



Dissertation

Master in Building Construction

***Ductility Considerations in Seismic Design of
Reinforced Concrete Building***

Mahmoud Helal SaadEldin Elawady

Leiria, December of 2017

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***Ductility Considerations in Seismic Design of
Reinforced Concrete Building***

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Dissertation developed under the supervision of Hugo Rodrigues, adjunct professor at the School of Technology and Management of the Polytechnic Institute of Leiria.

Leiria, *December of 2017*

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Dedication

I dedicate this thesis to the soul my parents; who were always supporting me during whole my life, they were a sample for giveness and the sacrifice trying to give me have a better future, they were alawys take care of my furture and encourage me so much to study and learn more as much as it is possible, they had a stong believe in me, they make me believe in myself. I know they already passed away, but they are with me in each step in my life to support me, and they are happy for me.

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Resumo (in Portuguese)

Este estudo foi realizado por fim a avaliar os métodos propostos pela norma sísmica europeia, a fim de verificar as vantagens da aplicação de diferentes classes de ductilidade no dimensionamento sísmico de edifícios de betão armado. Assim, foi dimensionado um edifício de betão armado de acordo com o prescrito no Eurocódigo 8 para as diferentes classes de ductilidade, e o final foi avaliada a sua performance sísmica de acordo com uma análise pushover, de acordo com o EC8. Considerou-se que o edifício em estudo está localizado em diferentes zonas de perigo sísmico (perigo baixo, médio e alto) no continente de Portugal, sujeitas a cargas horizontais definidas no código documento de aplicação nacional de acompanha o Eurocode 8.

Há uma recomendação estrita do código sísmico europeu (Eurocódigo 8) (EN 1998-1, 2004), para considerar uma das três classes de ductilidade mencionadas no Eurocódigo 8, classe baixa de ductilidade DC L, classe média de ductilidade DC M e alta classe de ductilidade DC H, o Eurocode 8 explica detalhadamente as regras para projetar a estrutura para a classe de ductilidade de três, no entanto, não é claro para o designer selecionar a classe de ductilidade adequada para que a estrutura tenha alcançado o maior desempenho da estrutura do edifício durante o terremoto com menos danos e custos econômicos.

A análise estática não-linear de empuxo, usada para avaliar o desenho dos quadros para três classes de ductilidade em diferentes zonas sísmicas, os resultados da avaliação mostram que, Ductility classe média DC M, têm um alto desempenho próximo a Ductility classe alta DC H, mesmo no alto as zonas sísmicas de perigo e os custos para DC M estão próximos de DC H, poderia ser menor se a mão-de-obra for incluída, porque a DC H custará mais pela mão-de-obra de acordo com a complexidade dos detalhes.

Palavras-chave:

Dimensionamento sísmico, ductilidade, avaliação do comportamento sísmico, pushover

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Abstract

This study is conducted to review the methods proposed by European national seismic codes in order to incorporate the concept of ductility of structural elements. In order to assess how to take in account the ductility class of a reinforced concrete building which is prescribed in Eurocode 8 and its influence in the structural design of the building, the structures are analysed with Sap2000 software for their responses to lateral loading by applying the static non linear push-over analysis. These structures are assumed to be located in different seismic hazard zones (Low, Medium and High hazard) in the mainland of Portugal, subjected to horizontal loads deducted from the European seismic code and designed using Eurocode 8.

There a strict recommendation from the European seismic code (Eurocode 8) (EN 1998-1, 2004), to consider one of three ductility classes mentioned in Eurocode 8, ductility class low DC L, ductility class medium DC M, and ductility class high DC H, Eurocode 8 explains in details the rules to design the structure for the the three ductility class, however it is not clear for the designer to select the suitable ductility class for the structure to have achieve the highest performance of building structure during the earthquake with less damages and economic cost.

Nonlinear pushover static analysis, used to assess the design of the frames for three ductility classes in different seismic zones, the assessment results show that, Ductility class medium DC M, have a high performance near to Ductility class high DC H, even in the high hazard seismic zones, and the costs for DC M are close for DC H, could be less, if the workmanship is included, because DC H will cost more for the workmanship according to its complexity of the detailing.

Keywords:

Seismic design, ductility, seismic performance, pushover

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1. Introduction

1.1. Objectives

This work is aimed to study the differences obtained with different ductility considerations in the design of reinforced concrete frame building. The Eurocode 8 (EN 1998-1, 2004) mention three different ductility classes for the seismic design, however it is not so clear how the designer can choose the suitable ductility to start the design. Therefore in this work it were studied a reinforced concrete frame building assuming that the building is located in different seismic zones in Portugal, and in each seismic zone, the building will be designed for the three different ductility classes. The results of the designs will show the differences in cross sections and details of the columns and the beams of the frames, which it affects on the total cost of the building, and it will show how it can be more economic if the designer is able to determine suitable ductility for the building depend on previous knowledge of seismic. Economic result is not only the main objective of thesis, building performance during the earthquake is an important factor to study, to achieve minimum damages on the building during the earthquake. In the end both cases economic point view and building performance will be analysed and discussed.

1.2. Motivation

The Eurocode 8 as a provision code for seismic design in Europe, Eurocode 8 recommends three ductility classes for seismic design, however it is not clear for designers to determine the required ductility class for the design, also the effect of the ductility class on the cost and the performance of the building. It is not easy decision to determine suitable ductility class for the structure to resist seismic actions in high performance with the optimum cost.

The aim of the present work was to evaluate the effect of ductility on the performance and the cost of the building.

1.3. Thesis Framework

The present work was arranged in six chapters. The present chapter gives the introduction and motivation for the developed work.

Chapter 2, presents a literature review, giving a background of earthquake and earthquake resisting-building, ductility of reinforced concrete frame building, beam-column joints in RC frame building, and hazard seismic zones in Portugal. In

Chapter 3, is discussed the seismic design of reinforced concrete frame building, comparing between seismic design code provisions for RC frame buildings and the aspects of the design for each code provision, and the rules for design and detail the RC frames according to Eurocode 8.

Chapter 4, is presented the case study of six floors reinforced concrete frame building, designed according to Eurocode 2 as an initial design and according to Eurocode 8 to resist seismic actions, the building has been considered in different seismic zones in Portugal, and designed for the three ductility classes recommended by Eurocode 8, and in.

Chapter 5, considering the seismic design rules of Eurocode 8 mentioned in chapter 3, the building has been designed and detailed for the three ductility classes and comparing between cross sections of concrete and reinforcement, and longitudinal and transverse reinforcement, the total from cost for each ductility class. After the building design in

Chapter 6, is performed the seismic assessment of the building, according to N2 method mentioned in Eurocode 8, using nonlinear pushover analysis. In

Chapter 7, is presented the main conclusion and the future works.

2. Literature Review

2.1. Earthquakes and Earthquake-Resisting building

2.1.1. Global Information about Earthquakes

Earthquakes are one of the most devastating natural hazards that cause great loss of life and livelihood. On average, 10,000 people die each year due to earthquakes, while annual economic losses are in the billions of dollars and often constitute a large percentage of the gross national product of the country affected(Hatheway, 1996). The earthquake threat has distinct characteristics which have important consequences for the ways in which society responds to the threat, and the part that structural engineers will play in that response(Booth, 2014).

The Richter magnitude scale is a logarithmic based scale which utilizes the amplitude of seismic vibrations, recorded on a standard seismograph, to determine the strength of an earthquake. A unit increment on the scale represents a ten-fold increase in amplitude and an increase in energy release of 31.6 times. Earthquakes of Richter magnitude 6, 7 and 8 are categorized respectively as moderate, major and great earthquakes. Richter magnitude 8.5 is estimated to be the maximum that maybe anticipated in California. The Loma Prieta earthquake had a magnitude of 7.1 and was the result of a rupture along a 25-mile-long segment of the San Andreas fault between Los Gatos and Watsonville. The San Francisco earthquake of April 1906 had a magnitude of 8.25 and was the result of a rupture along a 270 mile long segment of the San Andreas fault(Alan Williams, 2000).

The intensity of an earthquake is based on the damage and other observed effects on people, buildings, and other features. Intensity varies from place to place within the disturbed region. An earthquake in a densely populated area that results in many deaths and considerable damage may have the same magnitude as a shock in a remote area that does nothing more than frighten the wildlife. Large magnitude earthquakes that occur beneath the oceans may not even be felt by humans(Lindeburg and McMullin, 2014).

2.1.2. Earthquake-resisting buildings

Major earthquakes in this century have demonstrated repeatedly that well-constructed buildings designed without adequate consideration of seismic forces have often performed satisfactorily when subjected to intense ground motions(Eams and Earls, 2000). Today's design professionals know how to design and construct buildings and other structures that can resist even the most intense earthquake effects with little damage. However, designing structures in this manner can significantly increase their construction cost(Council, 2010).

An earthquake represents for the structure a demand to accommodate a given energy input or given imposed dynamic displacements. Seismic damage to structural elements, or even

to non-structural ones that follow the deformations of the structure, is due to deformations induced by the seismic response. Consistent with this reality, Eurocode 8(EN 1998-1, 2004) states that compliance criteria for the damage limitation limit state (i.e. performance level) should be expressed in terms of deformation limits. For equipment mounted or supported on the structure, limits relevant to damage may be expressed in terms of response accelerations at the positions of the equipment supports (Fardis *et al.*, 2005).

The primary objective of earthquake resistant design is to prevent building collapse during earthquakes thus minimising the risk of death or injury to people in or around those buildings. Because damaging earthquakes are rare, economics dictate that damage to buildings is expected and acceptable provided collapse is avoided(King, 1998).

Earthquake-resistant design of buildings is intended primarily to provide for the inertial effects associated with the waves of distortion that characterizes dynamic response to ground shaking. These effects account for most of the damage resulting from earthquakes. In a few cases, significant damage has resulted from conditions where inertial effects in the structure were negligible(Omer and Amine, no date).

The beneficial influence of the deformation capacity of a structural system, that is ductility in a global sense is now probably widely accepted. However, the sources of this highly desirable, indeed life-saving, structural property are seldom properly traced, or quantified. The exploitation of ductility for its intended purpose, to provide reserve deformation capacity without significant reduction of resistance to lateral forces, is often not fully implemented. Deformation capacity is the most important structural property in areas of high seismic risk and where economic constraints limit the level of seismic resistance that can be afforded (Paulay, 1996).

2.2. Ductility of Reinforced Concrete Frame Buildings

Ductility is the ability of the structure or structural components to undergo inelastic deformation alternating repetitive after the first melting, while maintaining sufficient strength and rigidity to support the load, so that the structure remains standing despite being cracked / damaged and on the verge of collapse(Department of Publics Works, 2002).

For economical resistance against strong earthquakes most structures must behave inelastically. The average plastic energy, dissipated by a structure during the design ground motion, usually gives enough information to evaluate the structural performance. For RC structures, the essential objectives of failure mode control are: brittle failure modes should be suppressed; an appropriate degree of ductility should be provided (but the degree to which the ductility should be enhanced is debatable). While most concrete structures are designed by equivalent static analysis and codified reinforcing rules aimed at providing ductility, it is important for designers to understand how the ductility demand arises. Having made an

estimate of the ductility, the members should be detailed to have the appropriate section ductility (Iskhakov, 2003).

The current design practice relies on the ductile flexural response at plastic hinges as the primary source of energy dissipation during earthquake loading cycles. This requires proper care in detailing the locations where plastic hinges are expected to occur. According to the seismic design provisions in modern building codes, buildings are supposed to resist minor earthquakes without damage, moderate earthquakes without significant structural damage, and in the case of a major earthquake, some structural and non-structural damage is allowed but without collapse (Mantawy, 2015).

Using spectral accelerations that result from a response spectrum, appropriately reduced by a, so called, behaviour factor (q in Europe) or force reduction factor (R in the U.S.). Ultimately, the structural system is designed for a lower level of strength, relying that stable energy absorption will be made feasible through specific geometric and minimum reinforcement requirements along with the associated detailing rules. Fundamental requirements (i.e. collapse prevention, damage limitation, minimum level of serviceability) are also achieved through capacity design for the enhancement of global ductility (Sextos and Skoulidou, 2012). The q factor represents a global characteristic of structural behaviour and a simple tool for designers, which allows performing an equivalent elastic analysis instead an inelastic one (Vaseva, 2003).

Fundamental requirements (i.e. collapse prevention, damage limitation, and minimum level of serviceability) are also achieved through capacity design for the enhancement of global ductility. According to (EN 1998-1, 2004) in particular, the above philosophy is materialised for reinforced concrete buildings through the choice of the Ductility Class, i.e., Low (DCL), Medium (DCM) and High (DCH), each corresponding to different structural and detailing requirements. Notably, the Lower ductility class is only recommended by National Annexes in low seismicity areas or for base-isolated structures (Sextos and Skoulidou, 2012). Chapter 3, discussing the differences of the ductility classes detailing requirement as per to Eurocode 8.

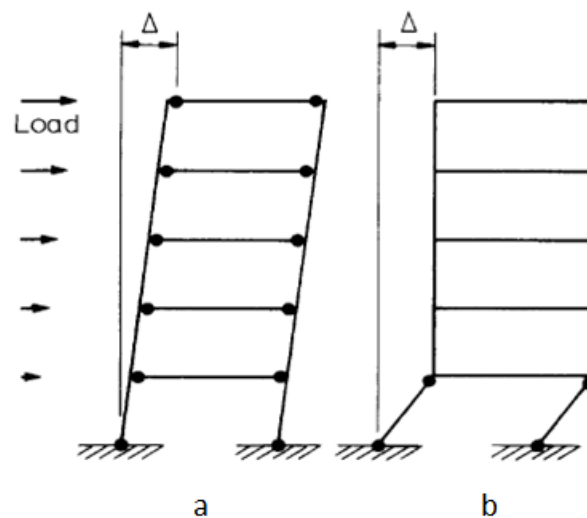


Figure 2.1 Sway mechanisms in laterally loaded multistorey frames.(Paulay, 1983)

Resistance earthquake design of RC building structure should ensure whole property resistance earthquake of structure, according to (Men and Qiu, 1998). Ductility demand of resistance earthquake of RC frame building structure is:

- Strong column and soft beam of RC frame
- Moment regulate in beam end of RC frame
- Shear-pressure ratio in beam of RC frame
- Strong shear and soft curve in column of RC frame
- Strong connect and soft member of RC frame point
- Shear-pressure ratio in column of RC frame
- Shear-pressure ratio in point of RC frame
- Axial pressure ratio in column of RC frame
- Axial force increases for frame supported column
- Moment increase for column base in base story of column

Building structure ductility factor μ is the ratio between the maximum deviation of the building structure due to the influence of the plan at the time of the earthquake reached the brink of collapse conditions and deviation δ_m building structure at the time of the first melting δ_y . At full elastic condition value $\mu = 1.0$ (Jamal *et al.*, 2014). Ductility level structure is influenced by the pattern of cracks or plastic hinge, wherein the plastic joints must be arranged to form at the ends of the beams and columns. Mathematically ductility is as a comparison parameter displacement structure collapsed during the displacement at the outer tensile reinforcement when experiencing fatigue (Department of Publics Works, 2002),(Department of Public Works, 2002).

According (Pauley *et al.*, 1992) ductility divided into:

1. Ductility strain is the ratio of maximum strain to strain on the beam melting experiencing tensile axial load or press;

$$\mu_{\epsilon} = \frac{\epsilon_u}{\epsilon_y} \quad (2.1)$$

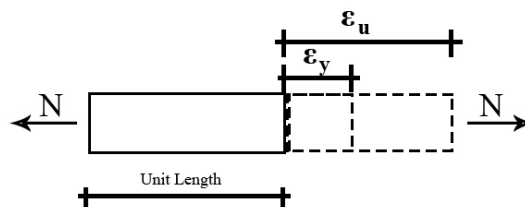


Figure 2.2 Ductility strain.

2. Ductility curvature μ_{θ} , is the ratio between the curvature angle (rotation angle per unit length) with a maximum angle of curvature of the melting of a structural element due to bending force;

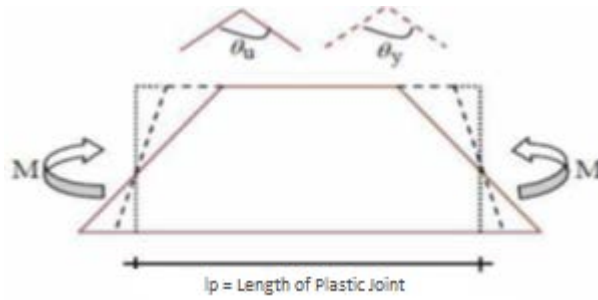


Figure 2.3 Ductility curvature.

3. Rotational ductility is the ratio between the maximum rotation angle of the rotation angle of melting;

$$\mu_\theta = \frac{\theta_u}{\theta_y} \quad (2.2)$$

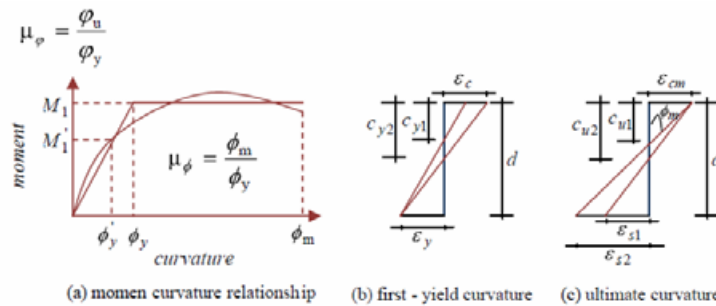


Figure 2.4 Ductility rotation.

4. Displacement ductility is the ratio between the maximum structural displacements in the lateral direction of the movement of the structure while melting.

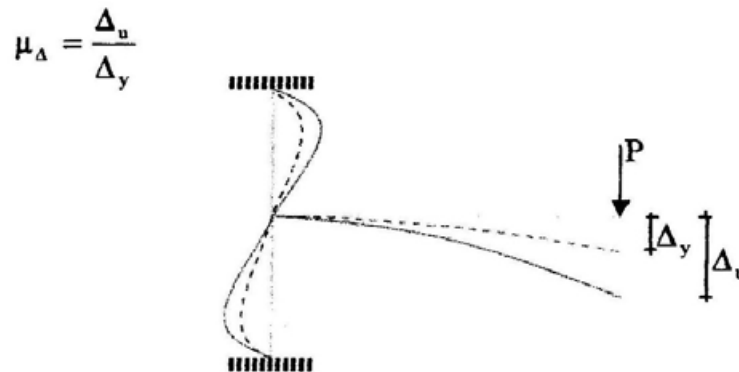


Figure 2.5 Ductility displacement.

The ductile response of the RC components in a frame system is one of the most important aspect in the seismic design. This response is achieved by careful attention to prevent brittle failure modes such as shear and anchorage in order for the member to behave inelastically under the seismic loading cycles. Each member has to provide ductility capacity more the expected ductility demand from the maximum design level earthquake.

The ductility demands for each component of the RC frames in the two buildings can be calculated from the curvature histories using the following formula (Banon, Irvine and Biggs, 1981)

$$\mu_{\varphi} = \frac{\varphi_y}{\varphi_m} \quad (2.3)$$

where “ φ_y ” is the yield curvature of the RC element that indicates the first yield of longitudinal reinforcing bars and “ φ_m ” is the maximum curvature along the element. The ductility demand of an RC member indicates certain levels of damage such as the yielding of the longitudinal bars as well as cracking and spalling of concrete.

2.3. Beam-Columns joints in RC Frame Buildings

The RC joint is defined as that portion of the column within the depth of the beam(s), including the slab, that frame into the column (Pinkham *et al.*, 1985).

Beam column joints in a reinforced concrete moment resisting frame are crucial zones for transfer of loads effectively between the connecting elements (i.e. beams and columns) in the structure. In normal design practice for gravity loads, the design check for joints is not critical and hence not warranted. But, the failure of reinforced concrete frames during many earthquakes has demonstrated heavy distress due to shear in the joints that culminated in the collapse of the structure. It is worth mentioning that the relevant research outcomes on beam column joints from different countries have led to conflicts in certain aspects of design. Coordinated programmes were conducted by researchers from various countries to identify these conflicting issues and resolve them (Park, R. and Hopkins, 1989)

Beam column joints are generally classified with respect to geometrical configuration and identified as interior, exterior and corner joints as shown in Figure 2.6 (Uma, 2015).

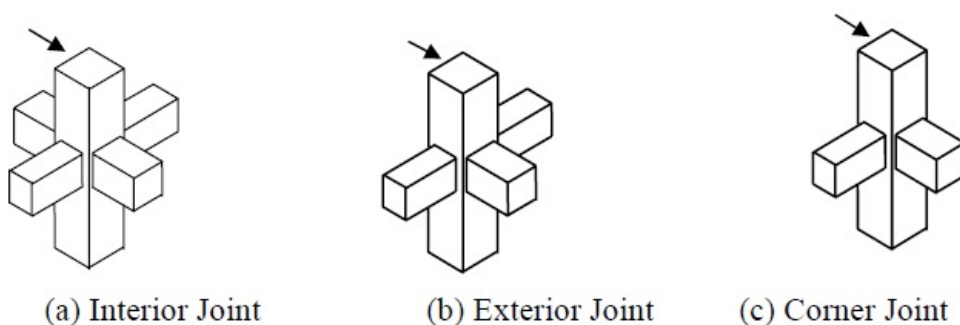


Figure 2.6 Types of Joints in a Moment Resisting Frame(Uma, 2015)

In RC moment resisting frame structures, the functional requirement of a joint, which is the zone of intersection of beams and columns, is to enable the adjoining members to develop and sustain their ultimate capacity. The demand on this finite size element is always severe and more complex due to the possible two-way actions in three-dimensional frame structures. However, the codes consider one direction of loading at a time and arrive at the design parameters for the joint (Uma, 2015).

Beam-column joints in moment resisting frames are usually subjected to large shear forces due to lateral earthquake forces. This is illustrated in Figure 2.7 for typical interior and exterior planar frame joints (Pak and Cheung, 1991). Reinforced concrete beam-column joints have limited energy dissipation characteristics and suffer from rapid strength degradation when they undergo inelastic deformations. As a result, inelastic deformations in ductile moment resisting frame structures designed for earthquake resistance must be located in regions other than the beam-column joints. Some design criteria for beam-column joints were suggested in New Zealand some years ago by (Paulay T. and Park R. and Priestley, 1978). They are described as follows:

1. The strength of a joint should not be less than the maximum strength of the weakest members it connects, to eliminate the need for repair in a relatively inaccessible region and to prevent the need for energy dissipation by mechanisms that undergo strength and stiffness degradation when subjected to cyclic loading in the inelastic range.
2. The capacity of a column should not be jeopardized by possible strength degradation within the joint.
3. During a moderate seismic disturbance, a joint should preferably respond within the elastic range. Joint deformations should not significantly affect stiffness of a building and hence inter-storey drift.
4. The joint reinforcement necessary to ensure satisfactory performance should not cause undue construction difficulties.

