce	Exposure coefficient
	Ct	Thermal coefficient	1	Ct	Thermal coefficient
	Sk	Climatic coefficient	0,75 kN/m ²	pg	Ground snow load
			Reduction factor		0,7
Results	Snow load		0,6 kN/m ²	Snow load	0,378 kN/m ²

The snow load on the roof was derived from the snow load on the ground multiplied by conversion factors which took into account thermal, exposure and shape coefficients. Roof characteristics such as roughness, surrounding terrain were also included in calculations. Shape coefficients are needed for an adjustment of the ground snow load to a snow load on the roof taking into account effects caused by non-drifted and drifted snow load arrangements. The roof shape coefficient depends on the roof angle. ASCE cares also about importance factor. Moreover, reduction factor which transfer value of ground snow load into top level load is applied in this code.

As a result of reduction factor used in ASCE snow load value obtained by this code is smaller around 27 % than snow load taken from Eurocode calculations.

5.6 Beams

5.6.1 Eurocode 2

- Critical combination : 19 (D + 0,3 L + 0,3 Ex – Ey - 0,3 Ez)
- Flexure design:

Section 1:

$$M_{Ed,top} = -825,6 \text{ kNm}$$

$$M_{Ed,bottom} = 756,31 \text{ kNm}$$

Section 2:

$$M_{Ed,bottom} = 206,4 \text{ kNm}$$

Section 3:

$$M_{Ed,top} = -747,3 \text{ kNm}$$

$$M_{Ed,bottom} = 659,4 \text{ kNm}$$

- Shear design:

$$V_{Rds} = 447,04 \text{ kN}$$

Table 5.6: Total weight of reinforcement in beam B46 according to EC 2.

Type	Name	Amount	Length [cm]	Mass per unit volume [kg/cm³]	Cross section area [cm²]	Rebar mass [kg]	Total weight [kg]
Steel A615Gr60							
1	φ26	3	542,00	0,007849	5,31	22,58	67,73
1 a	φ26	4	195,00	0,007849	5,31	8,12	32,49
1 b	φ26	2	195,00	0,007849	5,31	8,12	16,24
1 c	φ12	2	542,00	0,007849	1,13	4,81	9,62
1 d	φ22	4	180,00	0,007849	3,80	5,37	21,47
2 a	φ12	52	220,00	0,007849	1,13	1,95	101,50
2 b	φ10	8	220,00	0,007849	0,79	1,36	10,84
3 a	φ10	13	172,00	0,007849	0,79	1,06	13,78
3 b	φ12	13	84,00	0,007849	0,79	0,52	6,73
						Σ=	280,40

5.6.1 ACI 318-11

Critical combination : $15 (1,376 D + L + 0,39 E_x - 1,3E_y + 0,5 S)$
 $(1,2 D + [1,3 (0,3E_x - E_y) + 0,2S_{DS} \cdot D] + 1,0 L + 0,5 S)$

- **Flexure design:**

Section 1:

$$M_{Ed,top} = -1218,22 \text{ kNm}$$

$$M_{Ed,bottom} = 1112,84 \text{ kNm}$$

Section 2:

$$M_{Ed,bottom} = 304,55 \text{ kNm}$$

Section 3:

$$M_{Ed,top} = -1105,45 \text{ kNm}$$

$$M_{Ed,bottom} = 968,22 \text{ kNm}$$

- **Shear design:**

$$V_{Rds} = 661,97 \text{ kN}$$

Table 5.7: Total weight of reinforcement in beam B46 according to ACI 318-11.

Type	Name	Amount	Length [cm]	Mass per unit volume [kg/cm ³]	Cross section area [cm ²]	Rebar mass [kg]	Total weight [kg]
Steel A615Gr60							
1 a	φ26	5	544,00	0,007849	5,31	22,66	113,29
1 b	φ26	4	544,00	0,007849	5,31	22,66	90,63
1 c	φ26	4	180,00	0,007849	5,31	7,50	29,99
1 d	φ22	6	180,00	0,007849	3,80	5,37	32,21
1 e	φ26	3	198,00	0,007849	5,31	8,25	24,74
1 f	φ26	4	198,00	0,007849	5,31	8,25	32,99
1 g	φ12	2	544,00	0,007849	1,13	4,83	9,65
2 a	φ12	38	220,00	0,007849	1,13	1,95	74,17
2 b	φ12	4	220,00	0,007849	1,13	1,95	7,81
3 a	φ12	13	172,00	0,007849	1,13	1,53	19,84
3 b	φ12	13	190,00	0,007849	1,13	1,69	21,92
						Σ=	457,24

The total weight of reinforcement used in a beam is greater in accordance with calculations based on ACI 318-11, around 176,84 kg . Impact of the dead load increased by coefficient in load combination to 137 %, live load more than 70 % , earthquake force in X and Y direction, 9% and 30% respectively and additional influence of snow are had directly influence on heigher amount of reinforcement used.

$$\frac{\text{Total weight acc. to ASCE}}{\text{Total weight acc. to EC 2}} = \frac{472,13}{298,65 \text{ kg}} = 1,58$$

Section 1:

$$\frac{M_{\text{Ed,topACI}}}{M_{\text{Ed,top,EC}}} = \frac{-1218,22 \text{ kNm}}{-825,6 \text{ kNm}} = 1,48$$

$$\frac{M_{\text{Ed,bottom,ACI}}}{M_{\text{Ed,bottom,EC}}} = \frac{1112,84 \text{ kNm}}{756,31 \text{ kNm}} = 1,47$$

Section 2:

$$\frac{M_{\text{Ed,bottom,ACI}}}{M_{\text{Ed,bottom,EC}}} = \frac{304,55 \text{ kNm}}{206,4 \text{ kNm}} = 1,48$$

Section 3:

$$\frac{M_{\text{Ed,topACI}}}{M_{\text{Ed,top,EC}}} = \frac{-1105,45 \text{ kNm}}{-747,3 \text{ kNm}} = 1,48$$

$$\frac{M_{\text{Ed,bottom,ACI}}}{M_{\text{Ed,bottom,EC}}} = \frac{968,22 \text{ kNm}}{659,4 \text{ kNm}} = 1,47$$

$$\frac{V_{\text{Ed,ACI}}}{V_{\text{Ed2,EC}}} = \frac{661,97 \text{ kN}}{447,04 \text{ kN}} = 1,48$$

5.7 Columns

5.7.1 Eurocode 2

Critical combination for flexure : 5 (1,35D+1,5W+1,05L+0,75S)

Critical combination for shear : 15 (D +0,3 L+ Ex – 0,3Ey-0,3 Ez)

- Flexure design:

$$N_{Ed,EC} = 28581,2 \text{ kN}$$

$$M_{Ed2,EC} = -952,71 \text{ kNm}$$

$$M_{Ed3,EC} = -1345,2 \text{ kNm}$$

- Shear design:

$$V_{Ed2,EC} = 243,24 \text{ kN}$$

$$V_{Ed3,EC} = 139,77 \text{ kN}$$

Table 5.8: Total weight of reinforcement in column C2 according to EC 2.

Type	Name	Amount	Length [cm]	Mass per unit volume [kg/cm ³]	Cross section area [cm ²]	Rebar mass [kg]	Total weight [kg]
Steel A615Gr60							
1	φ25	28	425,00	0,007849	4,91	16,37	458,26
2	φ8	20	410,00	0,007849	0,50	1,62	32,34
3	φ8	20	308,00	0,007849	0,50	1,21	24,29
4	φ8	20	107,00	0,007849	0,50	0,42	8,44
5	φ8	20	300,00	0,007849	0,50	1,18	23,66
						Σ=	546,98

5.7.2 ACI 318-11

Critical combination for flexure : 15 (1,376 D +0,39 Ex – 1,3Ey + L + 0,5S)

Critical combination for shear : 24 (0,724D +0,39 Ex – 1,3Ey)

- Flexure design:

$$N_{Ed,ACI} = 29922,11 \text{ kN}$$

$$M_{Ed2,ACI} = 679,7 \text{ kNm}$$

$$M_{Ed3,ACI} = -2947,98 \text{ kNm}$$

- Shear design:

$$V_{Ed2,ACI} = 435,9 \text{ kN}$$

$$V_{Ed3,ACI} = 164,5 \text{ kN}$$

Table 5.9: Total weight of reinforcement in column C2 according to ACI 318-11.