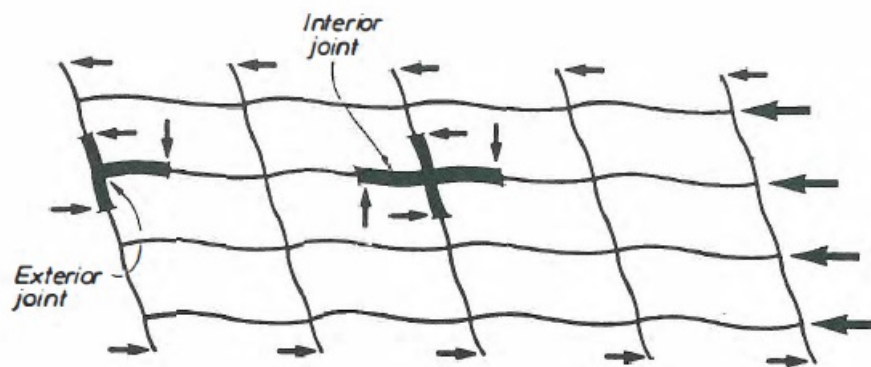


Figure 2.7 Exterior and Interior Beam-Column Joint Subassemblies of a Ductile Moment Resisting Frame Subjected to Lateral Loads. (Pak and Cheung, 1991)

Figure 2.8 below shows the deflected shape of an inelastic frame under earthquake attack;

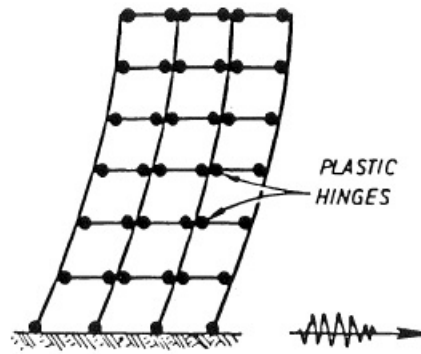


Figure 2.8 The deflected shape of an inelastic frame under earthquake attack.(Pak and Cheung, 1991)

2.4. Hazard Seismic Zones in Portugal

The Portuguese mainland and its adjacent Atlantic region are located on the western and southern margins of the Iberian Peninsula. The seismicity of this area is characterized by different regions with different seismic behaviors. The seismicity of the Portuguese territory increases in intensity from north to south, with a spatial distribution concentrated in the south and its adjacent Atlantic margins. Some seismological agencies have reported that the epicenters are concentrated near the city of Évora, in the Lisbon region, in the Lower Tagus Valley (LTV) region, and along the Algarve coast, especially in the southwest region in Cape of São Vicente and the Gorringe Bank (Ferrão *et al.*, 2016).

In southern Portugal, there are several tectonic structures: the Messejana fault, which crosses the entire southern region with a northeast–southwest orientation, and the Moura–Vidigueira and Loulé faults, which are most likely responsible for the large historical earthquakes in the Algarve region (Borges *et al.*, 2001).

The seismic hazards of the Portuguese mainland and adjacent Atlantic region are characterized by moderate-to-large onshore earthquakes and large-to-very large offshore earthquakes (Vilanova and Fonseca, 2007).

Investigation of the seismic hazards in this area continues to be critically important for the people of Portugal (Bezzeghoud, Borges and Caldeira, 2012)

The national seismographic network was created due to the 23 April 1909 (Mw 6.0), earthquake and it was further improved following an earthquake on 28 February 1969 (Ms 8.0). Until the mid- 1990s, however, the low quality of the instrumentation and the large geographic dispersion of the stations rendered the national seismographic network ineffective; it was able to locate only approximately 20% of the observed local and/or regional seismic events (Senos and Carrilho, 2003).

After several attempts to improve the coverage of the national seismographic network, 22 broadband stations were installed between 2006 and 2009. Since then, it has been possible to record all classes of earthquake magnitude that occur in Portugal (minor to major). Other organizations also have seismic equipment (e.g., the Universities of Lisbon and Évora), and this equipment provides high-quality data that contribute to the characterization of seismic

hazards(‘CSEM / EMSC Newsletter Recent improvements in the Broadband seismic networks in Portugal’, 2007).

In the computation of the hazard, the variability of the ground-motion model is taken into account by integrating over the aleatory uncertainty, leading to a mean value of hazard that is higher than the value obtained using the mean ground-motion model only (Bender and Perkins, 1993). Besides the aleatory uncertainty that results from the intrinsic variability of ground motion at a particular site, the incomplete knowledge about the appropriateness of the models used and the values of their parameters contributes to the overall uncertainty affecting a hazard estimate.

This aspect of hazard evaluation has been the object of intense debate in recent years. (Abrahamson and Bommer, 2005) point out that the weights assigned to each hazard value in the logic tree cannot be treated strictly as probabilities, because the models sampled by the combinations of branches do not span exhaustively the model space, nor are they mutually exclusive. As a result, (Abrahamson and Bommer, 2005) argue against the common practice of estimating mean hazard values by weighted averages of the logic tree individual branches. This procedure is also charged with being oversensitive to low-probability high-hazard end members. (McGuire, Cornell and Toro, 2005), on the contrary, taking an approach based on decision theory, favor the use of mean hazard values whenever a single-hazard curve or ground motion level has to be provided, as is the case for design (Vilanova and Fonseca, 2007).

Table 2.1 shows some of the strongest earthquakes in the period from 1300 to 2014. This table shows observations for some of the most significant earthquakes shown in Figure 2.9 (Ferrão *et al.*, 2016).

*Table 2. 1 Earthquakes in Portugal and its Adjacent Atlantic Margin between 1300 and 2014, with Maximum Observed Intensity $I_0 \geq 8$. (Ferrão *et al.*, 2016)*

Date (yyyy/mm/dd)	Longitude (°)	Latitude (°)	Locality	I_0	M^*	Observations
1309/02/22	-11.0	36.0	Epicenter offshore	VIII-IX	—	Felt in Portugal and Europe, without damage.
1344/01/01	-8.8	38.9	Benavente	VII- VIII	—	Destruction of Chapel of Lisbon and registration of deaths and falling buildings
1353/-/-	-8.1	37.3	Silves	VII- VIII	—	Much of the city of Silves (Algarve) was destroyed.
1356/08/24	-10.7	36.0	Epicenter offshore	VIII-IX	—	Felt in Spain (Seville and Córdoba); fall and ruins of chapel and buildings in Lisbon; duration of 2 times 15 min, followed by aftershocks; and damage similar to the 1531 and 1755 earthquakes.
1512/01/28	-9.2	38.7	Lisbon	VIII	—	Destruction of monastery and fall of 200 buildings in Lisbon; death of 2000 people (Lisbon).
1531/01/26	-9.0	38.9	Vila Franca de Xira	IX	—	Destruction of churches and about 1500 buildings in Lisbon; Tagus river agitated, overtaking the margins; high seismic activity; followed by aftershocks.
1587/11/-	-8.0	37.1	Loulé	VII- VIII	—	The village of Loulé (Algarve) was destroyed with about 170 deaths.

1722/12/27	-7.6	37.2	Epicenter offshore	VIII	—	Registrations fall of churches, monasteries, and buildings in Portimão; duration of 1 Ave Maria.
1755/11/01	-10.5	37.0	Epicenter offshore	X	M _w 8.5	Felt throughout Europe; registration of tsunami with destructive waves more extent in Portugal, Gulf of Cadiz (Spain), and northern Morocco.
1856/01/12	-8.0	37.1	Tavira	VII–VIII	—	Registration damage in temples and buildings in Algarve; Strong underground noises; followed by aftershocks.
1858/11/11	-9.0	38.2	Epicenter offshore	VIII	M _s 8.5	Foreshocks; registration of deaths and injuries; ruined buildings in Lisbon and Setúbal.
1909/04/23	-8.8	39.0	Benavente	X	M _w 6.0	Earthquake with violent shocks; destroyed several small towns; damage and effects in the whole country.
1910/09/11	-7.8	38.8	Redondo	VII–VIII	—	No information.
1964/03/15	-7.9	36.1	Epicenter offshore	VII	M _s 6.2	Mainshock followed by aftershocks and tsunami.
1969/02/28	-10.9	35.9	Epicenter offshore	VIII	M _s 8.0	Mainshock followed by aftershocks and small tsunami; record of 13 fatalities in Portugal; felt throughout the country, with higher intensity in Algarve (Vila do Bispo, Bensafrim, Portimão); felt in Bordeaux and the Canaries.
2007/02/12	-10.5	35.9	Epicenter offshore	V	M _L 5.9	Felt in Portugal, some cities in Spain and Morocco; without registration of casualties or damage.
2009/12/17	-9.9	36.5	Epicenter offshore	V	M _L 6.0	Mainshock followed by aftershocks; felt in Portugal, especially in Algarve region, with no damage registration; largest earthquake recorded since 1969.

Table 1 shows some of the strongest earthquakes in the period from 1300 to 2014. The magnitude is expressed using different scales: surface-wave magnitude (M_s), local magnitude (M_L), and moment magnitude (M_w). Latitude and longitude are also reported.

* A long dash indicates instrumental magnitudes are not available.

Concerning studies with a national coverage, Oliveira (Oliveira, C., Sousa, M., Costa, 1999) carried out the first comprehensive seismic hazard assessment for Portugal, whose results served as the foundation for the seismic hazard maps currently endorsed by the Portuguese design code (Ministério da Habitação e Obras Públicas, 1983). In the scope of the draft of the National Annex for the Eurocode 8 (EN 1998-1, 2004), several additional studies were also performed resulting in a seismic hazard map for moderate magnitude events at short distance, and large magnitude events at long distances (Oliveira, C., Sousa, M., Costa, 1999). In a study by Sousa (Sousa, 2006), a seismic risk assessment was carried out, using the seismic hazard model from (Sousa, 1996), and several simplified methodologies for the building vulnerability assessment and a detailed exposure model compiled based on data from the Building Census of 2001. Sousa (Sousa, 2006) calculated human and economic loss maps for several return periods, as well as loss maps for two deterministic events equivalent to the historical earthquakes of Lisbon in 1755 and Benavente in 1909. In (Vilanova and Fonseca, 2007), a seismic hazard model for Portugal is proposed using a logic tree approach to characterize the various epistemic uncertainties such as seismic sources characterization, selection of ground motion prediction equations, earthquake catalogues and methodologies to derive the magnitude-frequency relationship parameters. This effort resulted in the creation of a national hazard map for peak ground acceleration for a probability of exceedance of 10% in 50 years (Suckale and Grunthal, 2009).

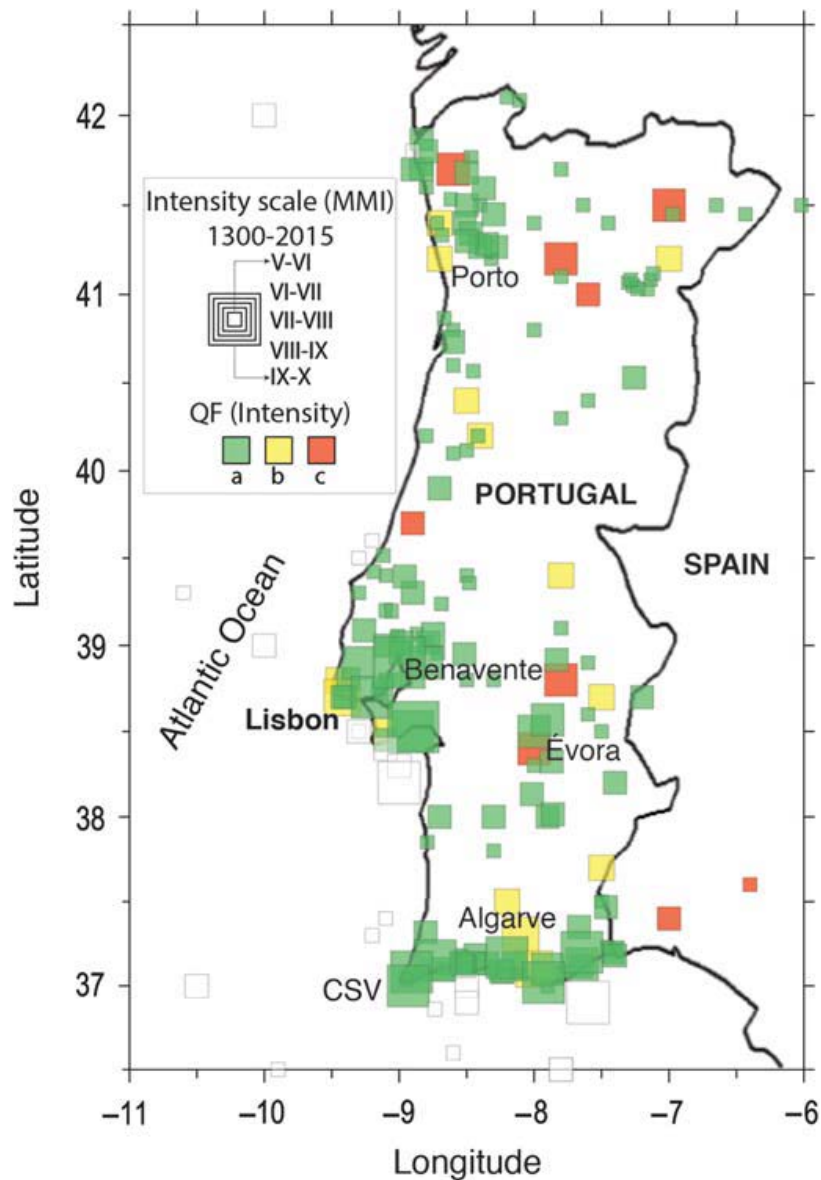


Figure 2. 9 Historical seismicity (1300–1960) and instrumental seismicity (1961–2014, IPMA database) with the associated quality factor (QF), using the modified Mercalli intensity (MMI) scale. (Ferrão et al., 2016)

In Eurocode 8 (EN 1998-1, 2004) the country seismic zonation is one of the Nationally Determined Parameters (NDP). Seismic zonation should be established for a reference peak ground acceleration on type A ground, a_{gR} , correspondent to the reference return period TNCR of seismic action for the no-collapse requirement, i.e. 475 years (Figure 1.3) (Campos Costa, Sousa and Carvalho, 2008). The initial proposal was discussed within Eurocode 8 working group, and incorporated some suggestions resulting from a public presentation for the technical and scientific Portuguese community. In accordance to those discussions, the initial proposal progressed to a five seismic zones territory division, for the long-distance scenario, and a three seismic zones division, for short distance scenario. Regarding the former scenario one intends to obtain a smoother acceleration transition between consecutive zones, and in the latter scenario some zones with approximate peak ground acceleration were eliminated (Carvalho, 2007).

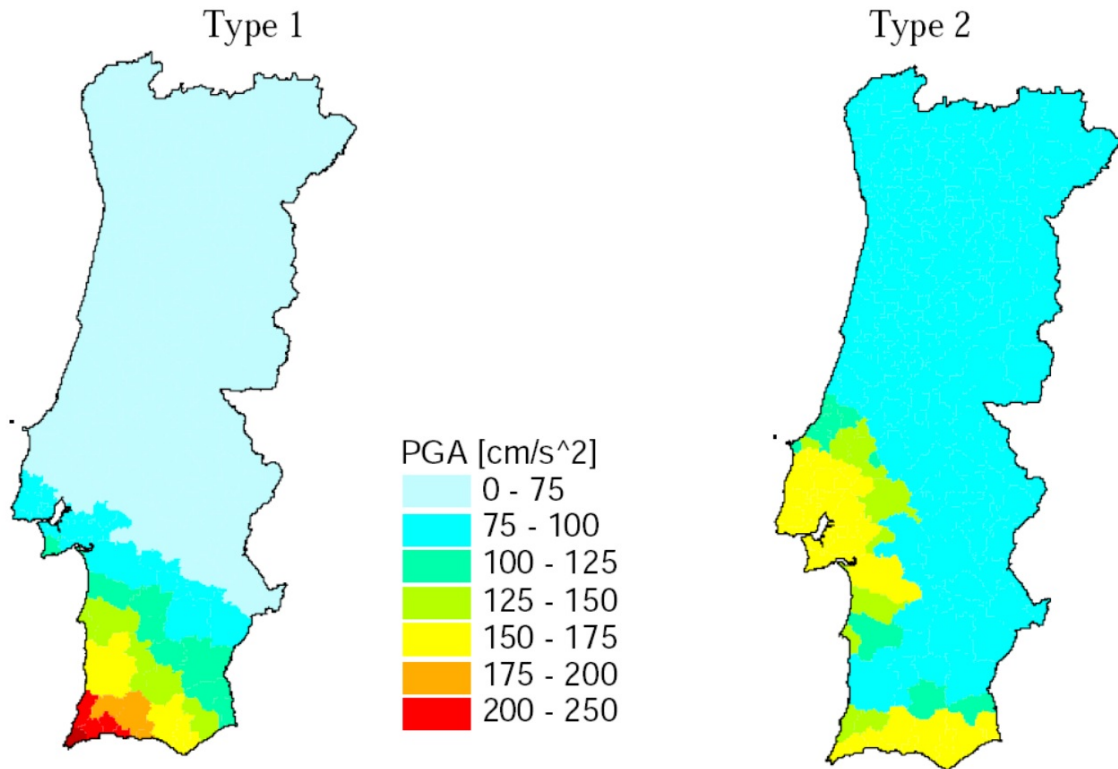


Figure 2.10 Mainland Portugal hazard maps for a exceedance probability of 10% in 50 years (Campos Costa, Sousa and Carvalho, 2008).

Table 2.2 presents the reference peak ground acceleration, a_{gR} for the considered seismic zones and for the two scenarios.

Table 2.2 Reference peak ground acceleration, a_{gR} (Campos Costa, Sousa and Carvalho, 2008).

Type 1 seismic action		Type 2 seismic action	
Seismic zone	a_{gR} [m/s^2]	Seismic zone	a_{gR} [m/s^2]
1.1	2.5	2.1	2.5
1.2	2.0	2.2	2.0
1.3	1.5	2.3	1.7
1.4	1.0	2.4	1.1
1.5	0.5	2.5	0.8

Figure 2.11 illustrates seismic zonation for the Portuguese National Annex of NP EN 1998-1 where, in what concerns Mainland Portugal, the two scenarios are distinguished. Seismic zonation was geographically disaggregated for each of the 278 Portuguese counties (Campos Costa, Sousa and Carvalho, 2008).

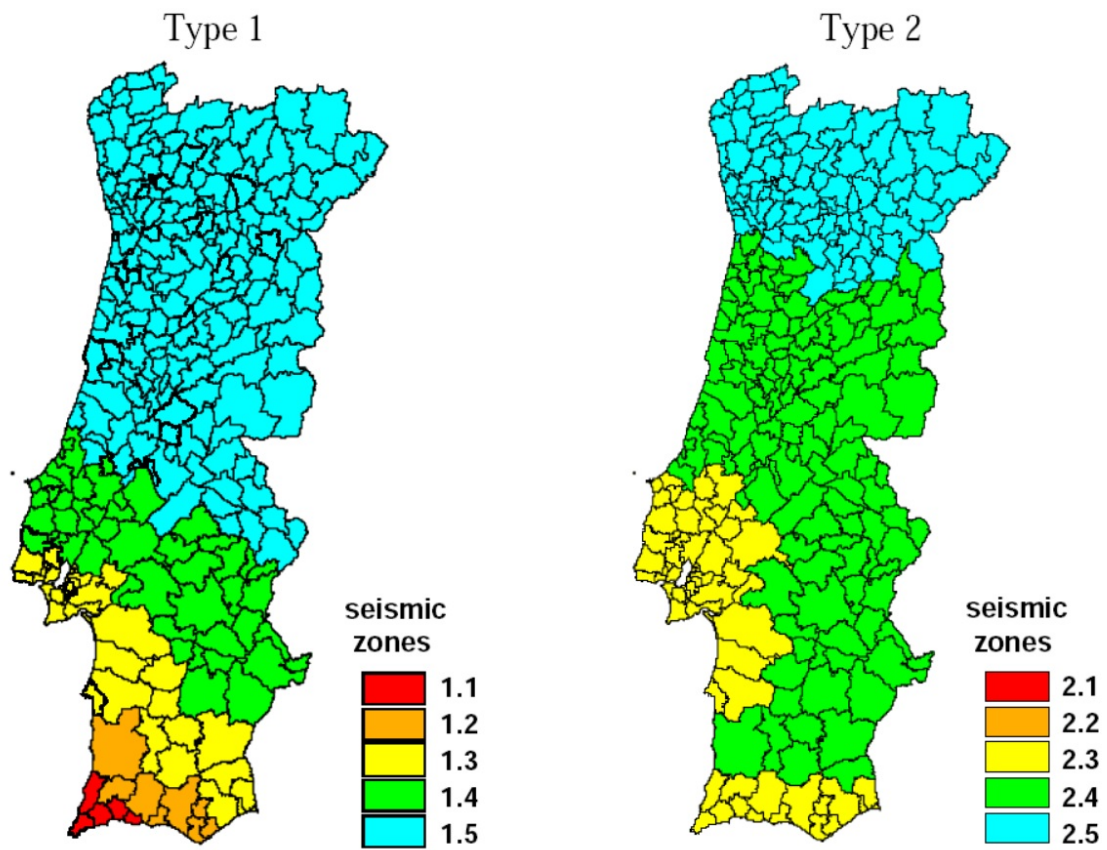


Figure 2.11 Mainland Portugal seismic zonation for Portuguese National Annex of NP EN 1998-1 (Campos Costa, Sousa and Carvalho, 2008)

3. Seismic Performance of Reinforced Concrete Frame Buildings

3.1. Reinforced Concrete Frame Buildings

Structural members in any structure must be designed to safely and economically support the self-weight of the structure and to resist all of the loads superimposed on the structure. In ordinary buildings, superimposed loads typically consist of live loads due to the inhabitants, dead loads due to items permanently attached to the building, and lateral loads due to wind or earthquakes. Some types of buildings must also be designed for extraordinary loads such as explosions or vehicular impact. A typical reinforced concrete building is made up of a variety of different reinforced concrete members. The members work together to support the applicable loads, which are transferred through load paths in the structure to the foundation members. The loads are ultimately supported by the soil or rock adjoining the foundations (Fanella, 2011).