Type	Name	Amount	Length [cm]	Mass per unit volume [kg/cm ³]	Cross section area [cm ²]	Rebar mass [kg]	Total weight [kg]
Steel A615Gr60							
1	φ40	38	425,00	0,007849	12,56	41,90	1592,12
2	φ10	26	408,00	0,007849	0,79	2,51	65,36
3	φ10	26	284,00	0,007849	0,79	1,75	45,50
4	φ10	26	296,00	0,007849	0,79	1,82	47,42
5	φ10	26	106,00	0,007849	0,79	0,65	16,98
						Σ=	1767,38

Total weight of flexural and shear reinforcement is greater while calculation according to ACI 318-11. Result obtained with the help of Eurocode 2 is just 38% of the amount required by american regulation. Critical combination for flexure in second part, is greatly higher than forces used for design in Eurocode. Almost in every aspect design forces are greater while calculations based on ACI 318-11.

$$\frac{\text{Total weight acc. to ASCE}}{\text{Total weight acc. to EC 2}} = \frac{1738,26 \text{ kg}}{546,98 \text{ kg}} = 3,18$$

$$\frac{N_{Ed,ACI}}{N_{Ed,EC}} = \frac{29922,11 \text{ kN}}{28581,2 \text{ kN}} = 1,04$$

$$\frac{M_{Ed2,ACI}}{M_{Ed2,EC}} = \frac{679,7 \text{ kNm}}{952,71 \text{ kNm}} = 0,71$$

$$\frac{M_{Ed3,ACI}}{M_{Ed3,EC}} = \frac{2947,98 \text{ kNm}}{1345,2 \text{ kNm}} = 2,19$$

$$\frac{V_{Ed2,ACI}}{V_{Ed2,EC}} = \frac{435,9 \text{ kN}}{243,24 \text{ kN}} = 1,79$$

$$\frac{V_{Ed3,ACI}}{V_{Ed3,EC}} = \frac{164,5 \text{ kN}}{139,77 \text{ kN}} = 1,18$$

5.8 Shear wall

5.8.1 Eurocode 2

Critical combination for flexure : DWALL18 (D+ EqY)

Critical combination for shear : DWALL5 (1,35D +1,05L+1,5 L) – Leg 1

DWALL17 (D + EqX) – Leg 2

Table 5.10: Shear wall, design for flexure- EC.

Station	Required	Required	Current	Flexural	N _{Ed}	M _{Ed2}	M _{Ed3}
Location	Rebar Area (mm ²)	Reinf Ratio	Reinf Ratio	Combo	kN	kN-m	kN-m
Top	47037	0,0075	0,0086	DWal18	13792,199	-13558,679	26192,705
Bottom	52625	0,0084	0,0086	DWal18	14298,643	-14428,846	25938,653

Table 5.11: Shear wall, design for shear –EC.

Station	ID	Rebar	Shear	N _{Ed}	V _{Ed}	V _{Rc}	V _{Rd}
Location		mm ² /m	Combo	kN	kN	kN	kN
Top	Leg 1	1285,16	DWal5	1836,4333	2958,768	887,3286	2958,768
Top	Leg 2	3600,56	DWal17	4649,9062	9605,3307	2310,2722	9605,3307
Bottom	Leg 1	1285,16	DWal5	2011,8815	2958,768	936,9745	2958,768
Bottom	Leg 2	3600,56	DWal17	5026,3888	9605,3307	2428,2356	9605,3307

Table 5.12: Total weight of reinforcement in Core 4 according to EC 2.

Type	Name	Amount	Length [cm]	Mass per unit volume [kg/cm ³]	Cross section area [cm ²]	Rebar mass [kg]	Total weight [kg]
Steel A615Gr60							
1	φ20	178	425	0,007849	3,14	10,47	1864,46
2 a	φ10	16	425	0,007849	0,79	2,62	41,90
2 b	φ10	16	425	0,007849	0,79	2,62	41,90
3 a	φ10	16	374	0,007849	0,79	2,30	36,87
3 b	φ10	16	374	0,007849	0,79	2,30	36,87
4	φ10	32	446	0,007849	0,79	2,75	87,94
5 a	φ10	112	54	0,007849	0,79	0,33	37,26
5 b	φ10	693	54	0,007849	0,79	0,33	230,57
6 a	φ12	33	1032	0,007849	1,13	9,16	302,16
6 b	φ12	33	1032	0,007849	1,13	9,16	302,16
7 a	φ12	33	981	0,007849	1,13	8,70	287,23
7 b	φ12	33	981	0,007849	1,13	8,70	287,23
8	φ12	198	506	0,007849	1,13	4,49	888,92
						Σ=	4445,47

5.8.2 ACI 318-11

Critical combination for flexure : DWALL41 (0,8D+ EqY)

Critical combination for shear : DWALL30 (0,9D +WY) – Leg 1

DWALL36 (1,3D + L+0,2 S+EqX) – Leg 2

Table 5.13: Shear wall, design for flexure –ACI.

Station Location	Required Rebar Area (mm ²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P _u kN	M _{u2} kN-m	M _{u3} kN-m
Top	63073	0,0096	0,0157	DWAl41	8281,7375	-14691,167	37096,374
Bottom	85566	0,013	0,0157	DWAl41	8686,8928	-18604,716	41083,602

Table 5.14: Shear wall, design for shear-ACI.

Station Location	ID	Rebar mm ² /m	Shear Combo	P _u kN	M _u kN-m	V _u kN	ΦV _c kN
Top	Leg 1	2318,38	DWAl30	-6596,6519	-14304,758	2048,0042	206,5695
Top	Leg 2	1250	DWAl36	36015,009	28914,559	5606,1575	4381,3884
Bottom	Leg 1	2266,38	DWAl30	-6479,6864	-20960,771	2048,0042	247,8746

Table 5.15: Total weight of reinforcement in Core 4 according to ACI.

Type	Name	Amount	Length [cm]	Mass per unit volume [kg/cm ³]	Cross section area [cm ²]	Rebar mass [kg]	Total weight [kg]
Steel A615Gr60							
1	φ26	50	425	0,007849	5,31	17,70	885,09
2 a	26	48	325	0,007849	5,31	13,54	649,76
2 b	26	40	325	0,007849	5,31	13,54	541,47
3	20	16	325	0,007849	3,14	8,01	128,16
4	20	42	325	0,007849	3,14	8,01	336,42
5	10	52	300	0,007849	0,79	1,85	96,12
6 a	10	16	644	0,007849	0,79	3,97	63,49
6 b	10	16	676	0,007849	0,79	4,17	66,64
7	10	624	55	0,007849	0,79	0,34	211,46
8 a	10	26	425	0,007849	0,79	2,62	68,08
8 b	10	26	425	0,007849	0,79	2,62	68,08
9 a	10	26	374	0,007849	0,79	2,30	59,91
9 b	10	26	374	0,007849	0,79	2,30	59,91
10 a	10	16	1032	0,007849	0,79	6,36	101,74
10 b	10	16	1032	0,007849	0,79	6,36	101,74
10 c	10	16	981	0,007849	0,79	6,04	96,71
10 d	10	16	981	0,007849	0,79	6,04	96,71
Σ=							2978,61

Combination for flexure in Eurocode calculations is greater about 20 % in the sense of dead load . Based on ACI 318-11, transverse reinforcement in Leg 2 , is design based on additional part of live load and snow. However, only wind impact and dead load control design for shear in Leg 1. However, in Eurocode impact of live load has to be taken into account while calculating shear reinforcement in Leg 1.

Total weight of flexure and shear rebars according to american standart is just 67 % of weight calculated accordance to Eurocode.

$$\frac{\text{Total weight acc. to ASCE}}{\text{Total weight acc. to EC 2}} = \frac{2978,61 \text{ kg}}{4445,47 \text{ kg}} = 0,67$$

Flexure design:

- EC

Cross sectional area of longitudinal bars:

$$A_{f,Leg1} = 54 \cdot \frac{\pi \cdot 2^2}{4} = 169,65 \text{ cm}^2$$

$$A_{f,Leg2} = 134 \cdot \frac{\pi \cdot 2^2}{4} = 420,97 \text{ cm}^2$$

- ACI

$$A_{f,Leg1} = 50 \cdot \frac{\pi \cdot 2,6^2}{4} + 16 \cdot \frac{\pi \cdot 2^2}{4} = 315,73 \text{ cm}^2$$

$$A_{f,Leg2} = 88 \cdot \frac{\pi \cdot 2,6^2}{4} + 42 \cdot \frac{\pi \cdot 2^2}{4} = 599,16 \text{ cm}^2$$

Shear design:

Cross sectional area of transverse bars:

- EC

X direction:

$$A_{f,Leg1} = 10 \cdot 16 \cdot \frac{\pi \cdot 1^2}{4} = 125,66 \text{ cm}^2$$

$$A_{s,Leg2} = 33 \cdot 4 \cdot \frac{\pi \cdot 1,2^2}{4} = 149,29 \text{ cm}^2$$

$$\sum = 274,95 \text{ cm}^2$$

Y direction:

$$A_{s,Leg1} = 16 \cdot 4 \cdot \frac{\pi \cdot 1^2}{4} = 50,26 \text{ cm}^2$$
$$A_{s,Leg2} = 31 \cdot 33 \cdot \frac{\pi \cdot 1,2^2}{4} = 1156,9 \text{ cm}^2$$
$$\sum = 1207,16 \text{ cm}^2$$

- ACI

X direction:

$$A_{f,Leg1} = 11 \cdot 26 \cdot \frac{\pi \cdot 1^2}{4} = 224,62 \text{ cm}^2$$
$$A_{s,Leg2} = 16 \cdot 4 \cdot \frac{\pi \cdot 1^2}{4} = 50,26 \text{ cm}^2$$
$$\sum = 274,88 \text{ cm}^2$$

Y direction:

$$A_{s,Leg1} = 26 \cdot 4 \cdot \frac{\pi \cdot 1^2}{4} = 50,26 \text{ cm}^2$$
$$A_{s,Leg2} = 30 \cdot 16 \cdot \frac{\pi \cdot 1^2}{4} = 377 \text{ cm}^2$$
$$\sum = 427,26 \text{ cm}^2$$

One of the main reason can be different deninition of boundary elements and assumptions about how much of the load carry concrete itself. Core 4 when calculated with European standards is treated along length as the whole, overlapping to each other boundary zone. Although ACI defines certain length of boundary zones in each leg. These results came from static calculation, raports prepared by ETABS.