As the building height increases, the importance of lateral forces due to wind and earthquake can be increased. Conventional load-bearing walls become unviable in such buildings, as the masonry wall thickness required in the lower storeys is large. The provision of thick masonry walls is not only expensive, it also reduces the flooring area in the rooms, and increases the mass, thereby attracting higher seismic forces. In such situations, frames (a skeleton of beams, columns and foundation) can be more apposite than load-bearing walls as the structural system. In very high seismic zones, it may be desirable to go for a framed construction even in single or two-storeyed buildings (Indian building Congress, 2007). Moment-resisting reinforced concrete frames are widely used as prime elements or in conjunction with structural (shear) walls for resisting seismic forces (Xu and Niu, 2003).

Many low-rise and medium-rise framed buildings have been constructed in the recent past, without proper attention paid in their design for wind or earthquake loads. This serious shortcoming in structural design and detailing has been exposed by the recent earthquakes in various parts of the country. And, there is now an increasing awareness about the need to design explicitly for seismic loads, in conformity with the prevailing codes of practices (Indian building Congress, 2007).

3.2. Seismic Design of RC Frame Buildings

3.2.1. Conceptual design of RC frames for earthquake resistance

The general layout and certain details of the geometry of an individual plane frame have a major impact on its seismic behaviour. Very important also is the overall layout of the frames in a frame structural system. Any single plane frame should run continuously from one side of the building plan to the other, without offsets, interruptions (i.e. missing beams between adjacent columns in a floor), or indirect supports of beams on other beams (Fardis *et al.*, 2015).

Foundations of the building have to be stronger than the columns and the columns need to be stronger than the beams to avoid the global collapse of the building. Usually, the main cause of failures is the failure of the columns, particularly in the lower storeys. Columns and beams have to be adequately reinforced and detailed to prevent shear or bending failure. Buildings with open ground storey are particularly unsafe, as the stresses in the ground floor columns are very severe. Columns in RC framed buildings may fail under an earthquake, either in shear or in bending. Shear failures occur mainly because the column sizes provided are inadequate to resist the seismic loads and also because of inadequate lateral ties provided. Bending (flexural) failures occur because of inadequate amount of steel bars provided vertically in the columns, particularly near the beam-column joints or column-foundation junctions, and may also occur due to poor quality of concrete (Indian building Congress, 2007).

According to (Fardis *et al.*, 2015) The ideal plane frame has:

1. Constant beam depth in all bays of a storey
2. Constant size of each column in all storeys
3. Approximately uniform spans
4. Interior columns of approximately the same size
5. Approximately the same height in all storeys

3.2.2. Design of RC frame building for code provisions

All modern national seismic design codes converge on the issue of design methodology. These are based on a prescriptive Force Based Design approach, where the design is performed using a linear elastic analysis, and inelastic energy dissipation is considered indirectly, through a response reduction factor (or behavior factor). This factor, along with other interrelated provisions, governs the seismic design forces and hence the seismic performance of code-designed buildings. However, different national codes vary significantly on account of various specifications which govern the design force level. The response reduction factor, as considered in the design codes, depends on the ductility and overstrength of the structure. Building codes define different ductility classes and specify corresponding response reduction factors based on the structural material, configuration and detailing. Another important issue, which governs the design and expected seismic performance of a building, is control of drift. Drift is recognized as an important control parameter by all the codes; however, they differ regarding the effective stiffness of RC members. Further, the procedures to estimate drift and the allowable limits on drift also vary considerably (Khose, 2012).

Different codes differ not only with respect to the design base shear but also employ different load and material factors (or strength reduction factors) for the design of members, and hence, the actually provided strength in different codes does not follow the same pattern as the design base shear. This has direct effect on the expected performance of buildings designed using different codes. Further, the other provisions of codes also indirectly govern the seismic performance. In the era of globalisation, there is a need for convergence of design methodologies to result in buildings with uniform risk of suffering a certain level of damage or

collapse. A first step in this direction is to compare the expected seismic performance of buildings designed using the provisions of different codes. A detailed comparison of various provisions of different codes and design base shear coefficients obtained for a given hazard were conducted by the authors earlier (Khose, Singh and Lang, 2012).

Khose (Khose, 2012) extends the comparison further and presents a comparative study of the expected seismic performance of a ductile RC frame building designed for four major codes, viz. the U.S. American (ASCE 7-10, 2010), European (EN 1998-1, 2004), New Zealand (NZS 1170.5, 2004) and Indian code (IS 1893 - Part 1, 2002). The provisions of different codes regarding effective stiffness of RC members, procedure to estimate drift, and allowable drift limits are also compared to each other.

3.2.3. Building Ductility Classification According to code provisions

Currently, all seismic design codes take into account the effect of inelastic energy dissipation by reducing the design seismic force by a ‘response reduction factor’ (also called ‘behavior factor’). Values of response reduction factor are provided for different ductility classes of buildings. In addition to ductility, the response reduction factor also takes into account the effect of overstrength. The New Zealand seismic code (NZS 1170.5, 2004) considers a separate structural performance factor in addition to the ductility factor, which represents the combined effect of the limited number of cycles having peak amplitude, overstrength, redundancy, and over-capacity due to damping in secondary components and in the foundation. Further, only the New Zealand seismic (NZS 1170.5, 2004) considers the effect of period on the relationship between ductility and the response reduction factor. All other codes provide constant response reduction factors for a particular construction type, irrespective of the period of vibration. The EUA code (ASCE 7-10, 2010) classifies RC frame buildings into three ductility classes: Ordinary Moment Resisting Frame (OMRF), Intermediate Moment Resisting Frames (IMRF) and Special Moment Resisting Frames (SMRF). The European code, Eurocode 8 (EN 1998-1, 2004) classifies the building ductility as Low (DCL), Medium (DCM) and High (DCH). NZS 1170.5 classifies structures into three ductility classes, namely Ductile Structures (DS), for which the structural ductility factor is greater than 1.25 but less than 6, Structures of Limited Ductility (SLD), which is a subset of DS with structural ductility factor between 1.25 and 3, and Nominal Ductile Structures (NDS), for which the ductility factor is between 1 and 1.25. (IS 1893 - Part 1, 2002) classifies RC frame buildings as Ordinary Moment Resisting Frames (OMRF) and Special Moment Resisting Frames (SMRF) (Khose, 2012).

Seismic design codes either provide guidelines for the design and detailing of RC buildings for different ductility classes or refer to complimentary design codes. These provisions, in general, consist of four requirements: (i) capacity design provisions to achieve a hierarchy of strength in order to avoid brittle failure modes, (ii) provision of special confining reinforcement (in the form of closely-spaced stirrups) at potential plastic hinge locations, (iii) anchorage of beam longitudinal reinforcement into columns, and (iv) design of beam-column joints to avoid shear failure. Table 3.1 summarizes the different provisions in the codes for different ductility classes of RC frame buildings (Khose, 2012).

Table 3.1 Overview of ductile detailing requirements for RC frame buildings in different seismic design codes (Khose, Singh and Lang, 2012)

Ductile Detailing Criteria		ASCE 7 ¹⁾			Eurocode 8			NZS 1170.5 ²⁾			IS 1893 ³⁾	
		OMRF	IMRF	SMRF	DCL	DCM	DCH	NDS	SLD	DS	OMRF	SMRF
Capacity Design	Strong Column Weak Beam	○	○	●	○	●	●	○	●	●	○	○
	Capacity Shear for Column	○	●	●	○	●	●	○	●	●	○	●
	Capacity Shear for Beam	○	●	●	○	●	●	○	●	●	○	●
Special Confinement Reinforcement	Column	○	●	●	○	●	●	○	●	●	○	●
	Beam	○	●	●	○	●	●	○	●	●	○	●
Special Anchorage Requirement	Interior Joint	○	○	●	○	●	●	●	●	●	○	●
	Exterior Joint	○	○	●	○	●	●	●	●	●	○	●
Joint Shear Design		○	○	●	○	○	●	●	●	●	○	○

¹⁾ Ductile detailing as per (ACI, 2008)

²⁾ Ductile detailing as per (NZS 3101:Part 1., 2006) and (NZS 1170.5, 2004)

³⁾ Ductile detailing of OMRF and SMRF as per (IS 465, 2000) and (IS 13920, 1993), respectively.

○ provision is not available

● provision is available

It is evident from Table 3.1 that it is not possible to have a one-to-one parity between different ductility classes of various codes. However, three broad categories of ductility can be considered, as shown in Table 3.2, where each category includes building classes with similar ductility provisions. Figure 3.1 shows the response reduction/behavior factors for different ductility classes of RC frames, according to different codes. There is a large difference in reduction factors for long-period and short-period structures according to the New Zealand Code (NZS 1170.5, 2004), whereas, as mentioned above, other codes do not consider the effect of period on response reduction factors. The reduction factors for medium and high ductility classes of (ASCE 7-10, 2010), NZS 1170.5 (for long-period structures) and the Indian Standard (IS 1893 - Part 1, 2002) (no high ductility class is available) are close, whereas the corresponding reduction factors in Eurocode 8 are quite low. For low ductility class, the response reduction factors of NZS 1170.5 and Eurocode 8 are close, whereas the response reduction factors of ASCE 7 and IS 1893 are identical and twice as high as those of Eurocode 8 (Khose, 2012). Figure 3.2 comparing between the reduction behaviour factors of the provision codes mentioned above, for the same ductility levels.

Table 3.2 Different ductility categories of RC frame buildings(Khose, Singh and Lang, 2012)

Category	Ductility classes			
	ASCE 7	Eurocode 8	NZS 1170.5	IS 1893
Low dissipative structures	OMRF	DCL	NDS	OMRF
Medium dissipative structures	IMRF	DCM	SLD	SMRF
High dissipative structures	SMRF	DCH	DS	-

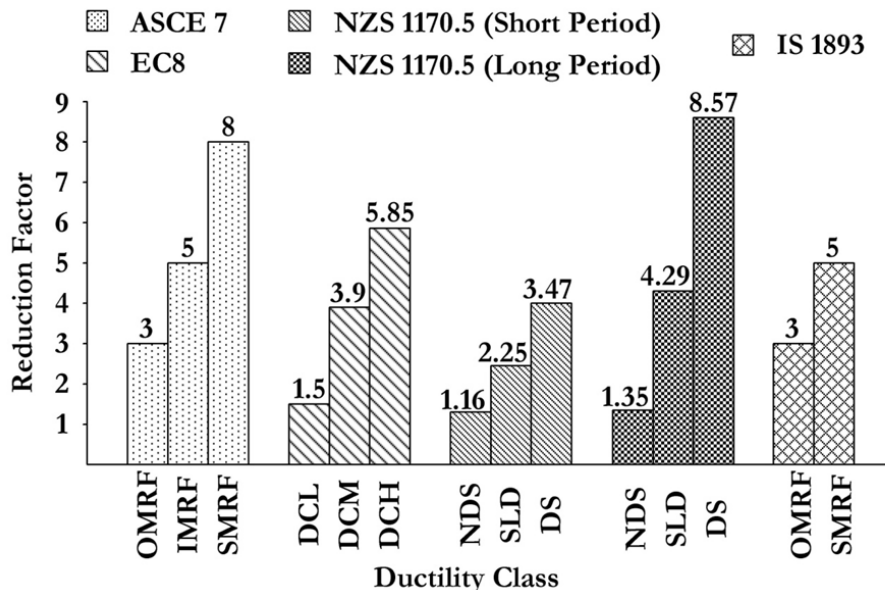


Figure 3.1 Comparison of reduction/behavior factors recommended in different national codes (Khose, Singh and Lang, 2012).

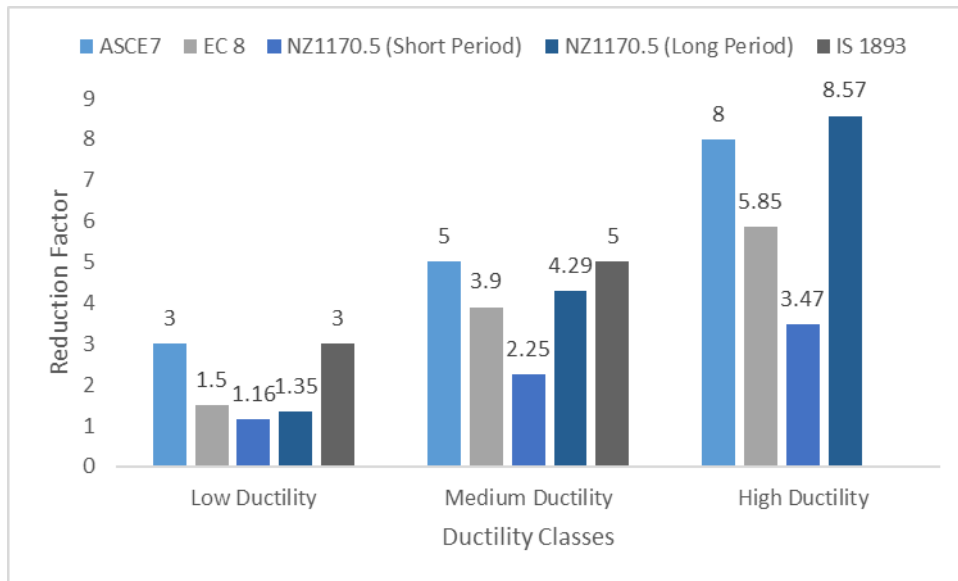


Figure 3.2 Comparison of ductility classes and their behaviour factors.

3.2.4. Seismic Design for Ductility according to EC 8

The EN Eurocodes have certain built-in flexibility on key items controlling the level of safety, serviceability and durability offered by structures designed according to them, taking into account economy. These aspects are considered to lie within national authority. To facilitate national choice on these aspects without sacrificing harmonisation of structural design codes at the European level, as well as to accommodate geographic, climatic, etc., differences (including differences in the seismotectonic environment), the system of “Nationally Determined Parameters” (NDPs) has been devised and adopted in the EN Eurocodes. NDPs include symbols (e.g., safety factors, the mean return period of the design seismic action, etc.), technical classes (e.g., ductility classes), or even procedures or methods (e.g., alternative models of calculation). Alternative classes or procedures/ methods considered as NDPs are fully described in the normative text of the EN Eurocode. For NDP-symbols, the EN Eurocode may give an acceptable range of values and will normally recommend in a non-normative note a value to be used. It may also recommend a class or a procedure/method, among the alternatives identified in the EN Eurocode text as NDPs (Panagiotakos and Fardis, 2004).

Eurocode 8, allows three alternative ductility classes for RC structures, Ductility Class Low (DC L), Ductility Class Medium (DC M), and Ductility Class High (DC H), for each ductility class there are many design and detailing aspects such as; the size of columns and beams, and detailing and dimensioning rules for the longitudinal and transverse reinforcement, and also one of the important aspect is the mechanism of the joints between the columns and beams under the seismic forces.

DC L is the ductility class with the behavior factor value $q = 1.5$ instead of $q = 1.0$, Buildings of DC L are not designed for ductility; only for strength. Except certain minimum conditions for the ductility of reinforcing steel, they have to follow just the dimensioning and detailing rules specified in Eurocode 2 for non-seismic actions.

Buildings of DC M or H have q -factor values higher than the default value of 1.5 used for DC L and considered as due to overstrength alone. DC H buildings enjoy higher values of q than DC M ones; in return, they are subject to stricter detailing rules and have higher safety margins in capacity design against shear. However, unlike DC L, DC M does not systematically require more steel than DC H: the total quantities of materials are essentially the same; in DC H, transverse reinforcement and vertical members have a larger share of the total quantity of steel than in DC M. DC M and H are expected to achieve about the same performance under the design seismic action, but DC M is slightly easier to design and implement and may give better performance in moderate earthquakes. DC H may provide larger safety margins than M against collapse under earthquakes (much) stronger than the design seismic action and may be more economic for high seismicity (Fardis *et al.*, 2015).

In Eurocode 8 (EC8), the value of the behaviour factor, q , of DC M and H buildings depends on:

- Ductility Class
- The type of lateral-force-resisting-system
- The regularity or lack thereof of the structural system in elevation

The value of the q -factor is linked, indirectly (through the ductility classification) or directly, to the local ductility and detailing requirements for members.

According to Eurocode 8, Basic value, q_o , of behavior factor per EC8 for height-wise regular frame moment-resistant buildings is

Table 3.3 Basic value, q_o , of behavior factor per EC8 for height-wise regular buildings

Lateral-load-resisting structural system:		DC M	DC H
1	Inverted pendulum	1.5	3
2	Torsionally flexible	2	3
3	Uncoupled wall system, not in one of the two categories above	3	$4\alpha_u / \alpha_l$
4	Any structural system other than the above	$3\alpha_u / \alpha_l$	$4.5\alpha_u / \alpha_l$

Where $\alpha_u/\alpha_l = 1.3$ for multi-storey multi-bay frames or frame-equivalent dual systems.

3.2.5. Pros and Cons of RC frames for earthquake resistance

Fardis (Fardis *et al.*, 2015) mention that there are different advantages and disadvantages of using Rc frame building for earthquake resistance,

The advantages of using RC frames for earthquake resistance may be summarised as follows:

1. Frames place few constraints on a building's architectural design, including the façade.
2. Frames may be cost-effective for earthquake resistance, because beams and columns are placed anyway for gravity loads; so, they may also provide earthquake resistance in both horizontal directions, if their columns are large.
3. Two-way frame systems, comprising several multi-bay plane frames per horizontal direction, are highly redundant, offering multiple load paths.
4. Due to their geometry (notably their slenderness), beams and columns are inherently ductile, less prone to (brittle) shear failure than walls.
5. Frames with concentric connections and regular geometry have well-known and understood seismic performance, thanks to the numerous experimental and analytical studies carried out in the past; moreover, they are rather easy to model and analyse for design purposes.

6. It is easier to design an earthquake resistant foundation element for a smaller vertical member than for a larger one (i.e. for a column in comparison to a wall).

There are also disadvantages of RC frames for earthquake resistance maybe summarised as:

1. Frames are inherently flexible; the cross-section of their members may be governed by the inter-storey drift limitation under the moderate earthquake for which limitation of damage to structural and non-structural elements is desired.
2. Column counter-flexure in the same storey allows soft-storey mechanisms.
3. Earthquake-resistance requirements on frames lead to large columns.
4. The reinforcement detailing of frames for ductility requires workmanship of high level for its execution and good supervision on site (specially to prepare the dense reinforcement and place/compact the concrete through the beam column joints in two-way frames).
5. Sizing and detailing of beam–column joints for bond and anchorage of beam bars crossing them is quite challenging. Difficulties increase with the use of higher strength materials, as the size of joints made of higher concrete strength is smaller, while higher steel strength implies higher bond stresses.

3.3. Sizing of RC Frame Members according to EC8

3.3.1. Sizing of Beams

To facilitate continuity of the beam top and bottom bars across a column, the cross-section of beams should be the same in all bays of a plane frame.

Beam depth is often controlled by gravity loads it is normally chosen around one-tenth of the average bay length in the frame.

The web should be sufficiently wide:

1. To avoid undue congestion of longitudinal bars (preferably placed in one layer),
2. To provide at least the minimum concrete cover of stirrups at the sides of the beam per (Eurocode 2 Part 1-1, 2004), and
3. To provide at least the minimum mean axial distance of longitudinal bars to the concrete surface per (Eurocode 2 Part1-2, 2004).

3.3.2. Sizing of Columns

Eurocode 8 sets a minimum length of 200 or 250 mm, for a side of a DC M or DC H column, respectively. The size of a column should be kept constant in all storeys, as determined from the most critical one, except for serious architectural reasons.

The ideal connection of a beam with a column is concentric, with the column being wider than the web of the beam on each side by at least 50 mm, to allow the beam longitudinal bars to pass through the confined core of the column section between its outermost bars.

The most cost-effective option, which also serves the requirement to have a clear structural system, is to have as uniform a size of columns in the building as feasible: experience from past earthquakes shows that larger columns in the system are more likely to fail than the smaller ones, even when they have higher vertical steel ratio.

To ensure a minimum flexural ductility of the column, Eurocode 8 sets upper limits on its axial load ratio:

- For DC M: $v_d \leq 0.65$
- For DC H: $v_d \leq 0.55$

Where $v_d = N_d / (A_c f_{cd})$, with N_d denoting the column axial load in the seismic design situation and A_c the column cross-sectional area.

Eurocode 8 sets a very restrictive, albeit fully warranted, lower limit to the column depth, h_c , parallel to a beam framing into the column, to accommodate the very high bond stresses along the length of a beam bar inside an interior beam–column joint, or the anchorage of beam bars terminating in a joint, either exterior or not. If the bar diameter is d_{bl} , then the limit is:

1. In an interior beam–column joint:

$$\frac{d_{bl}}{h_c} \leq \frac{7.5 f_{ctm}}{\gamma_{Rd} * f_{yd}} * \frac{1+0.8V_d}{1+K*\frac{\rho_2}{\rho_{1,max}}} \quad (3.1a)$$

2. In a beam–column joint that is exterior in the direction of the beam:

$$\frac{d_{bl}}{h_c} \leq \frac{7.5 f_{ctm}}{\gamma_{Rd} * f_{yd}} * 1 + 0.8V_d \quad (3.1b)$$

For DC M, $\gamma_{Rd} = 1.0$, $k = 0.5$.

For DC H, $\gamma_{Rd} = 1.2$, $k = 0.75$.

$\rho_{1,max}$ is the maximum value of beam top reinforcement ratio and ρ_2 is the bottom steel ratio (taken as $\rho_2 = 0.5 \rho_{1,max}$, if the beam reinforcement is not known yet and only the combination of its maximum allowed diameter and the column depth is being sought).

3.4. Detailed Design of RC Frame members in Flexure

3.4.1. Detailed Design of Beams in Flexure

3.4.1.1. Dimensioning of the beam longitudinal reinforcement for the ULS in flexure

The top and bottom bars at the two ends of each beam are dimensioned for the ULS in flexure with no axial force for the envelope of bending moments resulting from the analysis under:

- a. The combination of factored gravity loads ('persistent and transient design situation' per EN1990), and
- b. The combination of quasi-permanent gravity loads, $G + \psi_2 Q$, with plus and minus the design seismic action.