In case of shear reinforcement, discrepancies in applied rebars are due to the portion of the shear force carried by the concrete.

Eurocode:

LEG 1:

$$V_{Rd,c} = 936,97 \text{ kN}$$

$$V_{Ed} = 2958,77 \text{ kN}$$

$$\frac{V_{Rd,c}}{V_{Ed}} = \frac{936,97 \text{ kN}}{2958,77 \text{ kN}} = 0,32$$

$$\text{Rebar } \text{mm}^2/\text{m}: 1285,16$$

LEG 2:

$$V_{Rd,c} = 2310,27 \text{ kN}$$

$$V_{Ed} = 9605 \text{ kN}$$

$$\frac{V_{Rd,c}}{V_{Ed}} = \frac{2310,27 \text{ kN}}{9605 \text{ kN}} = 0,24$$

$$\text{Rebar } \text{mm}^2/\text{m}: 3600,56$$

ACI 318-11:

LEG 1:

$$V_{Rd,c} = 244,96 \text{ kN}$$

$$V_{Ed} = 2170,39 \text{ kN}$$

$$\frac{V_{Rd,c}}{V_{Ed}} = \frac{244,96 \text{ kN}}{5947,17 \text{ kN}} = 0,04$$

$$\text{Rebar } \text{mm}^2/\text{m}: 2318,38$$

LEG 2:

$$V_{Rd,c} = 4381,39 \text{ kN}$$

$$V_{Ed} = 5947,17 \text{ kN}$$

$$\frac{V_{Rd,c}}{V_{Ed}} = \frac{4381,39 \text{ kN}}{5947,17 \text{ kN}} = 0,74$$

$$\text{Rebar } \text{mm}^2/\text{m}: 1250$$

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APPENDICES

APPENDIX A: Story shear forces due to earthquake loading in X direction- EN 1998-1:2004

APPENDIX B: Story shear forces due to earthquake loading in Y direction- EN 1998-1:2004

APPENDIX C: Maximum story displacements due to earthquake loading in X direction- EN 1998-1:2004

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APPENDIX I : Beam design results from ETABS - EN 1998-1:2004

APPENDIX J: Column design results from ETABS- EN 1998-1:2004

APPENDIX K: Shear wall design results from ETABS- EN 1998-1:2004

APPENDIX L : Beam design results from ETABS – ACI 318-11

APPENDIX M: Column design results from ETABS- ACI 318-11

APPENDIX N: Shear wall design results from ETABS- ACI 318-11

LIST OF DESIGN DRAWINGS

Drawing 1 : Beam B46 , floor 10: Reinforcement detailing according to EC 2

Drawing 2 : Beam B46 , floor 10: Reinforcement detailing according to ACI 318-11

Drawing 3 : Column C2 , floor 1: Reinforcement detailing according to EC 2

Drawing 4 : Column C2 , floor 1: Reinforcement detailing according to ACI 318-11

Drawing 5 : Shear wall-core 4 , floor 1:Reinforcement detailing according to EC 2
(Arrangement 2)

Drawing 6 : Shear wall-core 4 , floor 1:Reinforcement detailing according to
ACI 318-11 (Arrangement 1)

**APPENDIX A: Story shear forces due to earthquake loading in X direction
- EN 1998-1:2004**

Story	Elevation	Location	X-dir [kN]	Y-dir [kN]	Story	Elevation	Location	X-dir [kN]	Y-dir [kN]
Story45	145	Top	1261,83	168,05	Story22	71,5	Top	17957,12	2397,68
		Bottom	1261,83	168,05			Bottom	17957,12	2397,68
Story44	143	Top	2786,40	374,05	Story21	68,25	Top	18376,87	2445,66
		Bottom	2786,40	374,05			Bottom	18376,87	2445,66
Story43	139,75	Top	4237,66	575,51	Story20	65	Top	18790,30	2489,41
		Bottom	4237,66	575,51			Bottom	18790,30	2489,41
Story42	136,5	Top	5512,52	752,49	Story19	61,75	Top	19198,73	2530,20
		Bottom	5512,52	752,49			Bottom	19198,73	2530,20
Story41	133,25	Top	6646,87	904,54	Story18	58,5	Top	19602,28	2569,24
		Bottom	6646,87	904,54			Bottom	19602,28	2569,24
Story40	130	Top	7667,10	1033,16	Story17	55,25	Top	20000,57	2607,78
		Bottom	7667,10	1033,16			Bottom	20000,57	2607,78
Story39	126,75	Top	8592,75	1141,57	Story16	52	Top	20397,48	2647,47
		Bottom	8592,75	1141,57			Bottom	20397,48	2647,47
Story38	123,5	Top	9438,15	1234,03	Story15	48,75	Top	20793,66	2689,36
		Bottom	9438,15	1234,03			Bottom	20793,66	2689,36
Story37	120,25	Top	10212,99	1315,16	Story14	45,5	Top	21183,47	2733,66
		Bottom	10212,99	1315,16			Bottom	21183,47	2733,66
Story36	117	Top	10925,91	1389,29	Story13	42,25	Top	21565,87	2780,64
		Bottom	10925,91	1389,29			Bottom	21565,87	2780,64
Story35	113,75	Top	11585,39	1460,08	Story12	39	Top	21939,25	2829,89
		Bottom	11585,39	1460,08			Bottom	21939,25	2829,89
Story34	110,5	Top	12198,25	1530,41	Story11	35,75	Top	22299,14	2880,46
		Bottom	12198,25	1530,41			Bottom	22299,14	2880,46
Story33	107,25	Top	12771,18	1602,19	Story10	32,5	Top	22642,25	2931,22
		Bottom	12771,18	1602,19			Bottom	22642,25	2931,22
Story32	104	Top	13311,47	1676,37	Story9	29,25	Top	22966,72	2980,58
		Bottom	13311,47	1676,37			Bottom	22966,72	2980,58
Story31	100,75	Top	13830,39	1754,02	Story8	26	Top	23267,36	3026,78
		Bottom	13830,39	1754,02			Bottom	23267,36	3026,78
Story30	97,5	Top	14333,79	1834,70	Story7	22,75	Top	23539,67	3068,20
		Bottom	14333,79	1834,70			Bottom	23539,67	3068,20
Story29	94,25	Top	14820,80	1916,31	Story6	19,5	Top	23783,57	3103,22
		Bottom	14820,80	1916,31			Bottom	23783,57	3103,22
Story28	91	Top	15294,99	1997,35	Story5	16,25	Top	23995,34	3130,43
		Bottom	15294,99	1997,35			Bottom	23995,34	3130,43
Story27	87,75	Top	15758,18	2076,13	Story4	13	Top	24168,44	3148,76
		Bottom	15758,18	2076,13			Bottom	24168,44	3148,76
Story26	84,5	Top	16212,48	2151,14	Story3	9,75	Top	24304,10	3157,44
		Bottom	16212,48	2151,14			Bottom	24304,10	3157,44
Story25	81,25	Top	16659,36	2221,33	Story2	6,5	Top	24399,66	3155,91
		Bottom	16659,36	2221,33			Bottom	24399,66	3155,91
Story24	78	Top	17098,70	2285,92	Story1	3,25	Top	24425,48	3142,86
		Bottom	17098,70	2285,92			Bottom	24425,48	3142,86
Story23	74,75	Top	17530,97	2344,60	Base	0	Top	0,00	0,00
		Bottom	17530,97	2344,60			Bottom	0,00	0,00