The cross-sectional area of the top reinforcement, A_{s1} , of each end region is dimensioned as the tension reinforcement required for an acting moment, M_{Ed} , equal to the maximum hogging moment at the column face; normally in this dimensioning, combination (b), with the beam seismic moments taken as hogging, controls over (a). The cross-sectional area of the bottom reinforcement, A_{s2} , is dimensioned as the tension reinforcement for an acting moment, M_{Ed} , equal to that at the column face, or at a nearby section where the sagging moment attains its maximum; in this case M_{Ed} is obtained from combination (b), but with the beam seismic moment taken as sagging. Besides, the main bottom bars of the beam are dimensioned from a section around mid-span, normally where the sagging moment from combination (a) attains its maximum value within the span.

The cross-sectional area, A_s , of the tension reinforcement may be conveniently dimensioned from the extreme value of the pertinent acting moment M_{Ed} (i.e. the extreme sagging moment for the bottom reinforcement or the extreme hogging one for the top bars), by taking the internal lever arm of the beam (between its tension and compression chords), z , as equal to the distance between the tension and compression bars, $d - d_2$, where d is the effective depth of the section and d_2 is the distance of the centroid of the compression bars from the extreme compression fibres:

$$A_s = M_{Ed} (f_{yd} (d - d_2)) \quad (3.2)$$

Where f_{yd} is the design yield stress of steel. Note that the absolute value of M_{Ed} is used; its sign determines the side of the section (top or bottom) where the tension fibers are and the tension reinforcement area, A_s , is placed.

All the above apply to beams in pure bending, without axial load. As a matter of fact, the values of beam axial forces which may come out of the analysis depend heavily on the modelling of the floor diaphragms and/or the way the external lateral loads are applied to the floors. So, normally they are fictitious and would better be neglected in dimensioning the beams.

3.4.1.2. Detailing of the beam longitudinal reinforcement for the ULS in flexure

In translating A_{s1} and A_{s2} into a combination of bar diameters and numbers, the designer should respect the detailing rules of Eurocode 8 summarized in Table 3.4. The rule at the fourth row concerning the maximum ratio of tension reinforcement, ρ_{max} , is the only one of these rules which is not prescriptive; at the same time it is the most restrictive: as the value of A_{s1} to be accommodated within a given beam width, b , cannot be reduced below what is necessary to resist the acting moment, M_{Ed} , the best way to meet the rule for ρ_{max} is by increasing the ratio of compression reinforcement, ρ' , at the end section.

The bar diameters chosen should also respect the maximum allowed by Equations 3.1 for a given section depth, h_c , of the column where these bars are anchored (at exterior columns) or pass through (at interior ones).

The bars chosen on the basis of the two end sections and the one around mid-span where the span bottom bars are determined, are terminated according to the positive and negative moment envelopes; they extend beyond the point where they are not needed according to the envelope by the 'tension shift' length $z = 0.9d - d_2$ in Eurocode 2.

The full string of beams ('continuous beam') in a frame should be designed in bending all together, combining reinforcement requirements to the right and left of interior joints. It is also recommended to combine different top or bottom bars into continuous ones, if their ends come close or overlap. To this end, few bar sizes (even a single size) should be used all along each string of beams.

Table 3.4 Eurocode 8 detailing of the longitudinal bars in primary beams (Fardis et al., 2015)

	DC H	DC M	DC L
'critical region' length at member end	1.5h	h	
$\rho_{min} = A_{s,min}/bd$ at the tension side	0.5 f_{ctm}/f_{yk}^a		0.26 f_{ctm}/f_{yk}^a , 0.13% ^b
$\rho_{max} = A_{s,max}/bd$ in critical regions ^b	$\rho' + 0.0018f_{cd}/(\mu_\phi \epsilon_{yd} f_{yd})^c$		0.04
$A_{s,min}$, top and bottom bars	2 Φ 14 (308 mm ²)	-	-
$A_{s,min}$, top bars in the span	0.25 $A_{s,top-supports}$	-	-
$A_{s,min}$, bottom bars in critical regions	0.5 $A_{s,top}^d$		-
$A_{s,min}$, bottom bars at supports	0.25 $A_{s,bottom-span}^b$		
Anchorage length for diameter d_{bL}	$l_{bd} = a_{tr}[1 - 0.15(c_d/d_{bL} - 1)](d_{bL}/4)f_{yd}/(2.25f_{ctd}a_{poor})^{f,g,h,i}$		

^a f_{ctm} (MPa) = $0.3(f_{ck}(\text{MPa}))^{2/3}$: 28-day, mean tensile strength of concrete; f_{yk} (MPa): nominal yield stress of longitudinal steel.

^b NDP (nationally determined parameter) per EC2; the value recommended in EC2 is given here.

^c ρ' : Steel ratio at the opposite side of the section; μ_ϕ : curvature ductility factor corresponding to the basic value of the behavior factor, q_o , applicable to the design; $\epsilon_{yd} = f_{yd}/E_s$.

^d This $A_{s,min}$ is additional to the compression steel from the ULS verification of the end section in flexure under the extreme hogging moment from the analysis for the seismic design situation.

^e Anchorage length in tension is reduced by 30% if the bar end extends by $\geq 5d_{bL}$ beyond a bend $\geq 90^\circ$.

^f c_d : Concrete cover of anchored bar, or one-half the clear spacing to the nearest parallel anchored bar, whichever is smaller.

$\xi_{tr} = 1 - k(n_w A_{sw} - A_{s,t,min})/A_s \geq 0.7$, with A_{sw} : Cross-sectional area of tie leg within the cover of the anchored bar; n_w : number of such tie legs over the length l_{bd} ; $k = 0.1$ if the bar is at a corner of a hoop or tie, $k = 0.05$ otherwise; $A_s = \pi d_b l^2 / 4$ and $A_{s,t,min}$ is specified in EC2 as equal to $0.25A_s$.

3.4.2. Detailed Design of Columns in Flexure

3.4.2.1. Strong Column – Weak Beam Capacity Design

To pursue the desired global ductility, Eurocode 8 promotes beam-sway mechanisms and takes measures to prevent a soft storey. A soft-storey mechanism develops in a frame system when the top and bottom ends of (all) the columns in a storey yield in opposite bending and start undergoing unrestrained flexural rotations there, without a notable increase of their bending moments beyond the corresponding moment resistance, M_{RC} (this is, in fact, how a flexural ‘plastic hinge’ is defined). The way to prevent soft storeys in frames is by forcing flexural plastic hinges out of the columns and into the beams, so that a beam-sway mechanism develops. To this end, within any vertical plane in which a soft storey is to be prevented, the two columns framing into a beam–column joint from above and below are dimensioned to be jointly stronger by 30% than the (one, two or more) beams connected to the same joint from any side:

$$\sum M_{Rd,c} \geq \sum 1.3 M_{Rd,c} \quad (3.3)$$

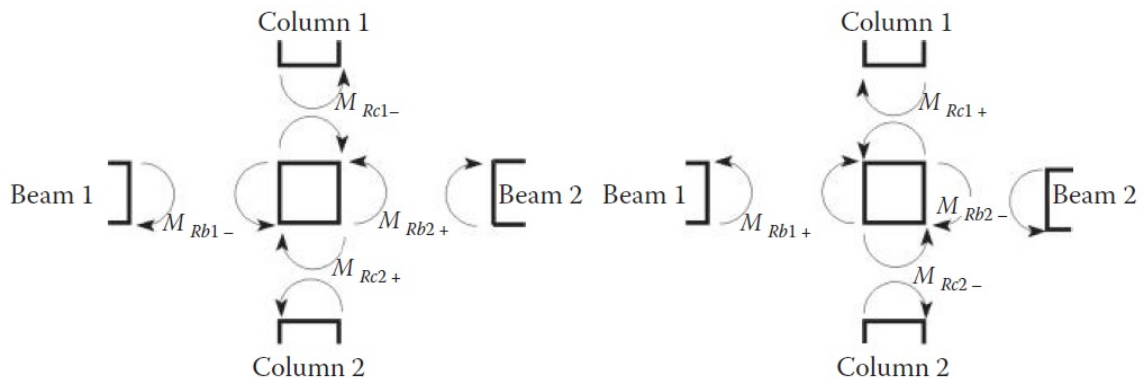


Figure 3.3 Direction of action of column and beam moment resistances around a joint in the capacity design check of the column for both directions of the response to the seismic action.

Where:

- $M_{Rd,c}$: design value of column moment resistance at the face of the joint, in the vertical plane of bending in which a soft storey is to be prevented (i.e. with moment vector at right angles to that plane), with the sum referring to the column sections above and below the connection.
- $M_{Rd,b}$: design value of beam moment resistance at the face of the joint, with the sum extending to all beam ends connected to the joint; beams that are not in the vertical plane in which a soft storey is to be prevented but at an angle α to it, enter Equation 2.3 with their $M_{Rd,b}^-$ value multiplied by $\cos \alpha$.

3.4.2.2. Dimensioning of Column longitudinal Reinforcement

The base section at the bottom storey of a column (the connection to the foundation), as well as all columns exempted from the capacity design rule, Equation 2.3, are dimensioned for the ULS in biaxial flexure with axial force, using triplets M_y – M_z – N from the analyses for the combination of the design seismic action with the quasi-permanent gravity loads, $G + \psi_2 Q$.

The column sections right above and right below a beam–column joint are served by the same vertical bars. Besides, as pointed out in Section 2.3.2, it is good practice to avoid changing the column section from one storey to the next. So, these two sections are dimensioned as a single one, for all M_y – M_z – N triplets that the analysis gives for them in the seismic design situation, each triplet being the single triplet due to the quasi-permanent gravity loads $G + \psi_2 Q$ plus a seismic one. Most critical of all triplets is the one giving the largest amount of reinforcement in one of the two sections; however, it is not easy to screen out non-critical ones. Generally, for the usual range of values of the dimensionless axial load, $N_d/A_c f_{cd}$, most critical among triplets with similar biaxial moments is the one having the lowest axial compression.

There are several iterative algorithms for the ULS verification of sections with any shape and amount and layout of reinforcement for a combination M_y – M_z – N . They employ section analysis and the σ – ε laws used for design (elastic perfectly plastic for steel, normally parabolic–rectangular for concrete) to find the strain distribution which satisfies equilibrium. It is checked then whether, in that strain distribution, the conventional ultimate strain of concrete, $\varepsilon_{cu,2}$, is exceeded at the corners of the section. However, there is no general algorithm for the direct calculation of the section reinforcement for a given M_y – M_z – N triplet. The traditional manual approach with design charts is not practical for the large number of columns of a real building; it is also very restrictive for the bar layout and the steel grade. A practical, yet approximate, step-by-step computational procedure is proposed in the following paragraphs for the direct dimensioning of symmetrically reinforced rectangular sections under a set of M_y – M_z – N triplets.

1. The mechanical reinforcement ratio, ω_{1d} of the steel bars placed along each one of two opposite sides of the section of length b , is estimated under uniaxial moment, M , with axial force, N , neglecting the orthogonal moment component; d is the effective depth at right angles to the vector of M ; each layer of bars with cross-sectional area A_{s1} is at centroidal distance d_1 from the nearest side of the section of length b . M , N and d_1 are normalised as :

$$\omega_{1d} = A_{s1}/(bd) \cdot (f_{yd}/f_{cd}) \quad (3.4a)$$

$$\omega_{2d} = A_{s2}/(bd) \cdot (f_{yd}/f_{cd}) \quad (3.4b)$$

$$\mu_d = \frac{M}{b \cdot d^2 \cdot f_{cd}} \quad (3.5a)$$

$$V_d = \frac{N_{Ed}}{b \cdot d \cdot f_{cd}} \quad (3.5b)$$

$$\delta_1 = \frac{d_1}{d} \quad (3.5c)$$

Section analysis is used, with the material σ - ε laws and criteria adopted in Eurocode 2 for the ULS design:

- a. Elastic–perfectly plastic steel, with a yield stress of f_{yd} and unlimited strain capacity
- b. Parabolic–rectangular σ - ε law for concrete, with design strength f_{cd} at strain ε_{c2} , with ultimate strain ε_{cu2} (for $f_{cd} \leq 50$ MPa, $\varepsilon_{c2} = 0.002$, $\varepsilon_{cu2} = 0.0035$).

Depending on the value of the dimensionless axial load, V_d , there are three possible cases:

The most usual case is to have yielding of the tension and the compression reinforcement; this happens if:

$$\delta_1 * \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} - \varepsilon_{yd}} \equiv V_2 \leq V_d < V_1 \equiv \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} + \varepsilon_{yd}} \quad (3.6)$$

Where $\varepsilon_{yd} = f_{yd}/E_s$. Then, the neutral axis depth, x , normalised to d as $\xi = x/d$, is to be substituted in terms of v_d in the following equation, to be solved directly for ω_{1d} :

$$\xi = \frac{x}{d} = \frac{V_d}{1 - (\varepsilon_{c2}/3\varepsilon_{cu2})} \quad (3.7)$$

$$(1 - \delta_1) * \omega_{1d} = \mu_d - \xi * \left[\frac{1-\xi}{2} - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}} \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4\varepsilon_{cu2}} * \xi \right) \right] \quad (3.8)$$

2. The procedure in step 1 above is applied first with all M_y - N pairs in the set of M_y - M_z - N combinations, with b the side length parallel to the vector of M_y and dimensions d , d_1 at right angles to it. The most critical pair gives the total area of reinforcement, A_{sy} , along each side parallel to the M_y -vector. This is repeated with all M_z - N pairs and the roles reversed, to find the total area of reinforcement, A_{sz} , along each one of the two other sides – those parallel to the M_z -vector. As the M_y - N pair from which A_{sy} is derived most likely does not belong in the same M_y - M_z - N combination as the pair M_z - N giving A_{sz} , these reinforcement requirements are superimposed on the section and translated into a bar layout meeting the Eurocode 8 detailing rules in Table 3.5 for column vertical bars, with the corner ones counting to both sides (Figure 3.3).

Table 3.5 EC8 detailing rules for vertical bars in primary columns

	DC H	DC M	DC L
$\rho_{min} = A_{s,min}/A_c$	1%		$0.1N_d/A_c f_{yd}$, 0.2% ^a
$\rho_{max} = A_{s,max}/A_c$	4%		4% ^a
Diameter, d_{bL}	≥ 8 mm		
Number of bars per side	≥ 3		≥ 2
Spacing along the perimeter of bars restrained by a tie corner or hook	≤ 150 mm	≤ 200 mm	-

Distance along perimeter of unrestrained bar to nearest restrained one	$\leq 150 \text{ mm}$
Lap splice length ^b	$l_0 = 1.5[1 - 0.15(c_d/d_{bL} - 1)]a_{tr}(d_{bL}/4)f_{yd}/(2.25f_{ctd})^{c,d,e}$

^a NDP (nationally determined parameter) per EC2; the value recommended in EC2 is given here.

^b Anchorage length in tension is reduced by 30% if the bar end extends by $\geq 5d_{bL}$ beyond a bend $\geq 90^\circ$.

^c c_d : Minimum of: concrete cover of lapped bar and 50% of clear spacing to adjacent lap splice.

^d $a_{tr} = 1 - k(2n_w A_{sw} - A_{s,t,min})/A_s$, with $k = 0.1$ if the bar is at a corner of a hoop or tie, $k = 0.05$ otherwise; A_{sw} : cross-sectional area of a column tie; n_w : number of ties in the cover of the lapped bar over the outer third of the length l_0 ; $A_s = \pi d_{bL}^2/4$ and $A_{s,t,min}$ is specified in EC2 as equal to A_s .

^e $f_{ctd} = f_{ctk,0.05}/\gamma_c = 0.7f_{ctm}/\gamma_c = 0.21f_{ck}^{2/3}/\gamma_c$: design value of 5%-fractile tensile strength of concrete.

3. If available, an iterative algorithm may be used in the end to verify that the section with the selected layout of reinforcement satisfies the ultimate strain of concrete, ε_{cu2} , under any one of the M_y - M_z - N triplets. If it does not, one bar may be added to each side, till the section meets the verification criteria.

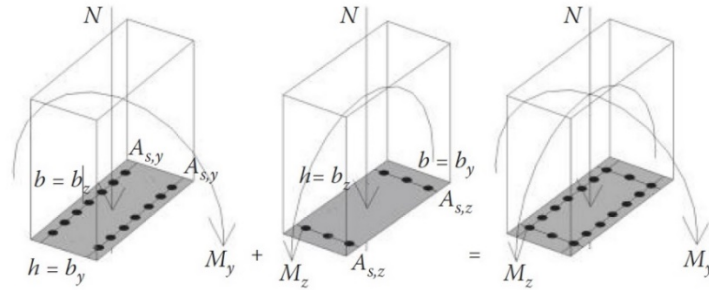


Figure 3.4 Dimensioning of reinforcement in column section for biaxial moments with axial force.

3.4.2.3. Calculation of the column moment resistance for given reinforcement and axial load

This section is about the design values of the ULS moment resistance of a column, $M_{Rd,c}$, to be used in Equation 3.3 and in the other ‘capacity design’ calculations of Section 3.5. In rectangular sections $M_{Rd,c}$ refers to centroidal axes parallel to the sides. The assumptions made in the previous section apply. The same for the notation introduced there, including Equations 3.5.

- The mechanical reinforcement ratio ω_{1d} refers to the tension flange only; for generality, the compression flange may have different reinforcement, A_{s2} , with mechanical ratio ω_{2d} ; its centroidal distance from the extreme compression fibres is still d_1 .
- There are intermediate bars between the tension and compression reinforcement, uniformly distributed along the length $(h - 2d_1)$ of the cross sectional depth h ; their total cross-sectional area, A_{sv} , is taken smeared along that length, with mechanical reinforcement ratio:

$$\omega_{2d} = A_{s2}/(bd) \cdot (f_{yd}/f_{cd}) \quad (3.4c)$$

Only one-half of each corner bar is included in A_{s1} or A_{s2} ; the other half counts as part of A_{sv} .

The most usual case is to have Tension and compression reinforcement yield, if the normalised axial load is in the range:

$$\omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1-\delta_1} * \left(\delta_1 * \frac{\varepsilon_{cu2} + \varepsilon_{yd}}{\varepsilon_{cu2} - \varepsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\varepsilon_{cu2} + \frac{\varepsilon_{c2}}{3}}{\varepsilon_{cu2} - \varepsilon_{yd}} \right) \equiv V_2 \leq V_d < V_1 \equiv \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1-\delta_1} * \left(\frac{\varepsilon_{cu2} - \varepsilon_{yd}}{\varepsilon_{cu2} + \varepsilon_{yd}} - \delta_1 \right) + \left(\frac{\varepsilon_{cu2} - \frac{\varepsilon_{c2}}{3}}{\varepsilon_{cu2} + \varepsilon_{yd}} \right) \quad (3.9)$$

The design value of moment resistance is obtained from:

$$\frac{M_{Rd,c}}{b*d^2*f_{cd}} = \xi \left[\frac{1-\xi}{2} - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4\varepsilon_{cu2}} \xi \right) + \frac{(1-\delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1-\delta_1} * \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right] \right] \quad (3.10)$$

with the normalised neutral axis depth computed from:

$$\xi = \frac{(1-\delta_1)*(V_d + \omega_{1d} - \omega_{2d}) + (1+\delta_1)\omega_{vd}}{(1-\delta_1)*\left(1 - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}}\right) + 2\omega_{vd}} \quad (3.11)$$

3.5. Detailed Design of RC Frame members in Shear

3.5.1. Capacity Design Shears in Beams or Columns

The monotonic or cyclic behaviour of concrete members in flexure is fairly ductile: after flexural yielding they can sustain significant inelastic deformations (i.e. rotations), with little loss of moment resistance. Their inherent flexural ductility can easily be improved further, by using dense, closed stirrups, or similar types of transverse reinforcement, which laterally confine the concrete and prevent the longitudinal bars from buckling. By contrast, concrete members are inherently brittle in shear, whether monotonic or cyclic: if they reach their shear resistance before yielding in flexure, they suffer a drastic, and often sudden, drop in resistance right after its peak. For this reason, shear failure of members before flexural yielding should be prevented by all means. Eurocode 8 accomplishes this goal by enforcing ‘capacity design’ of all members in shear.

Capacity design of beams or columns in shear is a more straightforward and effective application of the ‘capacity design’ concept than that in Section 3.4.2.1 for columns in flexure. This section will show that, once plastic hinges are presumed to form at the relevant member ends, equilibrium of moments suffices to establish the maximum possible shear force in the member which is physically permitted by the ‘capacities’, M_{Rd} , of the plastic hinges. By designing against this ‘capacity design’ shear, instead of the shear force from the analysis for the seismic design situation, we preclude shear failure of the beams or the column not only before flexural yielding, but also afterwards; indeed indefinitely, for any level of earthquake.

3.5.2. Detailed Design of Beams in Shear

3.5.2.1. Dimensioning of Beams for ULS in Shear

Eurocode 2 uses for the ULS resistance in shear the variable strut inclination truss model: a model with angle of inclination, θ , of the compression stress field in the web with respect to the member axis which varies in the range:

$$0.4 \leq \tan \theta \leq 1 \quad (22^\circ \leq \theta \leq 45^\circ) \quad (3.12)$$

According to this model:

1. Transverse reinforcement with design value of yield stress f_{ywd} and geometric ratio $\rho_w = A_{sh}/b_{wsh}$ (where A_{sh} is the total area of transverse reinforcement with spacing S_h along the beam) contributes a shear resistance equal to

$$V_{Rd,s} = \rho_w b_w z f_{ywd} \cot \theta \quad (3.13)$$

2. The shear resistance cannot exceed the following limit value, without failure of the web in diagonal compression:

$$V_{Rd,max} = 0.3 b_w z \left(1 - \frac{f_{ck}(MPa)}{250}\right) f_{cd} \sin 2\theta \quad (3.14)$$

The general procedure for dimensioning in shear a section x of a beam is the following:

1. $V_{Ed}(x)$ is set equal to $V_{Rd,max}$ and Equation 3.14 is inverted for a value of θ .
2. In the very unlikely case that $V_{Rd,max}$ is less than $V_{Ed}(x)$ even for $\theta = 45^\circ$, the width of the web is increased so that $\theta \leq 45^\circ$.
3. In the very usual case when the condition $V_{Ed}(x) = V_{Rd,max}$ gives a θ -value below the lower limit in Equation 3.12, θ is set equal to that limit.
4. The shear reinforcement is dimensioned by setting: $V_{Ed}(x) = V_{Rd,s}$ for the final value of θ .
5. Dimensioning of the shear reinforcement starts at a section at a distance d from the face of a supporting column; the so-dimensioned shear reinforcement at the section is maintained to the face of the column.
6. Apart from point 5 above, a reverse 'shift rule' applies to the shear reinforcement determined at section x : it can be maintained constant over a distance $z \cot \theta$ in the direction of increasing shears, that is, toward the nearest support.

The above apply both to the 'seismic' design situation and the 'persistent and transient' one. The shear reinforcement chosen should respect the detailing rules prescribed in Eurocodes 2 and 8, summarised in Table 3.6.

Apart from the special dimensioning rules for DC H beams highlighted in Section 3.5.2.2, the only difference that design against seismic actions as per Eurocode 8 or for non-seismic ones as per Eurocode 2 makes for beams in shear is the special detailing prescribed in Eurocode 8 for the stirrups in the end regions where plastic hinges are likely to form.

These are termed ‘critical regions’, and a conventional length is specified for them. The prescribed maximum stirrup spacing as a multiple of the longitudinal bar diameter aims at preventing buckling of these bars (which is much more likely for bars subjected to alternate tension and compression, as is the case during the earthquake action).

The stirrup diameter and spacing are constant within each ‘critical region’, obeying the relevant detailing rules in Table 3.6. They are determined from the condition $V_{Ed}(x) = V_{Rd,s}$ at a distance $x = d$ from the column face. A practical implication of the different detailing of ‘critical regions’ is that shear reinforcement in the rest of the beam is dimensioned from the condition $V_{Ed}(x) = V_{Rd,s}$ at a distance, x , from the column face equal to the ‘critical region’ length plus $z \cot \theta$. It is normally kept constant between the ‘critical regions’, as controlled by the most demanding section beyond a distance of $z \cot \theta$ from their ends.

Table 3.6 EC8 detailing rules for the transverse reinforcement of primary beams

	DC H	DC M	DC L
Outside critical regions			
Spacing, $s_h \leq$	0.75d		
$\rho_w = A_{sh}/b_w s_h \geq$	$(0.08\sqrt{f_{ck}}(\text{MPa}))/f_{yk}(\text{MPa})^a$		
In critical regions			
Diameter, $d_{bw} \geq$	6 mm		
Spacing, $s_h \leq$	$6 d_{bl}^b, h/4, 24d_{bw}, 175$	$8 d_{bl}^b, h/4, 24d_{bw}, 225$	-

^a NDP (nationally determined parameter) per EC2; the value recommended in EC2 is given here.

^b d_{bl} : minimum diameter of all top and bottom longitudinal bars within the critical region.

3.5.2.2. Special Rules for Seismic Design of Critical Regions In DC H Beams for ULS In Shear

In DC H beams, additional Eurocode 8 rules further differentiate the dimensioning of ‘critical regions’ in shear from the rest of the beam. For the dimensioning of these regions in the ‘seismic design situation’, Eurocode 8 sets in Equations 3.13, 3.14 the strut inclination, θ , equal to 45° . This choice (i.e. $\tan \theta = 1$) gives the minimum value of $V_{Rd,s}$ in the range of θ per Eurocode 2, Equation 2.12. It amounts to a classical Mörsh-Ritter 45° -truss for the design in shear without a concrete contribution term. The reason of this choice is that in plastic hinges the shear resistance due to the transverse reinforcement decreases with increasing inelastic cyclic deformations (Biskinis, Roupakias and Fardis, 2004); the magnitude of these deformations is significant in beams of DC H. Despite this apparently large penalty on $V_{Rd,s}$, the density of beam stirrups in the ‘critical regions’ of DC H beams is usually controlled by the detailing requirements at the last row of Table 3.4.

3.5.3. Detailed Design of Columns in Shear

In columns designed to Eurocode 8, plastic hinging under the design seismic action is the exception. If it does take place, it leads to lower ductility demands than in DC H beams, and hence, leads to a smaller reduction of shear resistance. So, Eurocode 8 neglects this reduction for columns.

Columns are subjected to almost full shear reversals. Nevertheless, Eurocode 8 does not require for them inclined bars against shear sliding, trusting their axial force to close through-cracks of the end section against the low plastic strains that may build up in the vertical bars. Sliding is also resisted by clamping and dowel action of the large diameter intermediate bars between the corners, which remain elastic when the peak shear and moment occur in the column. Therefore, the dimensioning of columns in shear takes place according to the Eurocode 2 alone, which is, taking into account the effect of axial load on shear resistance as follows:

1. A compressive axial force, N_d , increases the shear resistance, $V_{Rd,s}$, due to the transverse reinforcement by the transverse component of the strut which carries N from the compression zone at the top section of the column to that of the bottom at an inclination of z/H_{cl} to the column axis:

$$V_{Rd,s} = \frac{z}{H_{cl}} N_d + \rho_w b_w z f_{ywd} \cot \theta \quad (3.15)$$

2. Eurocode 2 introduces in $V_{Rd,max}$ an empirical multiplicative factor, which is a function of $v_d = N_d/A_c f_{cd}$ and takes into account: (a) the contribution of N_d to shear resistance, at the same time as (b) the burden placed on the inclined compression field accompanying the tension in the transverse reinforcement by the normal stress component in the strut due to N for $v_d > 0.5$:

$$V_{Rd,max} = 0.3 \min(1.25; 1 + v_d; 2.5(1 - v_d)) b_w z \left(1 - \frac{f_{ck}(Mpa)}{250}\right) f_{cd} \sin 2\theta \quad (3.16)$$

With these modifications in the shear resistance formulas, steps 1 to 4 of the general procedure in Section 3.5.2.1 for dimensioning beams in shear are also applicable to columns. This procedure is followed separately in the two transverse directions of the column, using the corresponding values of $V_{CD,c}$ as design shears. In rectangular columns, the side length at right angles to the plane of bending is used as b_w in Equations 2.15, 2.16 and 90% of the effective depth, d , in the other direction as z .

Column sides longer than about 250 mm in DC H or 300 mm in DC M should have intermediate vertical bars engaged at a corner of a stirrup or by the hook of a cross-tie (see relevant rule in Table 3.5, row 3 from the bottom). The legs of these intermediate stirrups or cross-ties contribute to the shear resistance per Equation 3.15 at right angles to the column side; their cross-sectional area enters in $\rho_w = A_{sh}/b_{wsh}$ multiplied by $\cos \alpha$, where α is the angle between the leg and the direction of the shear force. Although the cross-sectional area and/or spacing of intermediate stirrups or cross-ties may well differ from those of the perimeter hoops, they are usually chosen the same, for simplicity.

The transverse reinforcement should respect the detailing rules in Table 3.7. Except those concerning the effective mechanical ratio $a\omega_{wd}$ of stirrups, which have a fundamental basis, these rules are empirical. As in beams, the rule prescribing the maximum stirrup spacing in ‘critical regions’ as a multiple of the diameter of longitudinal bars aims at preventing buckling.

If the stirrup diameter and/or spacing are not controlled by the design shear, $V_{CD,c}$, which is constant along the column, but by the detailing rules, which are different in ‘critical regions’ and outside, the transverse reinforcement may be chosen different in each ‘critical region’ in a storey and in-between these regions. For simplicity, the transverse reinforcement

is often chosen the same throughout the storey, as controlled by the most demanding of the two ‘critical regions’.

Table 3.7 EC8 detailing rules for transverse reinforcement in primary columns

	DC H	DC M	DC L
Critical region length ^a \geq	1.5h _c , 1.5b _c , 0.6 m, H _{cl} /5	h _c , b _c , 0.45 m, H _{cl} /6	h _c , b _c ,
Outside the critical regions			
Diameter, d _{bw} \geq	6 mm, d _{bl} /4		
Spacing, s _w \leq	20d _{bl} , h _c , b _c , 400 mm		
At lap splices of bars with	12d _{bl} , 0.6h _c , 0.6b _c , 240 mm		
In critical regions ^b			
Diameter, d _{bw} \geq ^c	6 mm, 0.4√(f _{yd} /f _{ywd})d _{bl}	6 mm, d _{bl} /4	
Spacing, s _w \leq ^{c,d}	6d _{bl} , b _o /3, 125 mm	8d _{bl} , b _o /2, 175 mm	As outside critical regions
Mechanical ratio ω _{wd} \geq ^e	0.08	-	-
Effective mechanical ratio aω _{wd} \geq ^{d,e,f,g}	30 μ _φ *v _d ε _{yd} b _c /b _o - 0.035	-	-
In the critical region at the base of the column (at the connection to the foundation)			
Mechanical ratio ω _{wd} \geq	0.12	0.08	-
Effective mechanical ratio aω _{wd} \geq ^{d,e,f,h,i}	30 μ _φ v _d ε _{yd} b _c /b _o - 0.035		-

^a h_c, b_c, H_{cl}: column sides and clear length.

^b For DC M: If a value of q ≤ 2 is used for the design, the transverse reinforcement in critical regions of columns with an axial load ratio v_d ≤ 0.2 may follow only the rules for DC L columns.

^c For DC H: In the two lower storeys of the building, the requirements on d_{bw}, s_w apply over a distance from the end section not less than 1.5 times the critical region length.

Index c denotes the full concrete section; index o the confined core to the centreline of the perimeter hoop; b_o is the smaller side of this core.

^d Index c denotes the full concrete section; index o the confined core to the centreline of the perimeter hoop; b_o is the smaller side of this core.

^e ω_{wd}: volume ratio of confining hoops to confined core (to centreline of perimeter hoop) times f_{ywd}/f_{cd}.

^f a = (1 - s/2b_o)(1 - s/2h_o)(1 - {b_o/[(n_h - 1)h_o] + h_o/[(n_b - 1)b_o]} / 3); confinement effectiveness factor of rectangular hoops at spacing s, with n_b legs parallel to the side of the core with length b_o and n_h legs parallel to the side of length h_o.

^g For DC H: at column ends protected from plastic hinging through the capacity design check at beam-column joints, μ_φ* is the value of the curvature ductility to 2/3 of the basic value, q_o, of the behaviour factor applicable to the design; at the ends of columns where plastic hinging is not prevented, μ_φ* is taken equal to μ_φ defined in footnote h (see also footnote i); ε_{yd} = f_{yd}/E_s.

^h μ_φ: curvature ductility factor to the basic value, q_o, of the behaviour factor applicable to the design.

ⁱ For DC H: The requirement applies also in the critical regions at the ends of columns where plastic hinging is not prevented.

3.6. Detailing for Ductility

3.6.1. Critical Regions in Ductile Members

Of the two constituents of reinforced concrete, steel is ductile in tension but not in compression, as bars may buckle, shedding their force and risking fracture. Concrete is brittle, unless its lateral expansion is well restrained by confinement. So, the only way to build a RC member which is ductile and can reliably dissipate energy during inelastic seismic response is by combining:

Reinforcing bars in the direction where tensile principal stresses are expected to develop; and Concrete and reinforcement in the direction of compressive principal stresses, with dense ties to laterally confine the concrete and restrain the bars against buckling.

This is feasible wherever principal stresses and strains develop during the seismic response invariably in the directions where reinforcement can be conveniently placed. In one-dimensional RC members (beams, columns, slender walls), it is convenient to place the reinforcement in the longitudinal and transverse directions. Cyclic flexure indeed produces at the extreme fibres of a RC member principal stresses and strains in the longitudinal direction and allows effective use of reinforcement, both to take up directly the tension and to restrain the concrete and the compression bars transverse to their compressive stresses. Flexure is the only mechanism of force transfer in such a member, which allows using to advantage and reliably the ductility of tension reinforcement and effectively enhancing the ductility of concrete and of compression bars through lateral restraint. The regions of the member dominated by flexure under seismic loading are its ends, where the seismic moments take their maximum value. After flexural yielding of the end section, a flexural plastic hinge develops there, dissipating energy in alternate positive and negative bending. Eurocode 8 calls this region 'critical region', which has a more conventional connotation than the term 'dissipative zone', used also in Eurocode 8 for the part of a member or connection of any material where energy dissipation takes place by design.

A 'critical region' is a conventionally defined part of a primary RC member, up to a certain distance from:

1. An end of a column or beam connected to a beam or a vertical element, respectively, no matter whether the relative magnitude of the moment resistances of the members around the connection show that a plastic hinge at that end is likely. Cantilevering beams not designed for a vertical seismic action, or a beam end supported on a girder at a distance from a joint of the girder with a vertical member, cannot develop large seismic moments; so there is no beam 'critical region' in those cases.
2. A beam section where the hogging moment from the analysis for the seismic design situation attains its maximum value along the span; often that section is at the beam end or nearby and the 'critical region' coincides with one of those described under 1 above.

4. Case Study

In this study, a multi-storey reinforced concrete building is designed according to the specifications and rules of European Standard EN 1992-1-1:2004 “Eurocode 2” to estimate the initial cross section, longitudinal and transverse reinforcement and materials specifications of vertical and horizontal reinforced concrete elements “Columns and Beams” which are required to resist uniaxial loads as “dead and live loads”, then as per European Standard EN 1998-1:2004 “Eurocode 8” for the seismic design and analysis of the building. The building has been considered in different hazard seismic zones “High seismic zone, Medium seismic zones and Low seismic zone”. Eurocode 8 provides 3 classes of ductility for the structures design “Ductility Class High “DCH”, Ductility Class Medium “DCM” and Ductility Class Low “DCL” to resist the seismic actions and dissipate the seismic energy to mitigate the damages of the earthquake on the structures. For each hazard seismic zone, the building has been considered in three states of ductility classes; DCL, DCM and DCH. The building has been analysed using Nonlinear Pushover and Time History analysis methods, the results of the nonlinear analysis will be helpful to justify the importance of determine a specific ductility class for multi-storey regular RC buildings depend on seismic zone.

4.1. Building Characteristics

The structure of the building is a reinforced concrete moment resistant frame, the building is a regular building in plan and elevation and has importance class II (ordinary), it consists from six typical storeys above ground level and the foundation of the building has considered as raft foundation “Fixed-Foundation”. The typical storey height is equal to 3.20 m, the total span of the building in X-direction is equal to 15 m and it is divided to 3 bays with span of 5 m for each bay, and the total span of the building in Y-direction is equal to 16 m and it is divided to 4 bays with span of 4 m for each bay, The slabs structure is considered as a solid slab with thickness of 0.15 m. Figures 4.1 to 4.3 are showing the plan and elevation perspectives of the building.

4.2. Loads on the buildings

The building is designed to be a residential building, therefor the building is exposed to the dead loads (own weight (Frame members and Slabs), infill masonry walls weight and covers (finishes and furniture, etc.)) and live loads.

$$\text{Slab} = 3.75 \text{ KN/m}^2$$

$$\text{Cover load} = 2.50 \text{ KN/m}^2$$

$$\text{Live Load} = 2 \text{ KN/m}^2$$

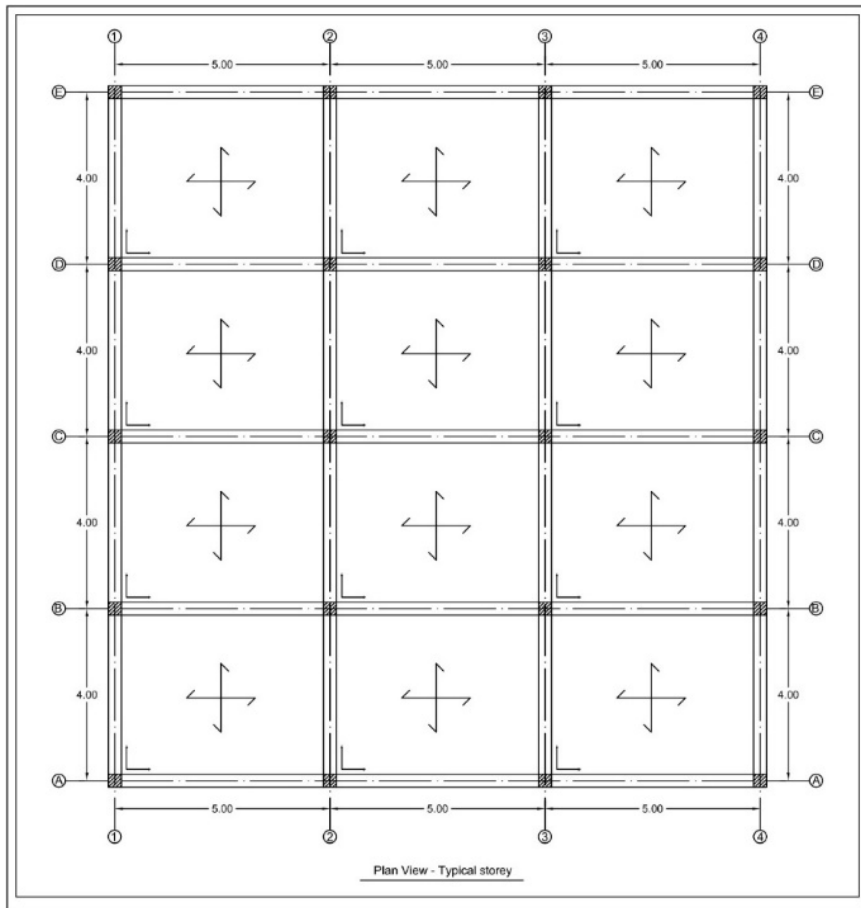


Figure 4.1 Plan view showing Axis, columns and beams of the building with the dimensions.

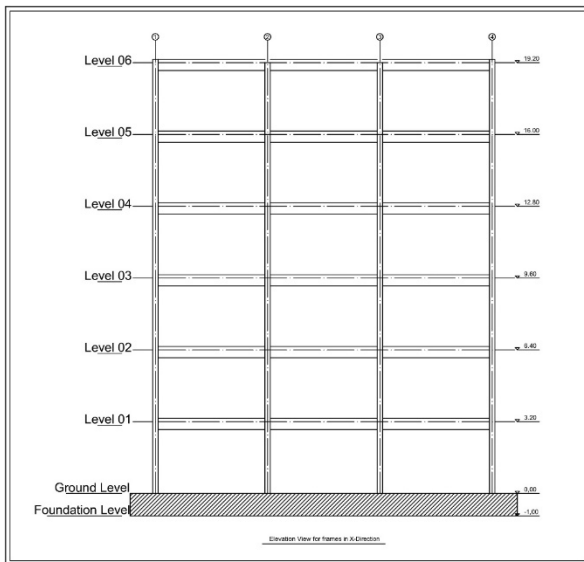


Figure 4.2 Elevation view of the building showing the frames in X direction

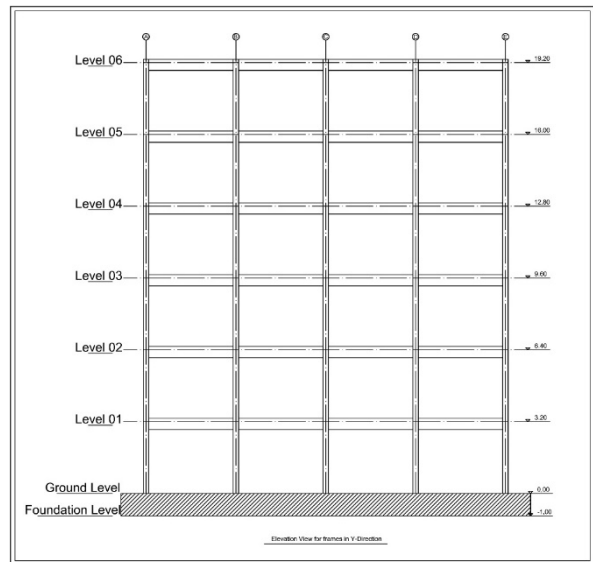


Figure 4.3 Elevation view for the building showing the frames in Y direction.

4.3. Hazard Seismic Zones

According to Portugal seismic zonation for Portuguese National Annex of NP EN 1998-1, the building has been designed in all seismic zones exist in Portugal, for Type 1 and Type 2 seismic actions, to choose the more critical Type seismic action in each zone,

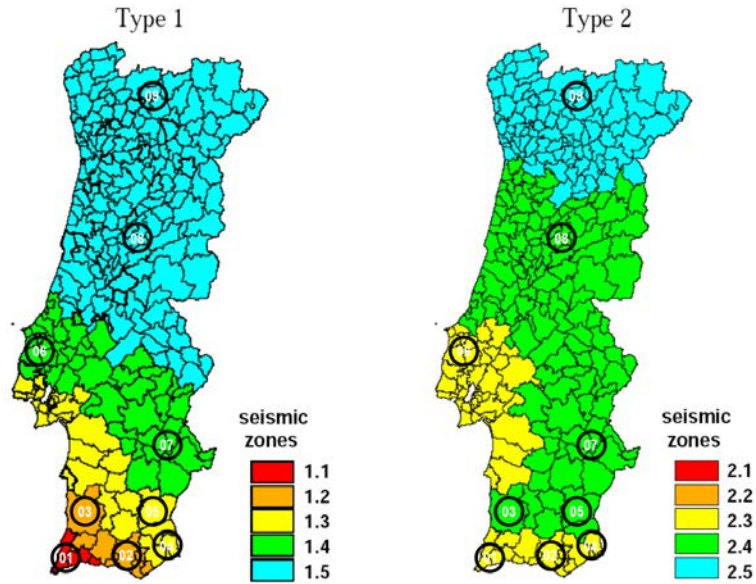


Figure 4.4 Hazard Seismic zones

Table 4.1 Hazard Seismic zones in Portugal Type 1 and Type 2

	Seismic Zones in Portugal	
	Type 01	Type 02
01	1.1	2.3
02	1.2	2.3
03	1.2	2.4
04	1.3	2.3
05	1.3	2.4
06	1.4	2.3
07	1.4	2.4
08	1.5	2.4
09	1.5	2.5

According to previous chapter, a full example for seismic design of the building with different ductility considerations; DC H, DC M, DC L for one seismic zone is solved in Appendix A, B and C respectively, to show how we can apply the equations from chapter 3 to

design a real frame multi storey model. In the same way the building has been designed in different seismic zones type 1 and its counterpart seismic zones type 2.

The results of the design are used to dimension and sizing the frame elements columns and beams and the connections between them ‘column-beam joint’ to have reinforce concrete frame building, which has a sufficient strength capacity with the considered ductility, and able to resist the horizontal loads and reduce earthquake’s damages and reduce the human and economic losses.