**APPENDIX B: Story shear forces due to earthquake loading in Y direction
- EN 1998-1:2004**

Story	Elevation	Location	X-dir [kN]	Y-dir [kN]	Story	Elevation	Location	X-dir [kN]	Y-dir [kN]
Story45	145	Top	162,38	948,20	Story22	71,5	Top	2349,83	17837,67
		Bottom	162,38	948,20			Bottom	2349,83	17837,67
Story44	143	Top	361,51	2084,49	Story21	68,25	Top	2407,05	18268,49
		Bottom	361,51	2084,49			Bottom	2407,05	18268,49
Story43	139,75	Top	557,43	3188,83	Story20	65	Top	2465,02	18674,08
		Bottom	557,43	3188,83			Bottom	2465,02	18674,08
Story42	136,5	Top	733,43	4213,89	Story19	61,75	Top	2523,89	19054,66
		Bottom	733,43	4213,89			Bottom	2523,89	19054,66
Story41	133,25	Top	890,85	5184,05	Story18	58,5	Top	2583,25	19409,68
		Bottom	890,85	5184,05			Bottom	2583,25	19409,68
Story40	130	Top	1031,27	6104,34	Story17	55,25	Top	2642,81	19739,99
		Bottom	1031,27	6104,34			Bottom	2642,81	19739,99
Story39	126,75	Top	1156,52	6976,17	Story16	52	Top	2702,62	20047,99
		Bottom	1156,52	6976,17			Bottom	2702,62	20047,99
Story38	123,5	Top	1268,63	7803,04	Story15	48,75	Top	2762,03	20334,48
		Bottom	1268,63	7803,04			Bottom	2762,03	20334,48
Story37	120,25	Top	1369,57	8594,03	Story14	45,5	Top	2819,72	20598,47
		Bottom	1369,57	8594,03			Bottom	2819,72	20598,47
Story36	117	Top	1461,13	9357,86	Story13	42,25	Top	2875,10	20840,91
		Bottom	1461,13	9357,86			Bottom	2875,10	20840,91
Story35	113,75	Top	1544,98	10098,05	Story12	39	Top	2927,16	21061,96
		Bottom	1544,98	10098,05			Bottom	2927,16	21061,96
Story34	110,5	Top	1622,54	10816,38	Story11	35,75	Top	2975,10	21262,73
		Bottom	1622,54	10816,38			Bottom	2975,10	21262,73
Story33	107,25	Top	1694,85	11513,80	Story10	32,5	Top	3018,37	21442,13
		Bottom	1694,85	11513,80			Bottom	3018,37	21442,13
Story32	104	Top	1762,91	12189,88	Story9	29,25	Top	3056,27	21598,65
		Bottom	1762,91	12189,88			Bottom	3056,27	21598,65
Story31	100,75	Top	1828,15	12851,53	Story8	26	Top	3088,27	21732,77
		Bottom	1828,15	12851,53			Bottom	3088,27	21732,77
Story30	97,5	Top	1891,24	13499,11	Story7	22,75	Top	3114,16	21841,68
		Bottom	1891,24	13499,11			Bottom	3114,16	21841,68
Story29	94,25	Top	1952,02	14124,21	Story6	19,5	Top	3133,66	21923,34
		Bottom	1952,02	14124,21			Bottom	3133,66	21923,34
Story28	91	Top	2011,08	14726,86	Story5	16,25	Top	3146,66	21978,99
		Bottom	2011,08	14726,86			Bottom	3146,66	21978,99
Story27	87,75	Top	2068,67	15306,32	Story4	13	Top	3153,35	22003,08
		Bottom	2068,67	15306,32			Bottom	3153,35	22003,08
Story26	84,5	Top	2125,26	15861,76	Story3	9,75	Top	3154,00	21994,06
		Bottom	2125,26	15861,76			Bottom	3154,00	21994,06
Story25	81,25	Top	2181,38	16393,29	Story2	6,5	Top	3149,25	21951,58
		Bottom	2181,38	16393,29			Bottom	3149,25	21951,58
Story24	78	Top	2237,24	16899,85	Story1	3,25	Top	3139,68	21839,26
		Bottom	2237,24	16899,85			Bottom	3139,68	21839,26
Story23	74,75	Top	2293,24	17381,20	Base	0	Top	0,00	0,00
		Bottom	2293,24	17381,20			Bottom	0,00	0,00

**APPENDIX C: Maximum story displacements due to earthquake loading
in X direction- EN 1998-1:2004**

Story	Elevation	Location	X-dir [mm]	Y-dir [mm]
Story45	145,00	Top	111,20	51,20
Story44	143,00	Top	109,80	50,80
Story43	139,75	Top	107,40	50,00
Story42	136,50	Top	105,00	49,30
Story41	133,25	Top	102,50	48,50
Story40	130,00	Top	100,10	47,70
Story39	126,75	Top	97,50	46,90
Story38	123,50	Top	95,00	46,00
Story37	120,25	Top	92,40	45,10
Story36	117,00	Top	89,70	44,10
Story35	113,75	Top	87,00	43,20
Story34	110,50	Top	84,30	42,20
Story33	107,25	Top	81,50	41,10
Story32	104,00	Top	78,70	40,00
Story31	100,75	Top	75,90	38,90
Story30	97,50	Top	73,00	37,80
Story29	94,25	Top	70,10	36,60
Story28	91,00	Top	67,20	35,40
Story27	87,75	Top	64,30	34,10
Story26	84,50	Top	61,30	32,90
Story25	81,25	Top	58,40	31,60
Story24	78,00	Top	55,40	30,20
Story23	74,75	Top	52,50	28,90
Story22	71,50	Top	49,50	27,50
Story21	68,25	Top	46,50	26,10
Story20	65,00	Top	43,60	24,70
Story19	61,75	Top	40,70	23,30
Story18	58,50	Top	37,80	21,90
Story17	55,25	Top	34,90	20,40
Story16	52,00	Top	32,00	19,00
Story15	48,75	Top	29,20	17,50
Story14	45,50	Top	26,50	16,10
Story13	42,25	Top	23,80	14,60
Story12	39,00	Top	21,10	13,20
Story11	35,75	Top	18,60	11,80
Story10	32,50	Top	16,10	10,40
Story9	29,25	Top	13,70	9,00
Story8	26,00	Top	11,50	7,60
Story7	22,75	Top	9,30	6,30
Story6	19,50	Top	7,30	5,00
Story5	16,25	Top	5,50	3,80
Story4	13,00	Top	3,90	2,70
Story3	9,75	Top	2,50	1,70
Story2	6,50	Top	1,30	0,90
Story1	3,25	Top	0,40	0,30
Base	0,00	Top	0,00	0,00

**APPENDIX D: Maximum story displacements due to earthquake loading
in Y direction- EN 1998-1:2004**

Story	Elevation	Location	X-dir [mm]	Y-dir [mm]
Story45	145,00	Top	86,40	417,80
Story44	143,00	Top	85,70	413,10
Story43	139,75	Top	84,60	405,30
Story42	136,50	Top	83,40	397,40
Story41	133,25	Top	82,20	389,20
Story40	130,00	Top	81,00	380,90
Story39	126,75	Top	79,70	372,40
Story38	123,50	Top	78,30	363,70
Story37	120,25	Top	76,80	354,70
Story36	117,00	Top	75,30	345,50
Story35	113,75	Top	73,70	336,10
Story34	110,50	Top	72,00	326,50
Story33	107,25	Top	70,30	316,70
Story32	104,00	Top	68,50	306,70
Story31	100,75	Top	66,60	296,40
Story30	97,50	Top	64,70	286,00
Story29	94,25	Top	62,60	275,40
Story28	91,00	Top	60,60	264,70
Story27	87,75	Top	58,40	253,80
Story26	84,50	Top	56,20	242,70
Story25	81,25	Top	54,00	231,60
Story24	78,00	Top	51,70	220,30
Story23	74,75	Top	49,30	208,90
Story22	71,50	Top	46,90	197,50
Story21	68,25	Top	44,40	186,10
Story20	65,00	Top	42,00	174,60
Story19	61,75	Top	39,40	163,10
Story18	58,50	Top	36,90	151,70
Story17	55,25	Top	34,30	140,30
Story16	52,00	Top	31,80	128,90
Story15	48,75	Top	29,20	117,80
Story14	45,50	Top	26,60	106,70
Story13	42,25	Top	24,10	95,80
Story12	39,00	Top	21,50	85,20
Story11	35,75	Top	19,00	74,80
Story10	32,50	Top	16,60	64,80
Story9	29,25	Top	14,20	55,10
Story8	26,00	Top	11,90	45,80
Story7	22,75	Top	9,70	37,10
Story6	19,50	Top	7,70	28,90
Story5	16,25	Top	5,70	21,40
Story4	13,00	Top	4,00	14,80
Story3	9,75	Top	2,50	9,10
Story2	6,50	Top	1,30	4,50
Story1	3,25	Top	0,40	1,30
Base	0,00	Top	0,00	0,00

**APPENDIX E: Story shear forces due to earthquake loading
in X direction- ACI 318-11**

Story	Elevation	Location	X-dir [kN]	Y-dir [kN]	Story	Elevation	Location	X-dir [kN]	Y-dir [kN]
Story45	145,00	Top	1005,11	138,75	Story22	71,50	Top	9485,66	1122,47
		Bottom	1005,11	138,75			Bottom	9485,66	1122,47
Story44	143,00	Top	2234,05	307,81	Story21	68,25	Top	9710,50	1129,91
		Bottom	2234,05	307,81			Bottom	9710,50	1129,91
Story43	139,75	Top	3402,56	468,64	Story20	65,00	Top	9970,56	1134,04
		Bottom	3402,56	468,64			Bottom	9970,56	1134,04
Story42	136,50	Top	4403,37	603,32	Story19	61,75	Top	10264,75	1137,48
		Bottom	4403,37	603,32			Bottom	10264,75	1137,48
Story41	133,25	Top	5250,94	710,65	Story18	58,50	Top	10591,13	1143,05
		Bottom	5250,94	710,65			Bottom	10591,13	1143,05
Story40	130,00	Top	5962,98	790,81	Story17	55,25	Top	10946,58	1153,84
		Bottom	5962,98	790,81			Bottom	10946,58	1153,84
Story39	126,75	Top	6556,58	845,75	Story16	52,00	Top	11330,20	1173,22
		Bottom	6556,58	845,75			Bottom	11330,20	1173,22
Story38	123,50	Top	7047,29	878,89	Story15	48,75	Top	11738,88	1203,60
		Bottom	7047,29	878,89			Bottom	11738,88	1203,60
Story37	120,25	Top	7449,08	894,65	Story14	45,50	Top	12163,90	1246,00
		Bottom	7449,08	894,65			Bottom	12163,90	1246,00
Story36	117,00	Top	7774,19	898,06	Story13	42,25	Top	12598,97	1300,17
		Bottom	7774,19	898,06			Bottom	12598,97	1300,17
Story35	113,75	Top	8033,07	894,47	Story12	39,00	Top	13037,87	1364,27
		Bottom	8033,07	894,47			Bottom	13037,87	1364,27
Story34	110,50	Top	8235,02	888,94	Story11	35,75	Top	13473,81	1435,73
		Bottom	8235,02	888,94			Bottom	13473,81	1435,73
Story33	107,25	Top	8389,15	885,79	Story10	32,50	Top	13900,25	1511,08
		Bottom	8389,15	885,79			Bottom	13900,25	1511,08
Story32	104,00	Top	8504,60	888,37	Story9	29,25	Top	14311,00	1586,80
		Bottom	8504,60	888,37			Bottom	14311,00	1586,80
Story31	100,75	Top	8591,47	898,88	Story8	26,00	Top	14699,42	1659,44
		Bottom	8591,47	898,88			Bottom	14699,42	1659,44
Story30	97,50	Top	8659,47	917,82	Story7	22,75	Top	15059,42	1725,92
		Bottom	8659,47	917,82			Bottom	15059,42	1725,92
Story29	94,25	Top	8717,20	943,92	Story6	19,50	Top	15383,60	1783,55
		Bottom	8717,20	943,92			Bottom	15383,60	1783,55
Story28	91,00	Top	8773,73	974,89	Story5	16,25	Top	15664,84	1830,35
		Bottom	8773,73	974,89			Bottom	15664,84	1830,35
Story27	87,75	Top	8837,24	1007,75	Story4	13,00	Top	15894,34	1864,80
		Bottom	8837,24	1007,75			Bottom	15894,34	1864,80
Story26	84,50	Top	8915,18	1039,64	Story3	9,75	Top	16063,68	1886,02
		Bottom	8915,18	1039,64			Bottom	16063,68	1886,02
Story25	81,25	Top	9014,21	1068,38	Story2	6,50	Top	16166,35	1893,85
		Bottom	9014,21	1068,38			Bottom	16166,35	1893,85
Story24	78,00	Top	9139,80	1092,19	Story1	3,25	Top	16194,97	1888,05
		Bottom	9139,80	1092,19			Bottom	16194,97	1888,05
Story23	74,75	Top	9295,98	1110,15	Base	0,00	Top	0,00	0,00
		Bottom	9295,98	1110,15			Bottom	0,00	0,00

**APPENDIX F: Story shear forces due to earthquake loading in Y
direction- ACI 318-11**

Story	Elevation	Location	X-dir [kN]	Y-dir [kN]	Story	Elevation	Location	X-dir [kN]	Y-dir [kN]
Story45	145,00	Top	133,97	588,32	Story22	71,50	Top	960,75	5511,32
		Bottom	133,97	588,32			Bottom	960,75	5511,32
Story44	143,00	Top	297,24	1282,86	Story21	68,25	Top	985,42	5629,15
		Bottom	297,24	1282,86			Bottom	985,42	5629,15
Story43	139,75	Top	452,72	1910,03	Story20	65,00	Top	1020,89	5749,26
		Bottom	452,72	1910,03			Bottom	1020,89	5749,26
Story42	136,50	Top	587,14	2427,27	Story19	61,75	Top	1066,39	5872,95
		Bottom	587,14	2427,27			Bottom	1066,39	5872,95
Story41	133,25	Top	701,47	2856,20	Story18	58,50	Top	1120,18	6001,05
		Bottom	701,47	2856,20			Bottom	1120,18	6001,05
Story40	130,00	Top	795,87	3209,77	Story17	55,25	Top	1181,21	6134,98
		Bottom	795,87	3209,77			Bottom	1181,21	6134,98
Story39	126,75	Top	871,55	3495,53	Story16	52,00	Top	1248,09	6277,67
		Bottom	871,55	3495,53			Bottom	1248,09	6277,67
Story38	123,50	Top	930,97	3722,64	Story15	48,75	Top	1318,53	6431,41
		Bottom	930,97	3722,64			Bottom	1318,53	6431,41
Story37	120,25	Top	976,04	3903,47	Story14	45,50	Top	1390,46	6597,30
		Bottom	976,04	3903,47			Bottom	1390,46	6597,30
Story36	117,00	Top	1007,88	4050,55	Story13	42,25	Top	1461,79	6776,55
		Bottom	1007,88	4050,55			Bottom	1461,79	6776,55
Story35	113,75	Top	1028,20	4174,29	Story12	39,00	Top	1530,65	6969,00
		Bottom	1028,20	4174,29			Bottom	1530,65	6969,00
Story34	110,50	Top	1039,27	4282,57	Story11	35,75	Top	1595,93	7172,75
		Bottom	1039,27	4282,57			Bottom	1595,93	7172,75
Story33	107,25	Top	1042,50	4381,14	Story10	32,50	Top	1655,93	7383,44
		Bottom	1042,50	4381,14			Bottom	1655,93	7383,44
Story32	104,00	Top	1038,65	4474,05	Story9	29,25	Top	1710,12	7596,91
		Bottom	1038,65	4474,05			Bottom	1710,12	7596,91
Story31	100,75	Top	1029,24	4565,43	Story8	26,00	Top	1757,43	7807,97
		Bottom	1029,24	4565,43			Bottom	1757,43	7807,97
Story30	97,50	Top	1016,17	4658,14	Story7	22,75	Top	1797,54	8013,48
		Bottom	1016,17	4658,14			Bottom	1797,54	8013,48
Story29	94,25	Top	1000,53	4753,06	Story6	19,50	Top	1829,94	8208,93
		Bottom	1000,53	4753,06			Bottom	1829,94	8208,93
Story28	91,00	Top	983,39	4851,27	Story5	16,25	Top	1855,00	8389,85
		Bottom	983,39	4851,27			Bottom	1855,00	8389,85
Story27	87,75	Top	967,08	4953,22	Story4	13,00	Top	1872,27	8546,07
		Bottom	967,08	4953,22			Bottom	1872,27	8546,07
Story26	84,50	Top	953,59	5059,08	Story3	9,75	Top	1882,37	8662,96
		Bottom	953,59	5059,08			Bottom	1882,37	8662,96
Story25	81,25	Top	944,10	5168,61	Story2	6,50	Top	1886,21	8723,33
		Bottom	944,10	5168,61			Bottom	1886,21	8723,33
Story24	78,00	Top	940,76	5280,87	Story1	3,25	Top	1884,63	8709,92
		Bottom	940,76	5280,87			Bottom	1884,63	8709,92
Story23	74,75	Top	946,03	5395,15	Base	0,00	Top	0,00	0,00
		Bottom	946,03	5395,15			Bottom	0,00	0,00

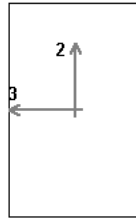
**APPENDIX G: Maximum story displacements due to earthquake loading
in X direction- ACI 318-11**

Story	Elevation	Location	X-dir [mm]	Y-dir [mm]
Story45	145,00	Top	64,30	35,00
Story44	143,00	Top	63,50	34,80
Story43	139,75	Top	62,20	34,40
Story42	136,50	Top	60,90	33,90
Story41	133,25	Top	59,50	33,50
Story40	130,00	Top	58,10	33,00
Story39	126,75	Top	56,70	32,50
Story38	123,50	Top	55,20	31,90
Story37	120,25	Top	53,70	31,40
Story36	117,00	Top	52,20	30,80
Story35	113,75	Top	50,70	30,10
Story34	110,50	Top	49,20	29,50
Story33	107,25	Top	47,60	28,80
Story32	104,00	Top	46,10	28,20
Story31	100,75	Top	44,50	27,50
Story30	97,50	Top	42,90	26,70
Story29	94,25	Top	41,30	26,00
Story28	91,00	Top	39,70	25,20
Story27	87,75	Top	38,00	24,40
Story26	84,50	Top	36,40	23,60
Story25	81,25	Top	34,80	22,80
Story24	78,00	Top	33,10	21,90
Story23	74,75	Top	31,50	21,00
Story22	71,50	Top	29,90	20,10
Story21	68,25	Top	28,20	19,20
Story20	65,00	Top	26,60	18,30
Story19	61,75	Top	24,90	17,40
Story18	58,50	Top	23,30	16,40
Story17	55,25	Top	21,70	15,50
Story16	52,00	Top	20,00	14,50
Story15	48,75	Top	18,40	13,50
Story14	45,50	Top	16,80	12,50
Story13	42,25	Top	15,20	11,50
Story12	39,00	Top	13,60	10,50
Story11	35,75	Top	12,10	9,40
Story10	32,50	Top	10,60	8,40
Story9	29,25	Top	9,10	7,30
Story8	26,00	Top	7,60	6,30
Story7	22,75	Top	6,30	5,20
Story6	19,50	Top	5,00	4,20
Story5	16,25	Top	3,80	3,20
Story4	13,00	Top	2,70	2,30
Story3	9,75	Top	1,70	1,50
Story2	6,50	Top	0,90	0,80
Story1	3,25	Top	0,30	0,30
Base	0,00	Top	0,00	0,00