After the new element’s size and dimensioning of the longitudinal and transverse reinforcement for the frame presented in chapter 5, a static Nonlinear analysis ‘Pushover analysis’, is use for the assessment of the building during the simulated earthquake, to evaluate the performance and efficiency of the frame structure with the different ductility in the same seismic zone for Type 1 and Type 2, presented in chapter 6.

Nonlinear static Pushover analysis, is a long process analysis, but it saves time more than Nonlinear dynamic analysis ‘Time History’ which needs a long time and losses too much time, however the Pushover analysis can give the same or almost the same results.

According to the long process of Pushover analysis, a computer software called ‘Sap2000’ is used to solve pushover equations and to get pushover curves much faster, without mistakes, the computer software is much faster and smarter according to the artificial intelligence, and the most important this is to choose a reliable software, with available online books and tutorials online, and has a good reputation for the design.

Figure 4.5 shows the building frame model from Sap2000, on the left side window is the designerd frame model in 2D prespective, and on the right side window is a full 3D model prespective for the full building.

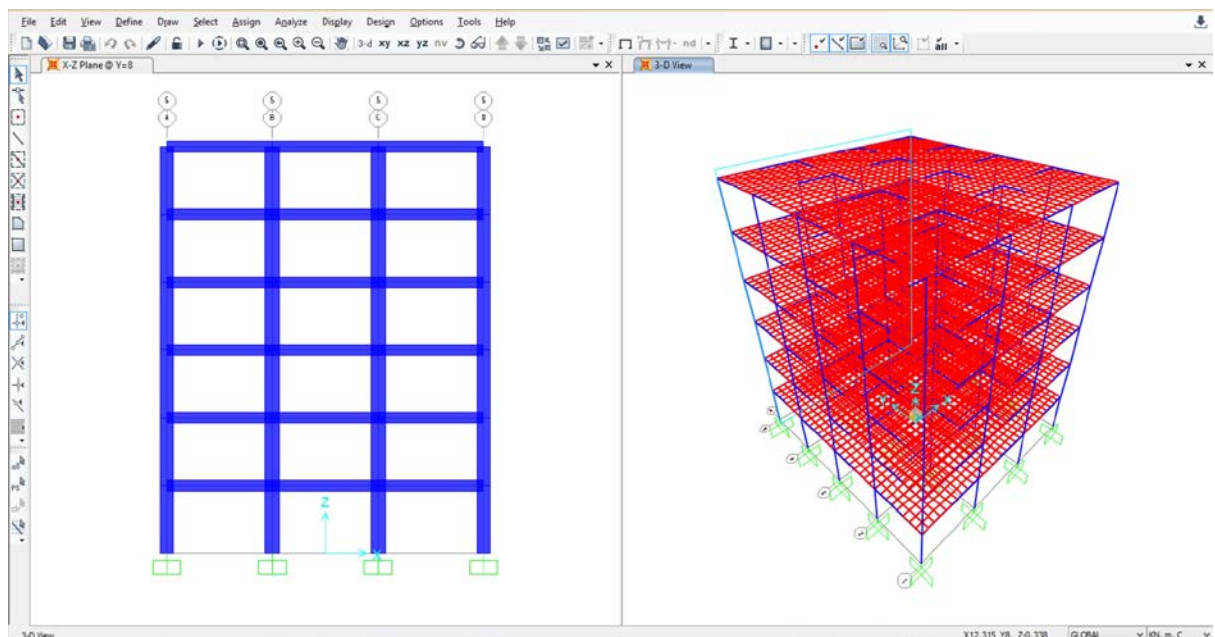


Figure 4 5 Building frame model in Sap2000, in 3D and 2D Perspectives

5. Design of the building

5.1. Design Assumption

5.1.1. Design according to Eurocode 2.

Using (Eurocode 2 Part 1-1, 2004) and Eurocode 2 Guides (Threlfall, 2009) for the initial design of the building, with the help of structural analysis software “SAP 2000 v19” to design the 6-storeys frame building mention in our case study, for the vertical uniaxial loads; the dead and live loads to design the vertical and horizontal RC elements “columns and beams” to achieve a safe concrete cross sections of the elements and the longitudinal reinforcement to resist the moment forces on the element and the transverse reinforcement to resist the shear forces on the elements and also to tie the longitudinal reinforcement, and determine the material should be used for the building elements according to the specifications of Eurocode 2, Design according to Eurocode 2 is out of the scope of this research, therefore we are going to show the results we got from the design directly.

Table 5.1 Concrete and reinforcement details for frame columns as per EC2

Eurocode 2		Exterior Columns		Interior Columns	
Concrete Cross section	Dim.	b_{column}	h_{column}	b_{column}	h_{column}
		30 cm	30 cm	30 cm	40 cm
	A_c	$9 * 10^2 \text{ cm}^2$		$12 * 10^2 \text{ cm}^2$	
Longitudinal reinforcement	d_{bl}	8 Ø 12		6 Ø 20	
	A_s	9.04 cm^2		12.06 cm^2	
	A_s / A_c	1 %		1 %	
Transverse Reinforcement		5 Ø 6 / m'		5 Ø 6 / m'	

Table 5.2 Concrete and reinforcement details for frame beams as per EC2

Eurocode 2		Exterior Beam		Intermediate Beam	
Concrete Cross section	Dim.	b_{beam}	h_{beam}	b_{beam}	h_{beam}
		25 cm	50 cm	25 cm	50 cm
	A_c	$12.5 * 10^2 \text{ cm}^2$		$12.5 * 10^2 \text{ cm}^2$	
Longitudinal reinforcement	$d_{\text{bl tens,top}}$	2 Ø 16 + 1 Ø 12		2 Ø 16 + 1 Ø 12	
	$d_{\text{bl tens,bottom}}$	2 Ø 16		3 Ø 12	
	$d_{\text{bl tens,top}}$	2 Ø 16 + 1 Ø 12		2 Ø 16 + 1 Ø 12	
	$d_{\text{bl comp}}$	2 Ø 12			
Transverse Reinforcement		6 Ø 8 / m'		6 Ø 8 / m'	

5.1.2. Design according to Eurocode 8.

Using the rules and recommendations of Eurocode 8, which are mentioned in chapter 3, the RC frame building mentioned in chapter 4 of case study, has been designed to resist seismic actions in different seismic zones inside the main land of Portugal, assuming that in each seismic zone the building will be in the three ductility classes mentioned in Eurocode 8 (DC L, DC M, DC H), to be able to have adequate results for each ductility class, to compare between the details of the frame, the total quantities for concrete and reinforcement, and to calculate the total cost for frames according to the price lists in Portugal.

Table 5.3 below show material classes for concrete and reinforcements, which are used during all the designer process for all ductility classes.

Table 5.3 Concrete and Reinforcement Material class (Betão, Nr and Nr, 2017)

Material	Class	Average Price
Concrete	C25/30, S3	67.15 €/ m ³
Longitudinal Reinforcement	A400	745-750 €/ Ton
Transverse Reinforcement	A400	805-825 €/ Ton

The prices shown in table 5.3 are just for the material price without the workmanship costs, which it is become more higher for DC M and DC H, due to the complexity of reinforcement detailing

There is a full example for seismic design of frame building according to Eurocode 8, for the three ductility classes DC L, DC M, and DC H, in Appendix A, B and C respectively.

5.2. Detailed design of frame columns

Frame columns are the most critical frame elements, as it is mentioned in chapter 3, the Eurocode 8 give strong criteria and recommendation for columns design, columns should be always strong more than beams (strong column- weak beam) to avoid building collapse during earthquakes.

Figure 5.1, shows the selected frame during the design, tables from 5.4 to 5.11 show the details of column design results according to Eurocode 8, for each seismic zone in Portugal, for three ductility classes (DC L, DC M and DC H).

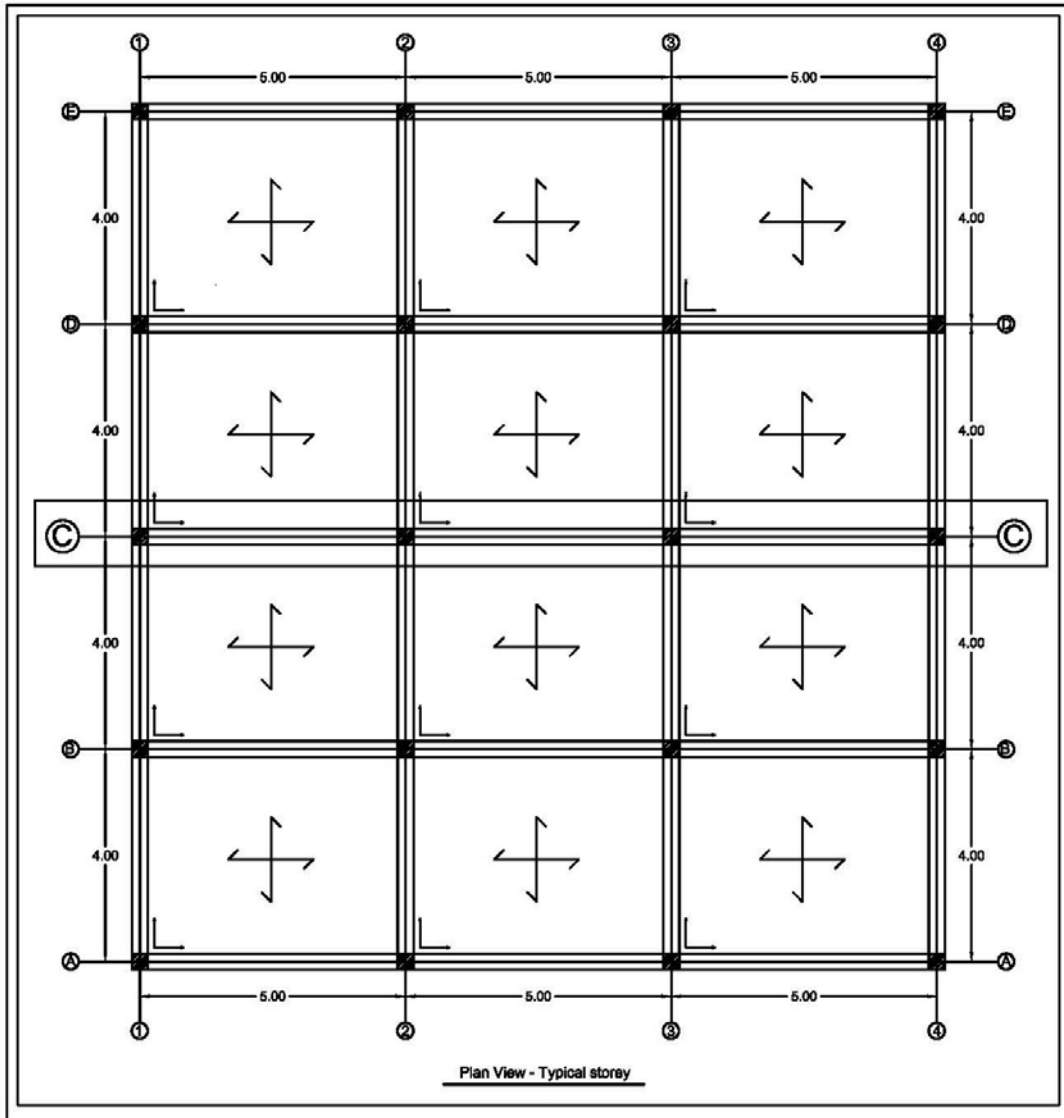


Figure 5 1 The selected RC frame for the design , Axis C-C

Table 5.4 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 1.1

Seismic Zone 1.1		Exterior Columns						Interior Columns						
		DCL		DCM		DCH		DCL		DCM		DCH		
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	
				65 cm	110 cm	35 cm	50 cm	35 cm	35 cm	65 cm	120 cm	35 cm	75 cm	35 cm
	A _c	71.5 *10 ² cm ²		17.5 *10 ² cm ²		12.25*10 ² cm ²		78 *10 ² cm ²		26.25 *10 ² cm ²		17.50 *10 ² cm ²		
Longitudinal reinforcement	d _{bl}	18 Ø 20		10 Ø 20		8 Ø 20		12Ø20+10Ø16		16 Ø 20		14 Ø 20		
	A _s	56.52 cm ²		31.4 cm ²		25.12 cm ²		57.78 cm ²		50.24 cm ²		43.96 cm ²		
	A _s / A _c	0.79 %		1.79 %		2.05 %		0.74 %		1.91 %		2.512 %		
	Lap splice length,		18 Ø 20		10 Ø 20		8 Ø 20		10 Ø 16	12 Ø 20	2 Ø 20	14 Ø 20	14 Ø 20	
			85 cm		90 cm		90 cm		60 cm	90 cm	85 cm	90 cm	100 cm	
Transverse Reinforcement	Critical Length	110 cm		50 cm		45 cm		120 cm		75 cm		50 cm		
	Inside critical length	6 Ø 6 / m'		7 Ø 6 / m'		8 Ø 8 / m'		7 Ø 6 / m'		7 Ø 6 / m'		8 Ø 8 / m'		
	Outside critical length	6 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		7 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		
	at lap splice	6 Ø 6 / m'		5 Ø 6 / m'		6 Ø 8 / m'		7 Ø 6 / m'		5 Ø 6 / m'		6 Ø 8 / m'		

Table 5.5 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 1.2

Seismic Zone 1.2		Exterior Columns						Interior Columns						
		DCL		DCM		DCH		DCL		DCM		DCH		
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	
				65 cm	110 cm	35 cm	50 cm	35 cm	35 cm	65 cm	120 cm	35 cm	75 cm	35 cm
	A _c	71.5 *10 ² cm ²		17.5 *10 ² cm ²		12.25*10 ² cm ²		78 *10 ² cm ²		26.25 *10 ² cm ²		17.50 *10 ² cm ²		
Longitudinal reinforcement	d _{bl}	18 Ø 20		6 Ø 20 + 4 Ø 16		8 Ø 20		16 Ø 20 + 6 Ø 16		12 Ø 20 + 4 Ø 16		14 Ø 20		
	A _s	56.52 cm ²		26.88 cm ²		25.12 cm ²		62.30 cm ²		45.72 cm ²		43.96 cm ²		
	A _s / A _c	0.79 %		1.536 %		2.05 %		0.8 %		1.74 %		2.512 %		
	Lap splice length,		18Ø20		4 Ø 16	6 Ø 20	8 Ø 20		6 Ø 16	16 Ø 20	4 Ø 16	12 Ø 20	14 Ø 20	
			85 cm		75 cm	90 cm	90 cm		60 cm	90 cm	65	100 cm	100 cm	
Transverse Reinforcement	Critical Length	110 cm		50 cm		45 cm		120 cm		75 cm		50 cm		
	Inside critical length	5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		
	Outside critical length	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		
	at lap splice	5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		

Table 5.6 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 1.3

Seismic Zone 1.3		Exterior Columns						Interior Columns					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}
		35 cm	80 cm	35 cm	40 cm	35 cm	35 cm	40 cm	100 cm	35 cm	50 cm	35 cm	50 cm
	A _c	28 *10 ² cm ²		14 *10 ² cm ²		12.25*10 ² cm ²		40 *10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²	
Longitudinal reinforcement	d _{bl}	14 Ø 20		4 Ø 20 + 6 Ø 16		8 Ø 20		14 Ø 20 + 8 Ø 16		14 Ø 20		14 Ø 20	
	A _s	43.96 cm ²		24.62 cm ²		25.12 cm ²		60.04 cm ²		43.96 cm ²		43.96 cm ²	
	A _s / A _c	1.57 %		1.75 %		2.05 %		1.5 %		2.51 %		2.51 %	
	Lap splice length,	14 Ø 20 85 cm	12 Ø 20 90 cm	6 Ø 16 65 cm	12 Ø 20 90 cm	8 Ø 20 90 cm		8 Ø 16 70 cm	2 Ø 20 85 cm	12 Ø 20 100 cm	14 Ø 20 95 cm		14 Ø 20 100 cm
Transverse Reinforcement	Critical Length	80 cm		45 cm		45 cm		100 cm		50 cm		50 cm	
	Inside critical length	5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'	
	Outside critical length	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'	
	at lap splice	5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'	

Table 5.7 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 1.4

Seismic Zone 1.4		Exterior Columns						Interior Columns					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}
		35 cm	50 cm	35 cm	40 cm	35 cm	35 cm	35 cm	85 cm	35 cm	50 cm	35 cm	50 cm
	A _c	26.25 *10 ² cm ²		14 *10 ² cm ²		12.25*10 ² cm ²		29.75 *10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²	
Longitudinal reinforcement	d _{bl}	12 Ø 20		4 Ø 20 + 6 Ø 16		8 Ø 20		14 Ø 20		14 Ø 20		14 Ø 20	
	A _s	37.68 cm ²		24.62 cm ²		25.12 cm ²		43.96 cm ²		43.96 cm ²		43.96 cm ²	
	A _s / A _c	2.15 %		1.75 %		2.05 %		1.47 %		2.51 %		2.51 %	
	Lap splice length,	12 Ø 20 85 cm		6 Ø 16 65 cm	12 Ø 20 90 cm	8 Ø 20 90 cm		2 Ø 20 85 cm	12 Ø 20 95 cm	14 Ø 20 95 cm		14 Ø 20 100 cm	
Transverse Reinforcement	Critical Length	50 cm		45 cm		45 cm		85 cm		50 cm		50 cm	
	Inside critical length	5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'	
	Outside critical length	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'	
	at lap splice	5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'	

Table 5.8 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 1.5

Seismic Zone 1.5		Exterior Columns						Interior Columns					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}
				35 cm	35 cm	35 cm	35 cm	35 cm	35 cm	35 cm	65 cm	35 cm	50 cm
	A _c	12.25*10 ² cm ²		12.25*10 ² cm ²		12.25*10 ² cm ²		22.75*10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²	
Longitudinal reinforcement	d _{bl}	8 Ø 20		8 Ø 20		8 Ø 20		14 Ø 20		14 Ø 20		14 Ø 20	
	A _s	25.12 cm ²		25.12 cm ²		25.12 cm ²		43.96 cm ²		43.96 cm ²		43.96 cm ²	
	A _s / A _c	2.05 %		2.05 %		2.05 %		1.93 %		2.51 %		2.51 %	
	Lap splice length,	8 Ø 20		8 Ø 20		8 Ø 20		2 Ø 20	12 Ø 20	14 Ø 20		14 Ø 20	
		85 cm		90 cm		90 cm		85 cm	100 cm	95 cm		95 cm	
Transverse Reinforcement	Critical Length	35 cm		45 cm		45 cm		65 cm		50 cm		50 cm	
	Inside critical length	5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'	
	Outside critical length	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'	
	at lap splice	5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'	

Table 5.9 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 2.3

Seismic Zone 2.3		Exterior Columns						Interior Columns					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}
				35 cm	40 cm	35 cm	35 cm	35 cm	35 cm	35 cm	65 cm	35 cm	50 cm
	A _c	14*10 ² cm ²		12.25*10 ² cm ²		12.25*10 ² cm ²		22.75*10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²	
Longitudinal reinforcement	d _{bl}	10 Ø 20		8 Ø 20		8 Ø 20		16 Ø 20		14 Ø 20		14 Ø 20	
	A _s	31.4 cm ²		25.12 cm ²		25.12 cm ²		50.24		43.96 cm ²		43.96 cm ²	
	A _s / A _c	2.24 %		2.05 %		2.05 %		2.2 %		2.51 %		2.51 %	
	Lap splice length,	10 Ø 20		8 Ø 20		8 Ø 20		2 Ø 20	14 Ø 20	2 Ø 20	12 Ø 20	14 Ø 20	
		85 cm		85 cm		90 cm		85 cm	100 cm	85 cm	100 cm	100 cm	
Transverse Reinforcement	Critical Length	40 cm		45 cm		45 cm		65 cm		50 cm		50 cm	
	Inside critical length	5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'	
	Outside critical length	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'	
	at lap splice	5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'	

Table 5.10 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 2.4

Seismic Zone 2.4		Exterior Columns						Interior Columns						
		DCL		DCM		DCH		DCL		DCM		DCH		
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	
		35 cm	35 cm	35 cm	35 cm	35 cm	35 cm	35 cm	50 cm	35 cm	50 cm	35 cm	50 cm	
	A _c	12.25*10 ² cm ²		12.25*10 ² cm ²		12.25*10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²		
Longitudinal reinforcement	d _{bl}	8 Ø 20		8 Ø 20		8 Ø 20		14 Ø 20		14 Ø 20		14 Ø 20		
	A _s	25.12 cm ²		25.12 cm ²		25.12 cm ²		43.96 cm ²		43.96 cm ²		43.96 cm ²		
	A _s / A _c	2.05 %		2.05 %		2.05 %		2.51 %		2.51 %		2.51 %		
	Lap splice length,		8 Ø 20		8 Ø 20		8 Ø 20		2 Ø 20	12 Ø 20	2 Ø 20	12 Ø 20	14 Ø 20	
			85 cm		85 cm		90 cm		85 cm	100 cm	85 cm	100 cm	100 cm	
Transverse Reinforcement	Critical Length	35 cm		45 cm		45 cm		50 cm		50 cm		50 cm		
	Inside critical length	5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		
	Outside critical length	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		
	at lap splice	5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		

Table 5.11 Concrete and reinforcement details for frame columns as per EC8 in Seismic Zone 2.5

Seismic Zone 2.5		Exterior Columns						Interior Columns						
		DCL		DCM		DCH		DCL		DCM		DCH		
Concrete Cross section	Dim.	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	b _{column}	h _{column}	
		35 cm	35 cm	35 cm	35 cm	35 cm	35 cm	35 cm	50 cm	35 cm	50 cm	35 cm	50 cm	
	A _c	12.25*10 ² cm ²		12.25*10 ² cm ²		12.25*10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²		26.25 *10 ² cm ²		
Longitudinal reinforcement	d _{bl}	8 Ø 20		8 Ø 20		8 Ø 20		14 Ø 20		14 Ø 20		14 Ø 20		
	A _s	25.12 cm ²		25.12 cm ²		25.12 cm ²		43.96 cm ²		43.96 cm ²		43.96 cm ²		
	A _s / A _c	2.05 %		2.05 %		2.05 %		2.51 %		2.51 %		2.51 %		
	Lap splice length,		8 Ø 20		8 Ø 20		8 Ø 20		2 Ø 20	12 Ø 20	2 Ø 20	12 Ø 20	14 Ø 20	
			85 cm		85 cm		90 cm		85 cm	100 cm	85 cm	100 cm	100 cm	
Transverse Reinforcement	Critical Length	35 cm		45 cm		45 cm		50 cm		50 cm		50 cm		
	Inside critical length	5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		5 Ø 6 / m'		8 Ø 6 / m'		8 Ø 8 / m'		
	Outside critical length	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 8 / m'		
	at lap splice	5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		5 Ø 6 / m'		6 Ø 6 / m'		6 Ø 8 / m'		

5.3. Detailed design of frame Beams

The seismic design of frame beams, concrete cross-section of beams considered have the same dimensions of the design according to Eurocode 2. Tables from 5.12 to 5.19, show the seismic design details of beams.