**APPENDIX H: Maximum story displacements due to earthquake loading
in Y direction- ACI 318-11**

Story	Elevation	Location	X-dir [mm]	Y-dir [mm]
Story45	145,00	Top	35,30	135,50
Story44	143,00	Top	35,00	134,00
Story43	139,75	Top	34,50	131,50
Story42	136,50	Top	34,00	128,90
Story41	133,25	Top	33,40	126,30
Story40	130,00	Top	32,80	123,60
Story39	126,75	Top	32,20	120,80
Story38	123,50	Top	31,60	118,00
Story37	120,25	Top	31,00	115,10
Story36	117,00	Top	30,30	112,20
Story35	113,75	Top	29,60	109,20
Story34	110,50	Top	28,90	106,10
Story33	107,25	Top	28,20	103,00
Story32	104,00	Top	27,50	99,90
Story31	100,75	Top	26,80	96,70
Story30	97,50	Top	26,00	93,50
Story29	94,25	Top	25,20	90,20
Story28	91,00	Top	24,40	87,00
Story27	87,75	Top	23,60	83,60
Story26	84,50	Top	22,80	80,30
Story25	81,25	Top	22,00	76,90
Story24	78,00	Top	21,10	73,50
Story23	74,75	Top	20,30	70,10
Story22	71,50	Top	19,40	66,60
Story21	68,25	Top	18,50	63,10
Story20	65,00	Top	17,60	59,60
Story19	61,75	Top	16,60	56,10
Story18	58,50	Top	15,70	52,60
Story17	55,25	Top	14,70	49,00
Story16	52,00	Top	13,70	45,50
Story15	48,75	Top	12,70	41,90
Story14	45,50	Top	11,70	38,40
Story13	42,25	Top	10,70	34,80
Story12	39,00	Top	9,70	31,30
Story11	35,75	Top	8,70	27,80
Story10	32,50	Top	7,70	24,40
Story9	29,25	Top	6,70	21,00
Story8	26,00	Top	5,70	17,70
Story7	22,75	Top	4,70	14,50
Story6	19,50	Top	3,70	11,50
Story5	16,25	Top	2,80	8,70
Story4	13,00	Top	2,00	6,10
Story3	9,75	Top	1,30	3,80
Story2	6,50	Top	0,70	1,90
Story1	3,25	Top	0,20	0,60
Base	0,00	Top	0,00	0,00

APPENDIX I : Beam design results from ETABS - EN 1992-1:2004



Beam Element Details Type: DC High

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story10	B46	BEAM	Comb19	0	4400	1

Section Properties

b (mm)	h (mm)	b _f (mm)	d _s (mm)	d _{ct} (mm)	d _{cb} (mm)
500	800	500	0	40	40

Material Properties

E _c (MPa)	f _{ck} (MPa)	Lt.Wt Factor (Unitless)	E _s (MPa)	f _{yk} (MPa)	f _{ywk} (MPa)
32599,84	40	1	200000	413,69	413,69

Design Code Parameters

γ _c	γ _s	α _{cc}	α _{ct}	α _{LCC}	α _{LCT}
1,5	1,15	1	1	0,85	0,85

Design Moment and Flexural Reinforcement for Moment, M_{Ed3}

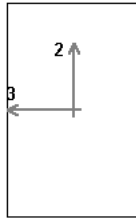
	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm ²	+Moment Rebar mm ²	Minimum Rebar mm ²	Required Rebar mm ²
Top (+2 Axis)	-825,6022		3202	0	1612	3202
Bottom (-2 Axis)		756,308	0	2917	1612	2917

Shear Force and Reinforcement for Shear, V_{Ed2}

Shear V _{Ed} kN	θ deg	Shear V _{Rdc} kN	Shear V _{Rds} kN	Rebar A _{sw} /S mm ² /m
447,0416	45	222,8561	447,0416	1816,85

Torsion Force and Torsion Reinforcement for Torsion, T_{Ed}

Torsion T _{Ed} kN-m	T _{cr} kN-m	Area A _k cm ²	Perimeter, u _k mm	Rebar A _t /s mm ² /m	Rebar A _{sl} /s mm ² /m
44,2887	0	2236,7	1984,6	519,99	546212,34



Beam Element Details Type: DC High

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story10	B46	BEAM	Comb19	2400	4400	1

Section Properties

b (mm)	h (mm)	b _f (mm)	d _s (mm)	d _{ct} (mm)	d _{cb} (mm)
500	800	500	0	40	40

Material Properties

E _c (MPa)	f _{ck} (MPa)	Lt.Wt Factor (Unitless)	E _s (MPa)	f _{yk} (MPa)	f _{ywk} (MPa)
32599,84	40	1	200000	413,69	413,69

Design Code Parameters

γ _c	γ _s	α _{cc}	α _{ct}	α _{LCC}	α _{LCT}
1,5	1,15	1	1	0,85	0,85

Design Moment and Flexural Reinforcement for Moment, M_{Ed3}

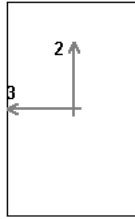
	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm ²	+Moment Rebar mm ²	Minimum Rebar mm ²	Required Rebar mm ²
Top (+2 Axis)	-206,4006		765	0	1612	1612
Bottom (-2 Axis)		206,4006	0	765	1612	1612

Shear Force and Reinforcement for Shear, V_{Ed2}

Shear V _{Ed} kN	θ deg	Shear V _{Rdc} kN	Shear V _{Rds} kN	Rebar A _{sw} /S mm ² /m
324,9089	45	177,2723	345,1439	1402,72

Torsion Force and Torsion Reinforcement for Torsion, T_{Ed}

Torsion T _{Ed} kN-m	T _{cr} kN-m	Area A _k cm ²	Perimeter, u _k mm	Rebar A _t /s mm ² /m	Rebar A _{sl} /s mm ² /m
5,2256	0	2236,7	1984,6	61,35	64447,8



Beam Element Details Type: DC High

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF
Story10	B46	BEAM	Comb19	4400	4400	1

Section Properties

b (mm)	h (mm)	b _f (mm)	d _s (mm)	d _{ct} (mm)	d _{cb} (mm)
500	800	500	0	40	40

Material Properties

E _c (MPa)	f _{ck} (MPa)	Lt.Wt Factor (Unitless)	E _s (MPa)	f _{yk} (MPa)	f _{ywk} (MPa)
32599,84	40	1	200000	413,69	413,69

Design Code Parameters

γ _c	γ _s	α _{cc}	α _{ct}	α _{LCC}	α _{LCT}
1,5	1,15	1	1	0,85	0,85

Design Moment and Flexural Reinforcement for Moment, M_{Ed3}

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm ²	+Moment Rebar mm ²	Minimum Rebar mm ²	Required Rebar mm ²
Top (+2 Axis)	-747,2993		2881	0	1612	2881
Bottom (-2 Axis)		659,3969	0	2525	1612	2525

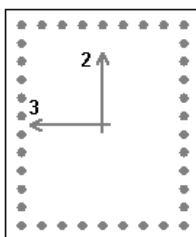
Shear Force and Reinforcement for Shear, V_{Ed2}

Shear V _{Ed} kN	θ deg	Shear V _{Rdc} kN	Shear V _{Rds} kN	Rebar A _{sw} /S mm ² /m
349,2379	45	205,8976	385,09	1565,07

Torsion Force and Torsion Reinforcement for Torsion, T_{Ed}

Torsion T _{Ed} kN-m	T _{cr} kN-m	Area A _k cm ²	Perimeter, u _k mm	Rebar A _t /s mm ² /m	Rebar A _{sl} /s mm ² /m
22,0579	0	2236,7	1984,6	258,98	272040,73

APPENDIX J: Column design results from ETABS- EN 1992-1:2008



Column Element Details Type: DC High

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	SOM	LLRF
Story1	C2	COLUMN 1 (1-15)	Comb5	0	3250		0,4

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1000	1200	58,5	30

Material Properties

E _c (MPa)	f _{ck} (MPa)	Lt.Wt Factor (Unitless)	E _s (MPa)	f _{yk} (MPa)	f _{ywk} (MPa)
32599,84	40	1	200000	413,69	413,69

Design Code Parameters

γ _c	γ _s	α _{cc}	α _{CT}	α _{LCC}	α _{LCT}
1,5	1,15	1	1	0,85	0,85

Axial Force and Biaxial Moment Design For N_{Ed} , M_{Ed2} , M_{Ed3}

Design N _{Ed} kN	Design M _{Ed2} kN-m	Design M _{Ed3} kN-m	Minimum M ₂ kN-m	Minimum M ₃ kN-m	Rebar % %	Capacity Ratio Unitless
28581,203	-952,7068	-1345,207	952,7068	1143,2481	1,15	0,865

Axial Force and Biaxial Moment Factors

	M _{0Ed} Moment kN-m	M _{add} Moment kN-m	Minimum Ecc mm	β Factor Unitless	Length mm
Major Bend(M3)	-1103,0804	0	40	1	2450
Minor Bend(M2)	-952,7068	0	33,3	1	2450

Axial Compression Ratio

Conc Capacity (α _{cc} * A * f _{cd}) kN	Compressive Ratio N _{Ed} / (α _{cc} * A * f _{cd})	Comp Ratio Limit	Seismic Load?	Ratio OKay?
32000	0,893	0,55	No	Yes

Shear Design for V_{Ed2} , V_{Ed3}

	Shear V _{Ed} kN	Shear V _{Rdc} kN	Shear V _{Rds} kN	tan(θ) Unitless	Rebar A _{sw} / s mm ² /m
Major, V _{Ed2}	243,2416	1474,3468	0	0,4	0
Minor, V _{Ed3}	98,7466	1477,7709	0	0,4	0

APPENDIX K: Shear wall design results from ETABS- EN 1992-1:2004

Arrangement 1:

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
Story1	CORE 4	31934,4	8660,6	12470	500	0,4

Material Properties

E _c (MPa)	f _{ck} (MPa)	Lt.Wt Factor (Unitless)	f _{yk} (MPa)	f _{ywk} (MPa)
35000	C40/50 (40)	C40/50 (1)	A615Gr60 (413,69)	A615Gr60 (413,69)