Table 5.12 Concrete and reinforcement details for frame beams in Seismic Zone 1.1

Seismic Zone 1.1		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
		25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	8 Ø 16		4 Ø 16		4 Ø 12		9 Ø 16		5 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	4 Ø 16		3 Ø 16		3 Ø 16		4 Ø 16		3 Ø 16		2 Ø 16	
	d _{bl tens,top}	9 Ø 16		5 Ø 16		3 Ø 16		9 Ø 16		5 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8 / m'		14 Ø 8 / m'		-		7 Ø 8 / m'		14 Ø 8 / m'	
	Exterior End	10 Ø 8 / m'		7 Ø 8 / m'		6 Ø 8 / m'		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	11 Ø 8 / m'		7 Ø 8 / m'		6 Ø 8 / m'		11 Ø 8 / m'		7 Ø 8 / m'		11 Ø 8 / m'	

Table 5.13 Concrete and reinforcement details for frame beams in Seismic Zone 1.2

Seismic Zone 1.2		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
		25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	5 Ø 16 + 2 Ø 12		3 Ø 16		4 Ø 12		8 Ø 16		4 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	4 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		2 Ø 16 + 1 Ø 12		2 Ø 16	
	d _{bl tens,top}	8 Ø 16		4 Ø 16		3 Ø 16		8 Ø 16		4 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8 / m'		14 Ø 8 / m'		-		11 Ø 8 / m'		14 Ø 8 / m'	
	Exterior End	9 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	10 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		9 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'	

Table 5.14 Concrete and reinforcement details for frame beams in Seismic Zone 1.3

Seismic Zone 1.3		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
		25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	5 Ø 16		3 Ø 16		4 Ø 12		6 Ø 16		4 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	4 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		2 Ø 16		2 Ø 16	
	d _{bl tens,top}	6 Ø 16		4 Ø 16		3 Ø 16		6 Ø 16		4 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8		14 Ø 8		-		11 Ø 8		14 Ø 8	
	Exterior End	8 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	8 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		8 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'	

Table 5.4 Concrete and reinforcement details for frame beams in Seismic Zone 1.4

Seismic Zone 1.4		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
		25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	4 Ø 16		3 Ø 16		2 Ø 16		5 Ø 16		4 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		2 Ø 16		2 Ø 16	
	d _{bl tens,top}	5 Ø 16		4 Ø 16		3 Ø 16		5 Ø 16		4 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8 / m'		14 Ø 8 / m'		-		11 Ø 8 / m'		14 Ø 8 / m'	
	Exterior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	7 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'	

Table 5.5 Concrete and reinforcement details for frame beams in Seismic Zone 1.5

Seismic Zone 1.5		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
				25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	4 Ø 12		2 Ø 16		2 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	3 Ø 16		3 Ø 16		3 Ø 16		2 Ø 16		2 Ø 16		2 Ø 16	
	d _{bl tens,top}	3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8 / m'		14 Ø 8 / m'		-		11 Ø 8 / m'		14 Ø 8 / m'	
	Exterior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	6 Ø 8 / m'		6 Ø 8		6 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'	

Table 5.6 Concrete and reinforcement details for frame beams in Seismic Zone 2.3

Seismic Zone 2.3		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
				25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	3 Ø 16		2 Ø 16		4 Ø 12		4 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		2 Ø 16		2 Ø 16	
	d _{bl tens,top}	4 Ø 16		3 Ø 16		3 Ø 16		4 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8 / m'		14 Ø 8 / m'		-		11 Ø 8 / m'		14 Ø 8 / m'	
	Exterior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'	

Table 5.7 Concrete and reinforcement details for frame beams in Seismic Zone 2.4

Seismic Zone 2.4		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
				25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	4 Ø 12		2 Ø 16		2 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	5 Ø 12		3 Ø 16		3 Ø 16		2 Ø 16		2 Ø 16		2 Ø 16	
	d _{bl tens,top}	3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8 / m'		14 Ø 8 / m'		-		11 Ø 8 / m'		14 Ø 8 / m'	
	Exterior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'	

Table 5.19 Concrete and reinforcement details for frame beams in Seismic Zone 2.5

Seismic Zone 2.5		Exterior Beam						Intermediate Beam					
		DCL		DCM		DCH		DCL		DCM		DCH	
Concrete Cross section	Dim.	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}	b _{beam}	h _{beam}
				25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm	25 cm	50 cm
	A _c	12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²		12.5*10 ² cm ²	
Longitudinal reinforcement	d _{bl tens,top}	2 Ø 16		2 Ø 16		2 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl tens,bottom}	3 Ø 16		3 Ø 16		3 Ø 16		2 Ø 16		2 Ø 16		2 Ø 16	
	d _{bl tens,top}	3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16		3 Ø 16	
	d _{bl comp}	2 Ø 12											
Transverse Reinforcement	Critical Length	50 cm		75 cm		75 cm		50 cm		75 cm		75 cm	
	In critical length	-		11 Ø 8		14 Ø 8		-		11 Ø 8		14 Ø 8	
	Exterior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		-		-		-	
	Middle span	5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'		5 Ø 6 / m'	
	Interior End	6 Ø 8 / m'		6 Ø 8 / m'		6 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'		5 Ø 8 / m'	

According to the results referred in tables from 5.4 to 5.11, for detailing of columns, and tables from 5.12 to 5.19, for detailing of beams. Figures from 5.2 to 5.5, are showing the details drawing for DC L and DC M and DC H, in seismic zone 1.1. Appendix D, is presenting all the detailed drawing for the three ductility classes for all the seismic zones.

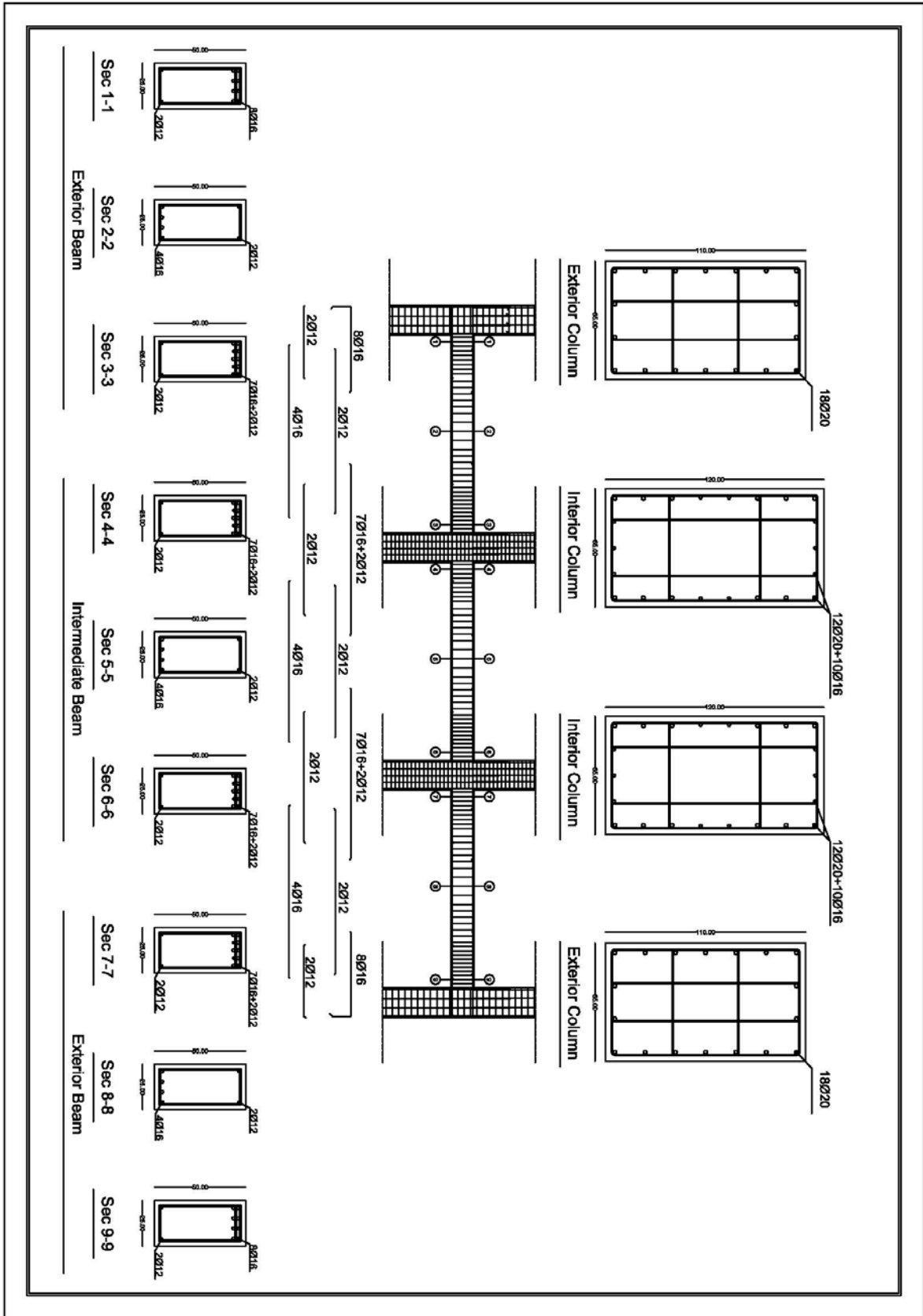


Figure 5.2 Detailed layout design of columns and beams of RC frame for DC L in seismic zone 1.1

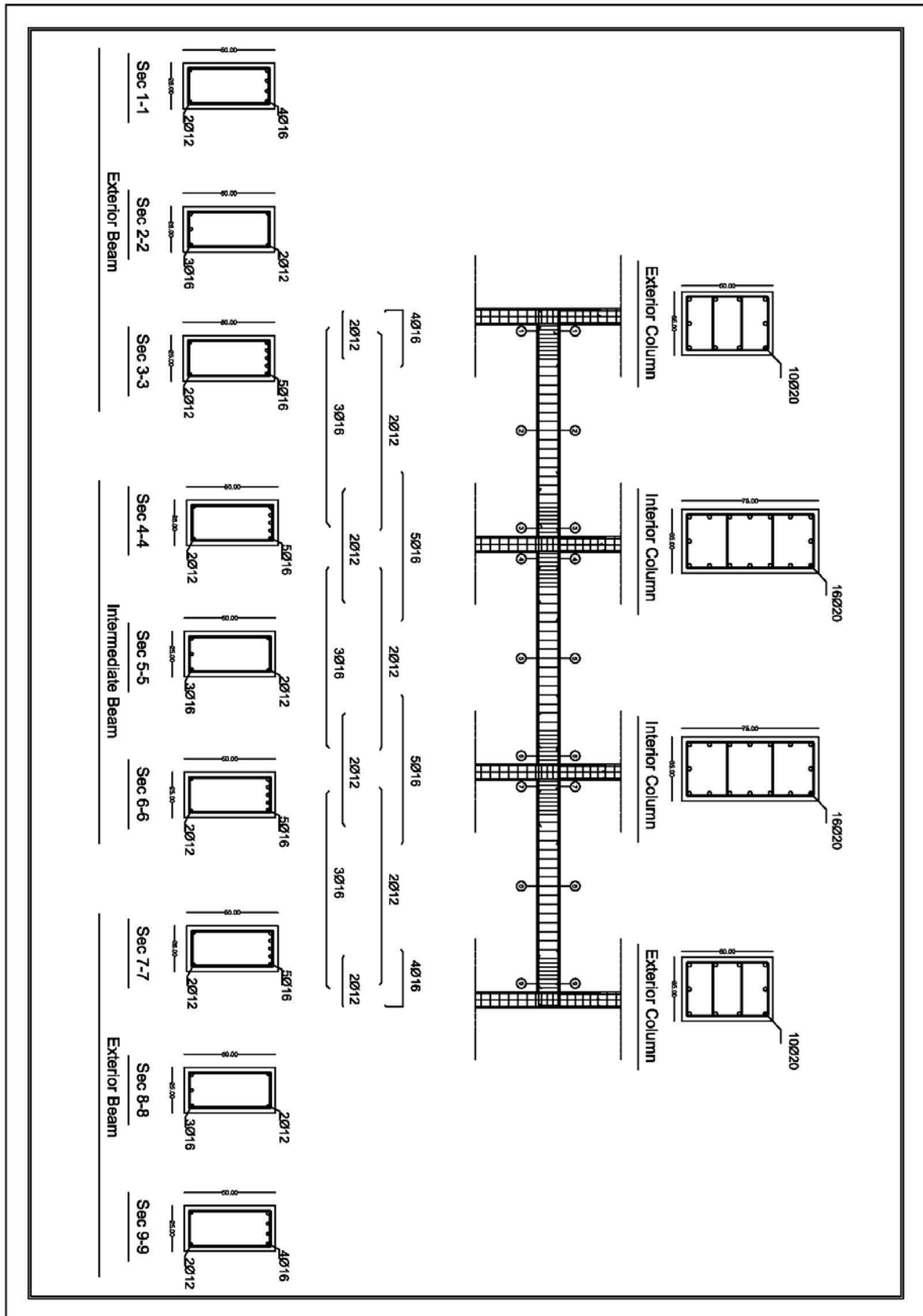


Figure 5.3 Detailed layout design of columns and beams of RC frame for DCM in seismic zone 1.1

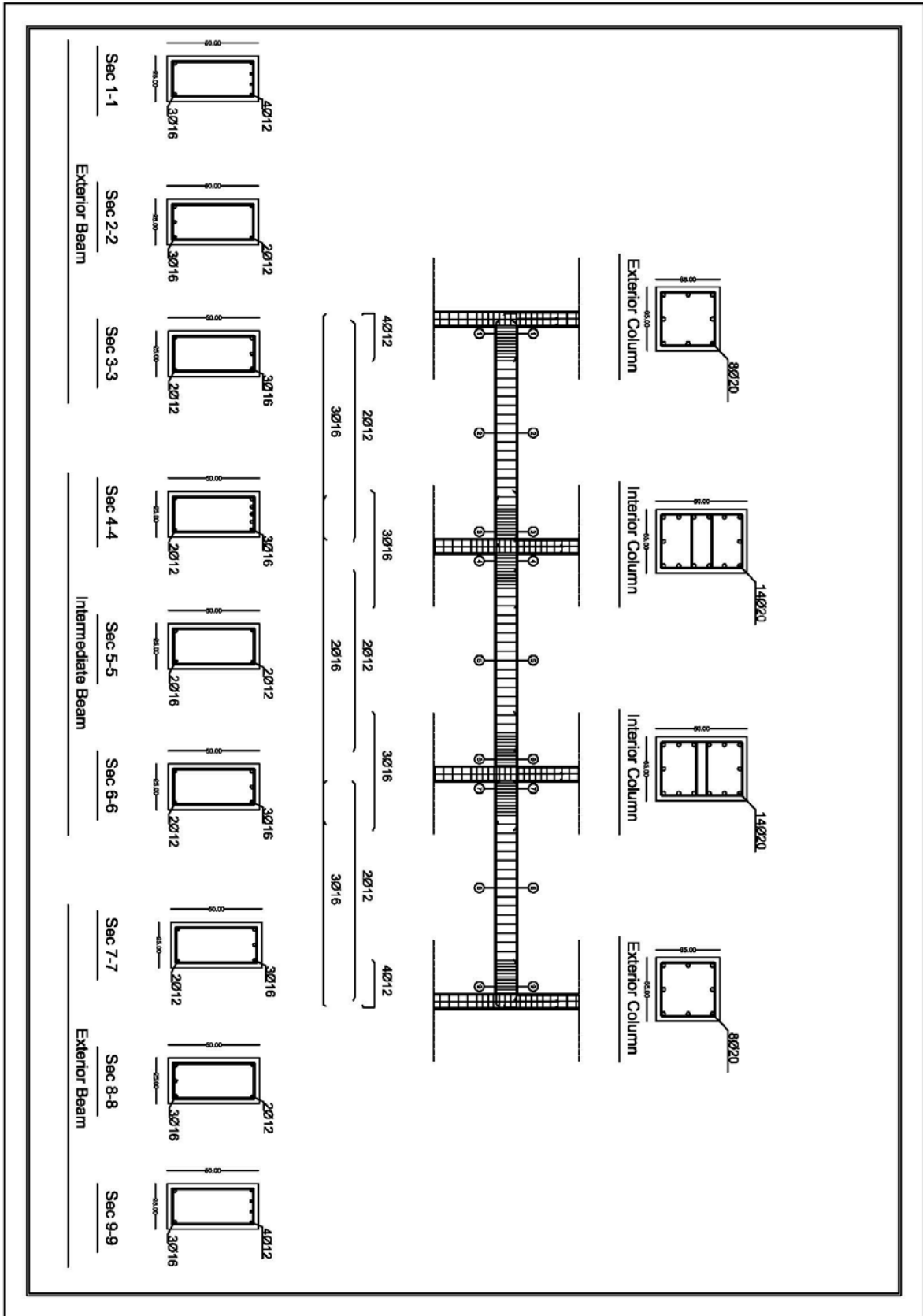


Figure 5.4 Detailed layout design of columns and beams of RC frame for DC H in seismic zone 1.1

5.4. Total quantities for the frame

The total quantities of required concrete and reinforcement for the designed frame had been calculated depend on the design details shown in section 5.3, the graphs below are showing the total concrete required for the three ductility classes (DC L, DC M, and DC H) for each seismic zone. The graphs can be used to compare the results much easier and faster. The graphs also show steel / concrete ratio in (kg/m^3). Figures from 5.5 to 5.20 below are showing total concrete and reinforcement quantities in m^3 and Kg respectively, and reinforcement / concrete ration in Kg/m^3 , for the designed frame referred in figure 5.1.

Seismic Zone 1.1

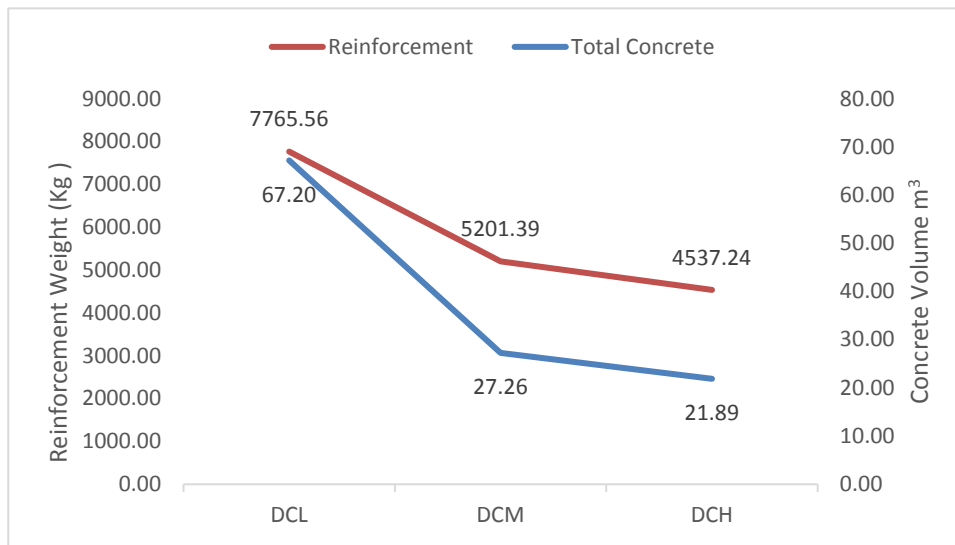


Figure 5.5 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 1.1

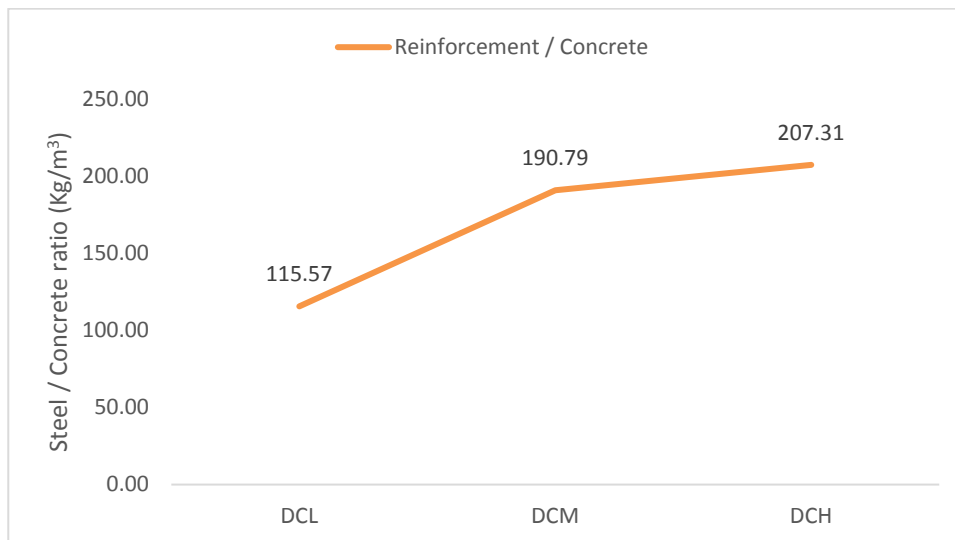


Figure 5.6 Reinforcement / concrete ratio (Kg/m^3) for DC L, D M and DC H frames in Seismic zone 1.1

In seismic zone 1.1, the required quantities of concrete and reinforcement are less for medium and high ductility classes, however, reinforcement / concrete ratio is increasing, with the increase of ductility level.