Design Code Parameters

γ _c	γ _s	α _{cc}	α _{LCC}	IP _{MAX}	IP _{MIN}	P _{MAX}
1,5	1,15	1	0,85	0,04	0,005	0,8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X ₁ mm	Left Y ₁ mm	Right X ₂ mm	Right Y ₂ mm	Length mm	Thickness mm
Top	Leg 1	35380	8250	35380	11450	3200	500
Top	Leg 2	26110	8250	35380	8250	9270	500
Bottom	Leg 1	35380	8250	35380	11450	3200	500
Bottom	Leg 2	26110	8250	35380	8250	9270	500

Flexural Design for N_{Ed} , M_{Ed2} and M_{Ed3}

Station Location	Required Rebar Area (mm ²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	N _{Ed} kN	M _{Ed2} kN-m	M _{Ed3} kN-m	Pier A _g mm ²
Top	44641	0,0068	0,0157	DWal18	13792,1989	-13558,6789	26192,7049	6591286
Bottom	48251	0,0073	0,0157	DWal18	14298,6431	-14428,8462	25938,6527	6591286

Shear Design

Station Location	ID	Rebar mm ² /m	Shear Combo	N _{Ed} kN	V _{Ed} kN	V _{Rc} kN	V _{Rd} kN
Top	Leg 1	1285,16	DWal5	1836,4333	2958,768	865,7257	2958,768
Top	Leg 2	3600,56	DWal17	4649,9062	9605,3307	2253,33	9605,3307
Bottom	Leg 1	1285,16	DWal5	2011,8815	2958,768	907,5416	2958,768
Bottom	Leg 2	3600,56	DWal17	5026,3888	9605,3307	2350,6547	9605,3307

Boundary Element Check

Station Location	ID	Edge Length (mm)	Governing Combo	N _{Ed} kN	M _{Ed} kN-m	Normalized Comp. Stress	Normalized Stress Limit	C Depth mm
Top-Left	Leg 1	2174,4	DWal11	25011,7599	-641,3754	0,614	0,15	2478
Top-Right	Leg 1	2174,4	DWal11	25011,7599	710,0491	0,617	0,15	2478
Top-Left	Leg 2	5508,7	DWal12	63078,7067	-4470,7831	0,534	0,15	6374,7
Top-Right	Leg 2	5508,7	DWal12	63078,7067	1914,217	0,52	0,15	6374,7
Bottom-Left	Leg 1	2212	DWal11	25141,7216	-1409,4554	0,651	0,15	2516,2
Bottom-Right	Leg 1	2212	DWal11	25141,7216	1450,0116	0,653	0,15	2516,2
Bottom-Left	Leg 2	5617,2	DWal12	63455,1892	-5869,615	0,544	0,15	6485,2
Bottom-Right	Leg 2	5617,2	DWal12	63455,1892	3033,5171	0,529	0,15	6485,2

Arrangement 2

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
Story1	CORE 4	31934,4	8660,6	12470	500	0,4

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt.Wt Factor (Unitless)	f_{yk} (MPa)	f_{ywk} (MPa)
35000	C40/50 (40)	C40/50 (1)	A615Gr60 (413,69)	A615Gr60 (413,69)

Design Code Parameters

γ_c	γ_s	α_{cc}	α_{LCC}	IP_{MAX}	IP_{MIN}	P_{MAX}
1,5	1,15	1	0,85	0,04	0,005	0,8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X_1 mm	Left Y_1 mm	Right X_2 mm	Right Y_2 mm	Length mm	Thickness mm
Top	Leg 1	35380	8250	35380	11450	3200	500
Top	Leg 2	26110	8250	35380	8250	9270	500
Bottom	Leg 1	35380	8250	35380	11450	3200	500
Bottom	Leg 2	26110	8250	35380	8250	9270	500

Flexural Design for N_{Ed} , M_{Ed2} and M_{Ed3}

Station Location	Required Rebar Area (mm ²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	N_{Ed} kN	M_{Ed2} kN-m	M_{Ed3} kN-m	Pier A_g mm ²
Top	47037	0,0075	0,0086	DW al18	13792,1989	-13558,6789	26192,7049	6235000
Bottom	52625	0,0084	0,0086	DW al18	14298,6431	-14428,8462	25938,6527	6235000

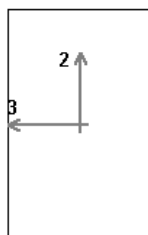
Shear Design

Station Location	ID	Rebar mm ² /m	Shear Combo	N_{Ed} kN	V_{Ed} kN	V_{Rc} kN	V_{Rd} kN
Top	Leg 1	1285,16	DW al5	1836,4333	2958,768	887,3286	2958,768
Top	Leg 2	3600,56	DW al17	4649,9062	9605,3307	2310,2722	9605,3307
Bottom	Leg 1	1285,16	DW al5	2011,8815	2958,768	936,9745	2958,768
Bottom	Leg 2	3600,56	DW al17	5026,3888	9605,3307	2428,2356	9605,3307

Boundary Element Check

Station Location	ID	Edge Length (mm)	Governing Combo	N_{Ed} kN	M_{Ed} kN-m	Normalized Comp. Stress	Normalized Stress Limit	C Depth mm
Top-Left	Leg 1	2211,9	DW al11	25011,7599	-641,3754	0,614	0,15	2516
Top-Right	Leg 1	2211,9	DW al11	25011,7599	710,0491	0,617	0,15	2516
Top-Left	Leg 2	5616,9	DW al12	63078,7067	-4470,7831	0,534	0,15	6484,9
Top-Right	Leg 2	5616,9	DW al12	63078,7067	1914,217	0,52	0,15	6484,9
Bottom-Left	Leg 1	2266,5	DW al11	25141,7216	-1409,4554	0,651	0,15	2571,4
Bottom-Right	Leg 1	2266,5	DW al11	25141,7216	1450,0116	0,653	0,15	2571,4
Bottom-Left	Leg 2	5774,4	DW al12	63455,1892	-5869,615	0,544	0,15	6645,2
Bottom-Right	Leg 2	5774,4	DW al12	63455,1892	3033,5171	0,529	0,15	6645,2

APPENDIX L : Beam design results from ETABS – ACI 318-11



Beam Element Details (Summary)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story10	B46	BEAM	Comb15	0	4400	1	Sway Special

Section Properties

b (mm)	h (mm)	b _r (mm)	d _s (mm)	d _{ct} (mm)	d _{cb} (mm)
500	800	500	0	40	40

Material Properties

E _c (MPa)	f _c (MPa)	Lt.Wt Factor (Unitless)	f _y (MPa)	f _{ys} (MPa)
35000	40	1	413,69	413,69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{Cspiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}
0,9	0,65	0,75	0,75	0,6	0,85

Design Moment and Flexural Reinforcement for Moment, M_{u3}

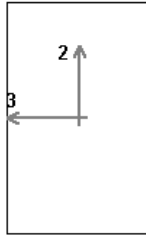
	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm ²	+Moment Rebar mm ²	Minimum Rebar mm ²	Required Rebar mm ²
Top (+2 Axis)	-1218,2207		4652	0	1447	4652
Bottom (-2 Axis)		1112,8467	0	4218	1447	4218

Shear Force and Reinforcement for Shear, V_{u2}

Shear V _{u2} kN	Shear ΦV_c kN	Shear ΦV_s kN	Shear V _p kN	Rebar A _v /S mm ² /m
661,9745	0	661,9745	709,9045	3509,18

Torsion Force and Torsion Reinforcement for Torsion, T_u

Φ^*T_u kN-m	T _{cr} kN-m	Area A _o cm ²	Perimeter, p _h mm	Rebar A _t /s mm ² /m	Rebar A _t mm ²
72,0049	24,238	2484,8	2244,4	466,99	1491



Beam Element Details (Summary)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story10	B46	BEAM	Comb15	2400	4400	1	Sway Special

Section Properties

b (mm)	h (mm)	b _f (mm)	d _s (mm)	d _{ct} (mm)	d _{cb} (mm)
500	800	500	0	40	40

Material Properties

E _c (MPa)	f' _c (MPa)	Lt.Wt Factor (Unitless)	f _y (MPa)	f _{ys} (MPa)
35000	40	1	413,69	413,69

Design Code Parameters

Φ _T	Φ _{CTied}	Φ _{CSpiral}	Φ _{Vns}	Φ _{Vs}	Φ _{Vjoint}
0,9	0,65	0,75	0,75	0,6	0,85

Design Moment and Flexural Reinforcement for Moment, M_{u3}

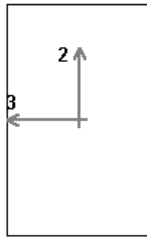
	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm ²	+Moment Rebar mm ²	Minimum Rebar mm ²	Required Rebar mm ²
Top (+2 Axis)	-304,5552		1096	0	1447	1447
Bottom (-2 Axis)		304,5552	0	1096	1447	1447

Shear Force and Reinforcement for Shear, V_{u2}

Shear V _{u2} kN	Shear ΦV _c kN	Shear ΦV _s kN	Shear V _p kN	Rebar A _v /S mm ² /m
715,1755	0	715,1755	709,9045	3032,97

Torsion Force and Torsion Reinforcement for Torsion, T_u

Φ*T _u kN-m	T _{cr} kN-m	Area A _o cm ²	Perimeter, p _h mm	Rebar A _t /s mm ² /m	Rebar A _t mm ²
6,9284	24,238	2484,8	2244,4	0	0



Beam Element Details (Summary)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story10	B46	BEAM	Comb15	4400	4400	1	Sway Special

Section Properties

b (mm)	h (mm)	b _f (mm)	d _s (mm)	d _{ct} (mm)	d _{cb} (mm)
500	800	500	0	40	40

Material Properties

E _c (MPa)	f' _c (MPa)	Lt.Wt Factor (Unitless)	f _y (MPa)	f _{ys} (MPa)
35000	40	1	413,69	413,69

Design Code Parameters

Φ _T	Φ _{CTied}	Φ _{Cspiral}	Φ _{Vns}	Φ _{Vs}	Φ _{Vjoint}
0,9	0,65	0,75	0,75	0,6	0,85

Design Moment and Flexural Reinforcement for Moment, M_{u3}

	Design -Moment kN-m	Design +Moment kN-m	-Moment Rebar mm ²	+Moment Rebar mm ²	Minimum Rebar mm ²	Required Rebar mm ²
Top (+2 Axis)	-1105,4535		4187	0	1447	4187
Bottom (-2 Axis)		968,2224	0	3633	1447	3633

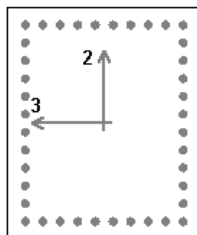
Shear Force and Reinforcement for Shear, V_{u2}

Shear V _{u2} kN	Shear ΦV _c kN	Shear ΦV _s kN	Shear V _p kN	Rebar A _v /S mm ² /m
776,2413	0	776,2413	709,9045	3291,94

Torsion Force and Torsion Reinforcement for Torsion, T_u

Φ*T _u kN-m	T _{cr} kN-m	Area A _o cm ²	Perimeter, p _h mm	Rebar A _t /s mm ² /m	Rebar A _t mm ²
32,845	24,238	2484,8	2244,4	213,02	2061

APPENDIX M: Column design results from ETABS- ACI 318-11



Column Element Details (Flexural Details)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story1	C2	COLUMN 1 (1-15)	Comb15	0	3250	0,4	Sway Special

Section Properties

b (mm)	h (mm)	dc (mm)	Cover (Torsion) (mm)
1000	1200	90	37,3

Material Properties

E_c (MPa)	f'_c (MPa)	Lt.Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
35000	40	1	413,69	413,69

Design Code Parameters

Φ_T	Φ_{CTied}	$\Phi_{CSpiral}$	Φ_{Vns}	Φ_{Vs}	Φ_{Vjoint}
0,9	0,65	0,75	0,75	0,6	0,85

Axial Force and Biaxial Moment Design For P_u , M_{u2} , M_{u3}

Design P_u kN	Design M_{u2} kN-m	Design M_{u3} kN-m	Minimum M2 kN-m	Minimum M3 kN-m	Rebar Area mm ²	Rebar % %
29922,1078	679,703	-2347,9851	1353,6762	1533,2088	45952	3,83

Factored & Minimum Biaxial Moments

	NonSway M_{ns} kN-m	Sway M_s kN-m	Factored M_u kN-m	Minimum M_{min} kN-m	Minimum Eccentricity mm
Major Bending(M_{u3})	-44,0779	-2260,3043	-2304,3822	1533,2088	51,2
Minor Bending(M_{u2})	29,4633	632,0635	661,5269	1353,6762	45,2

Axial Force and Biaxial Moment Factors

	C_m Factor Unitless	δ_{ns} Factor Unitless	δ_s Factor Unitless	K Factor Unitless	Length mm
Major Bend(M3)	1	1,018922	1	1	2450
Minor Bend(M2)	1	1,027476	1	1	2450

Column Element Details (Shear Details)

Level	Element	Section ID	Combo ID	Station Loc	Length (mm)	LLRF	Type
Story1	C2	COLUMN 1 (1-15)	Comb24	0	3250	0,4	Sway Special

Shear Design for V_{u2} , V_{u3}

	Rebar A_v /s mm ² /m	Design V_u kN	Design P_u kN	Design M_u kN-m	ΦV_c kN	ΦV_s kN	ΦV_n kN
Major Shear(V2)	1582,14	435,9014	-2149,1061	-2278,1043	0	435,9014	435,9014
Minor Shear(V3)	1142,51	164,4575	-2149,1061	-619,6412	0	258,0621	258,0621

Design Forces

	Factored V_u kN	Factored P_u kN	Factored M_u kN-m	Capacity V_p kN
Major Shear(V2)	435,9014	-2149,1061	2242,5044	0
Minor Shear(V3)	164,4575	-2149,1061	644,4859	0

Capacity Shear (Part 1 of 2)

	Shear V_p kN	Long.Rebar $A_{s(Bot)}$ %	Long.Rebar $A_{s(Top)}$ %	Cap.Moment M_{posBot} kN-m
Major Shear(V2)	0	3,83	3,72	10080,5397
Minor Shear(V3)	0	3,83	3,72	8414,7862

Capacity Shear (Part 2 of 2)

Cap.Moment M_{negTop} kN-m	Cap.Moment M_{negBot} kN-m	Cap.Moment M_{posTop} kN-m
9814,0064	10080,5397	9814,0064
8188,2732	8414,7862	8188,2732

Design Basis

Shr Reduc Factor Unitless	Strength f_{ys} MPa	Strength f_{cs} MPa	Area A_g cm ²
1	413,69	40	12000

Concrete Shear Capacity

	Design V_u kN	Conc.Area A_{cu} cm ²	Tensn.Rein A_{st} mm ²
Major Shear(V2)	435,9014	11100	22976
Minor Shear(V3)	164,4575	10920	22976

Shear Rebar Design

	Stress v MPa	Conc.Cpcty v_c MPa	Uppr.Limit v_{max} MPa	Φv_c MPa	Φv_{max} MPa	RebarArea A_v /s mm ² /m
Major Shear(V2)	0,39	0,5	4,2	0,3	0	1582,14
Minor Shear(V3)	0,15	0,5	4,2	0,3	2,52	1142,51

APPENDIX N: Shear wall design results from ETABS- ACI 318-11

ACI 318-11 Pier Design

Pier Details

Story ID	Pier ID	Centroid X (mm)	Centroid Y (mm)	Length (mm)	Thickness (mm)	LLRF
Story1	CORE 4	31934,4	8660,6	12470	500	0,4

Material Properties

E _c (MPa)	f' _c (MPa)	Lt.Wt Factor (Unitless)	f _y (MPa)	f _{ys} (MPa)
35000	40	1	413,69	413,69

Design Code Parameters

Φ_T	Φ_C	Φ_v	Φ_v (Seismic)	IP _{MAX}	IP _{MIN}	P _{MAX}
0,9	0,65	0,75	0,6	0,04	0,0025	0,8

Pier Leg Location, Length and Thickness

Station Location	ID	Left X ₁ mm	Left Y ₁ mm	Right X ₂ mm	Right Y ₂ mm	Length mm	Thickness mm
Top	Leg 1	35380	8250	35380	11450	3200	500
Top	Leg 2	26110	8250	35380	8250	9270	500
Bottom	Leg 1	35380	8250	35380	11450	3200	500
Bottom	Leg 2	26110	8250	35380	8250	9270	500

Flexural Design for P_u, M_{u2} and M_{u3}

Station Location	Required Rebar Area (mm ²)	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	P _u kN	M _{u2} kN-m	M _{u3} kN-m	Pier A _g mm ²
Top	63073	0,0096	0,0157	DWal41	8281,7375	-14691,1671	37096,3736	6591286
Bottom	85566	0,013	0,0157	DWal41	8686,8928	-18604,7162	41083,602	6591286

Shear Design

Station Location	ID	Rebar mm ² /m	Shear Combo	P _u kN	M _u kN-m	V _u kN	ΦV_c kN	ΦV_n kN
Top	Leg 1	2318,38	DWal30	-6596,6519	-14304,7577	2048,0042	206,5695	2048,0042
Top	Leg 2	1250	DWal36	36015,0093	28914,5587	5606,1575	4381,3884	7257,5366
Bottom	Leg 1	2266,38	DWal30	-6479,6864	-20960,7713	2048,0042	247,8746	2048,0042
Bottom	Leg 2	1250	DWal36	36504,4366	46247,0174	5606,1575	4381,3884	7257,5366

Boundary Element Check

Station Location	ID	Edge Length (mm)	Governing Combo	P _u kN	M _u kN-m	Stress Comp MPa	Stress Limit MPa	C Depth mm	C Limit mm
Top-Left	Leg 1	1070,3	DWal36	30086,6725	-819,1327	19,76	8	1390,3	761,9
Top-Right	Leg 1	1073,6	DWal36	30086,6725	911,2003	19,87	8	1393,6	761,9
Top-Left	Leg 2	2650,5	DWal37	76686,6687	-8813,8943	17,78	8	3577,5	2207,1
Top-Right	Leg 2	2606,4	DWal37	76686,6687	5196,0748	17,27	8	3533,4	2207,1
Bottom-Left	Leg 1	1114,9	DWal36	30255,6226	-1863,4652	21,09	8	1434,9	761,9
Bottom-Right	Leg 1	1117,3	DWal36	30255,6226	1932,3856	21,17	8	1437,3	761,9
Bottom-Left	Leg 2	2750,4	DWal37	77176,096	-15178,2878	18,77	8	3677,4	2207,1
Bottom-Right	Leg 2	2701,7	DWal37	77176,096	11194,8874	18,21	8	3628,7	2207,1

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