Seismic Zone 1.2

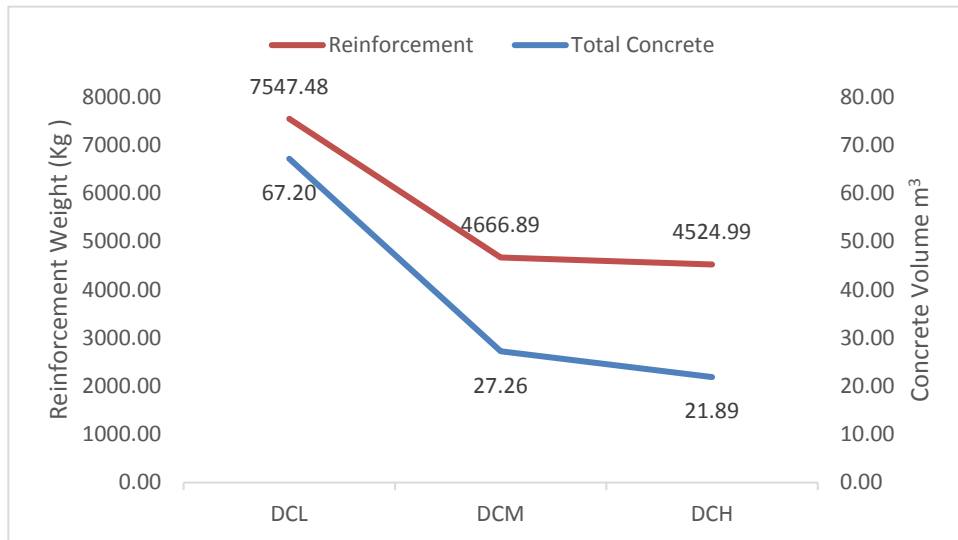


Figure 5.7 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 1.2

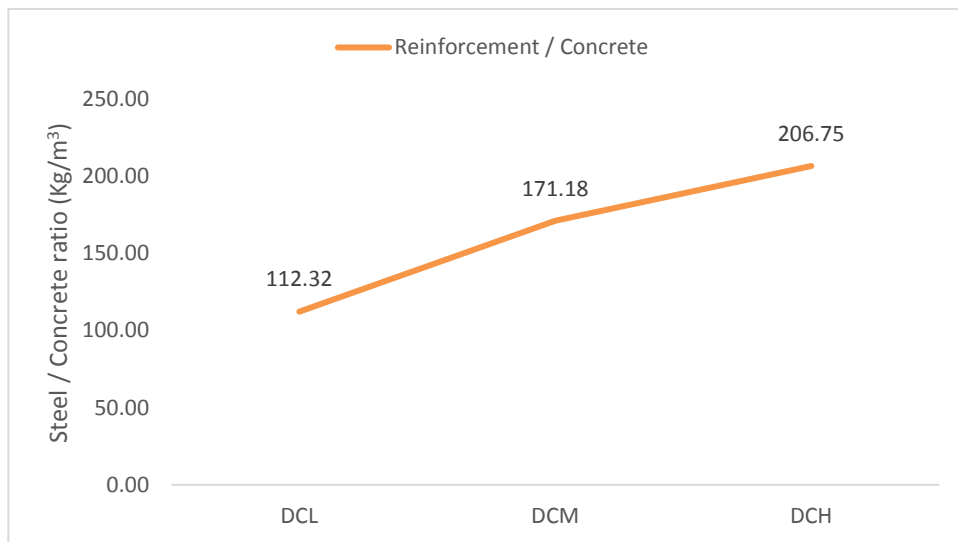


Figure 5.8 Reinforcement / concrete ratio (Kg/m³) for DC L, D M and DC H frames in Seismic zone 1.2

In seismic zone 1.2, the required quantities of concrete and reinforcement are less for medium and high ductility classes, however, reinforcement / concrete ratio is increasing, with the increase of ductility level.

Seismic Zone 1.3

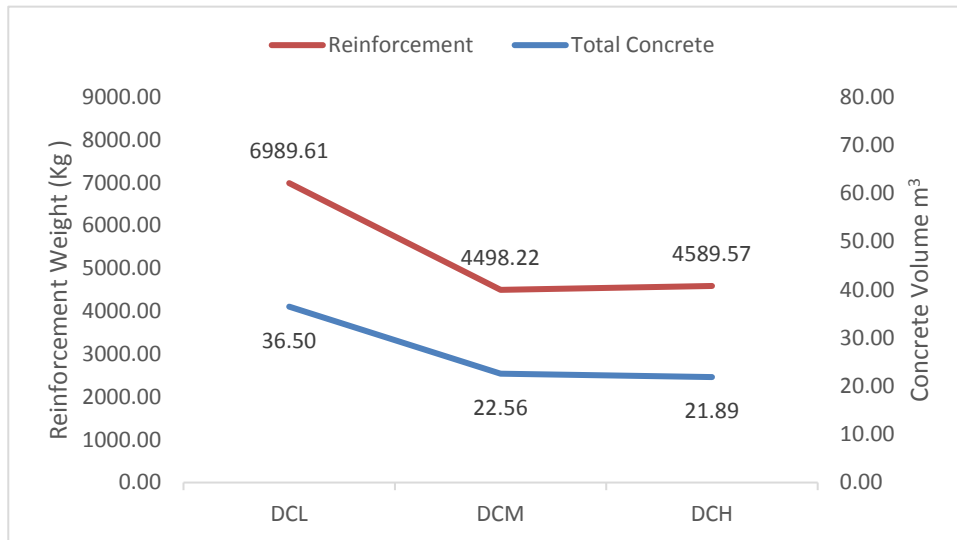


Figure 5.9 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 1.3

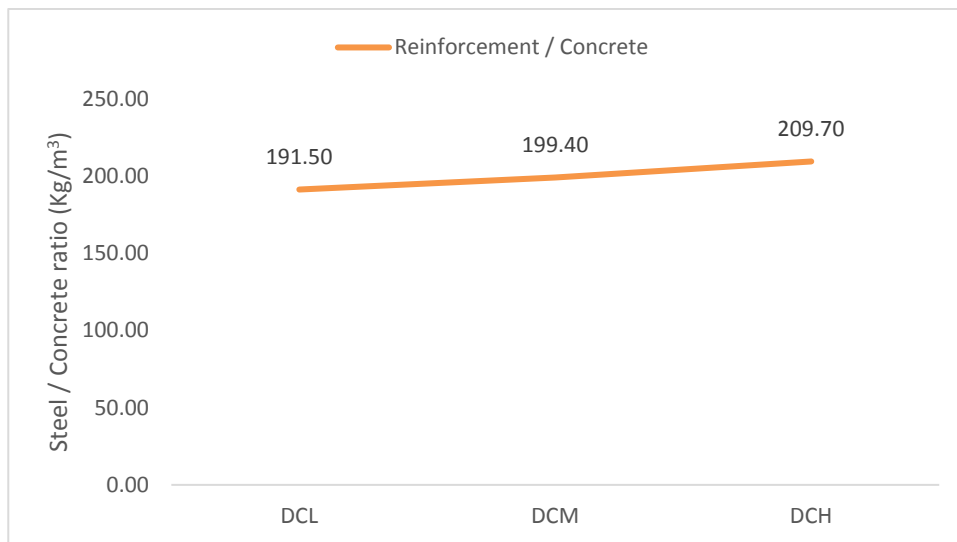


Figure 5.10 Reinforcement / concrete ratio (Kg/m³) for DC L, D M and DC H frames in Seismic zone 1.3

In seismic zone 1.3, the required quantities of concrete and reinforcement are less for medium and high ductility classes, however, reinforcement / concrete ratio is increasing, with the increase of ductility level.

Seismic Zone 1.4

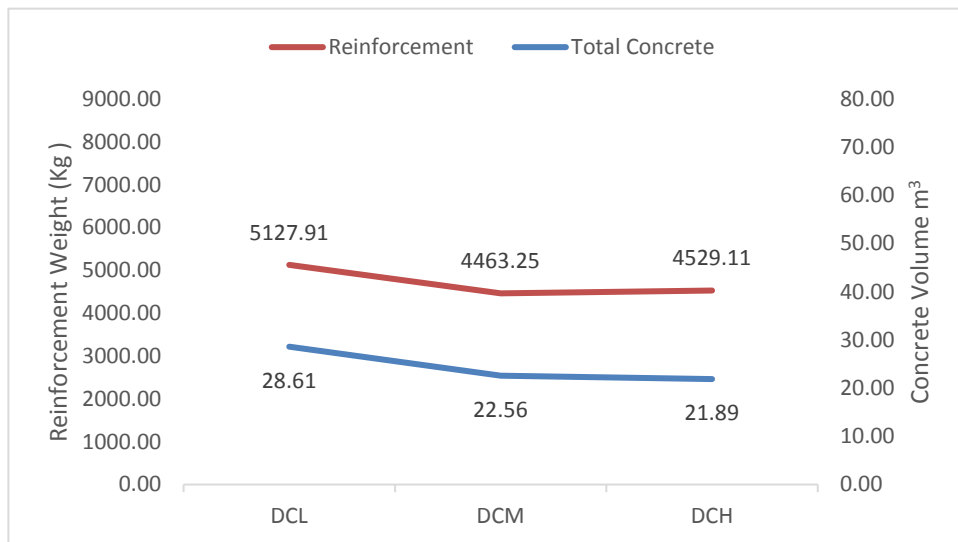


Figure 5.11 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 1.4

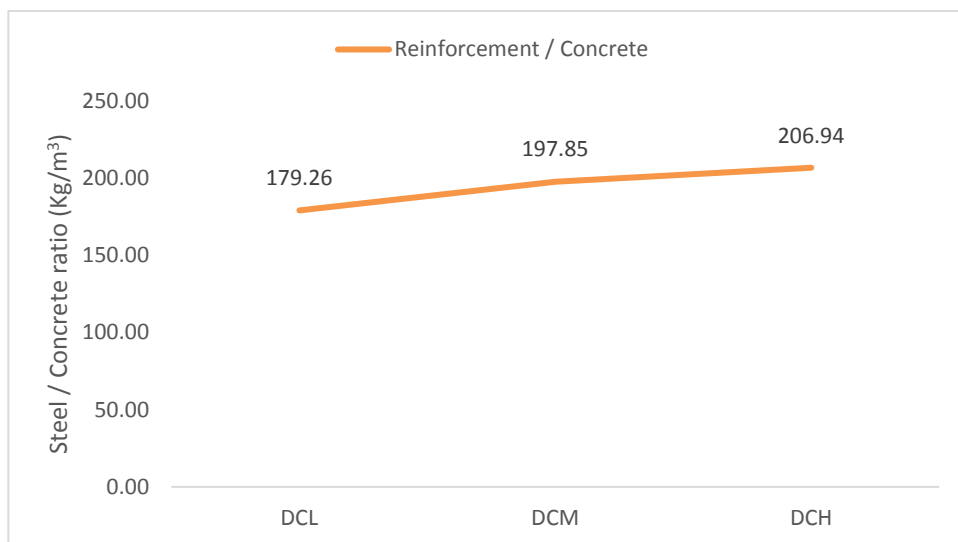


Figure 5.12 Reinforcement / concrete ratio (Kg/m³) for DC L, D M and DC H frames in Seismic zone 1.4

In seismic zone 1.4, the required quantities of concrete and reinforcement are less for medium and high ductility classes, however, reinforcement / concrete ratio is increasing, with the increase of ductility level.

Seismic Zone 1.5

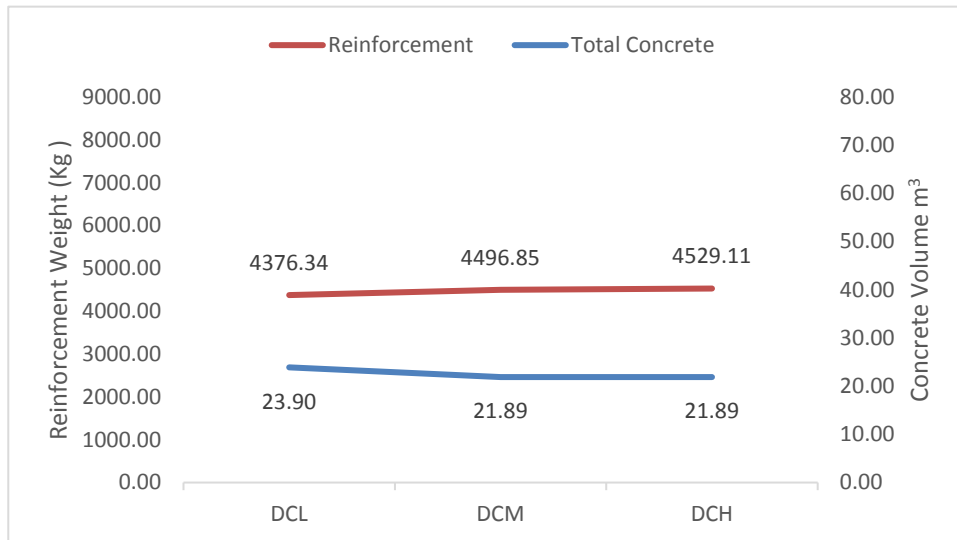


Figure 5.13 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 1.5

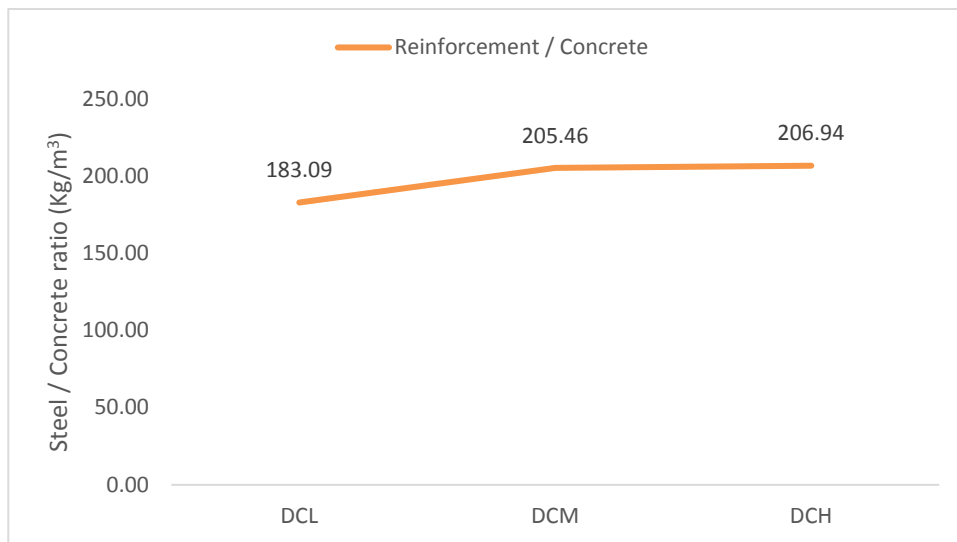


Figure 5.14 Reinforcement / concrete ratio (Kg/m³) for DC L, D M and DC H frames in Seismic zone 1.5

In seismic zone 1.5, the required quantities of concrete and reinforcement are almost the same for three ductility classes, also, the difference in reinforcement / concrete ratio is for ductility classes is small.

Seismic Zone 2.3

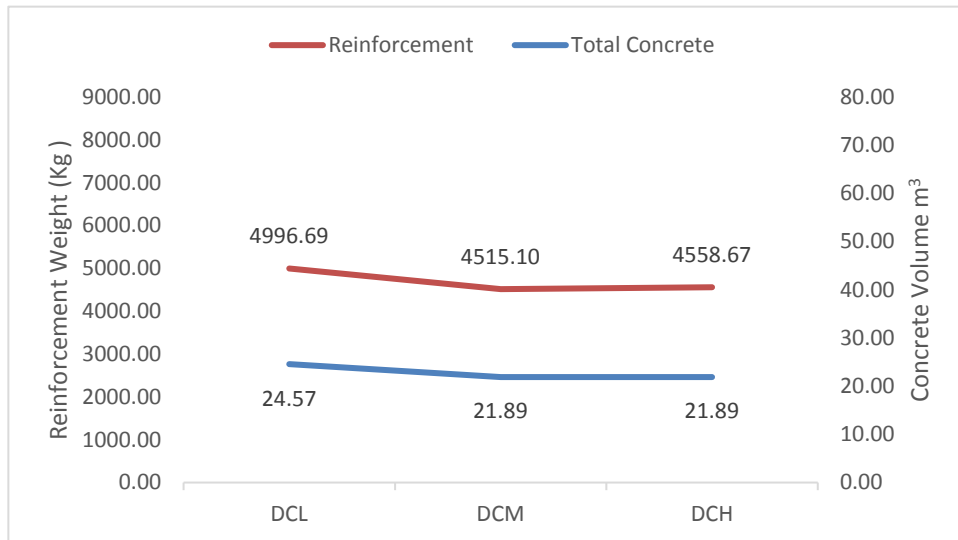


Figure 5.15 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 2.3

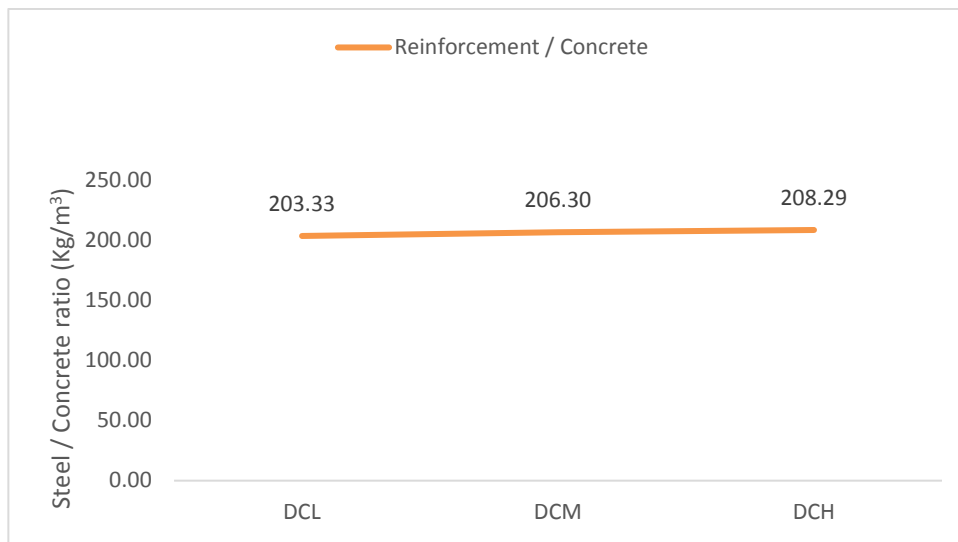


Figure 5.16 Reinforcement / concrete ratio (Kg/m³) for DC L, D M and DC H frames in Seismic zone 2.3

In seismic zone 2.3, the required quantities of concrete and reinforcement are less for medium and high ductility classes, however, reinforcement / concrete ratio is increasing, with the increase of ductility level.

Seismic Zone 2.4

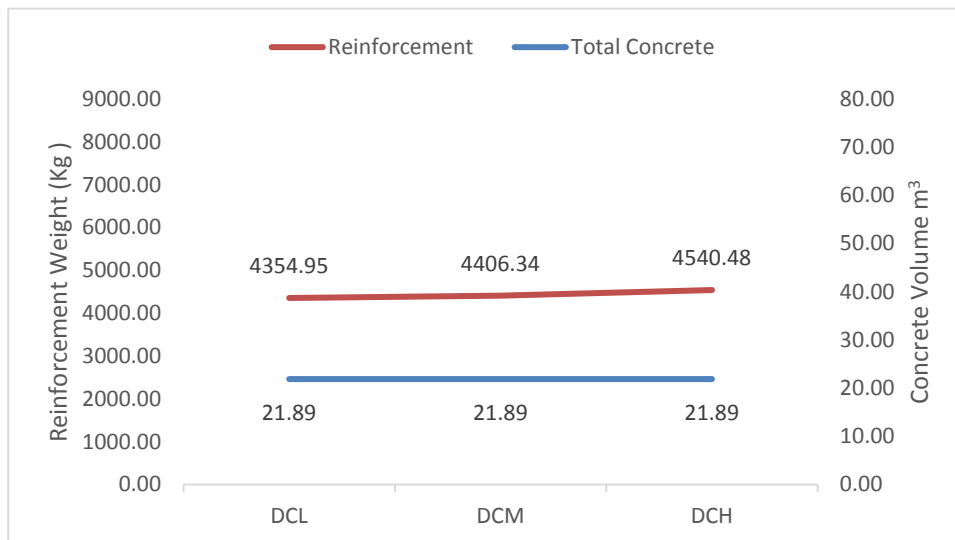


Figure 5.17 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 2.4

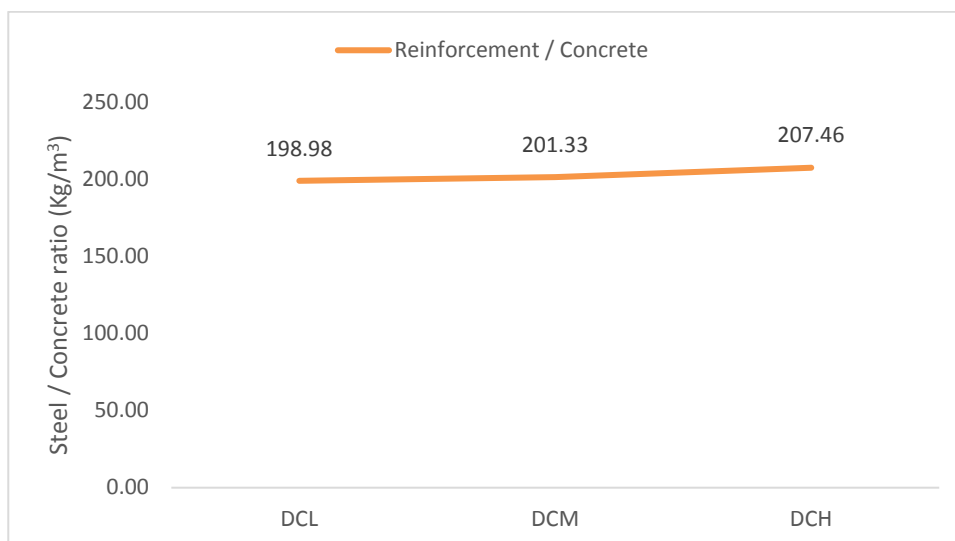


Figure 5.18 Reinforcement / concrete ratio (Kg/m³) for DC L, D M and DC H frames in Seismic zone 2.4

In seismic zone 2.4, the required quantities of concrete and reinforcement are almost the same for three ductility classes, also, the difference in reinforcement / concrete ratio for ductility classes is small.

Seismic Zone 2.5

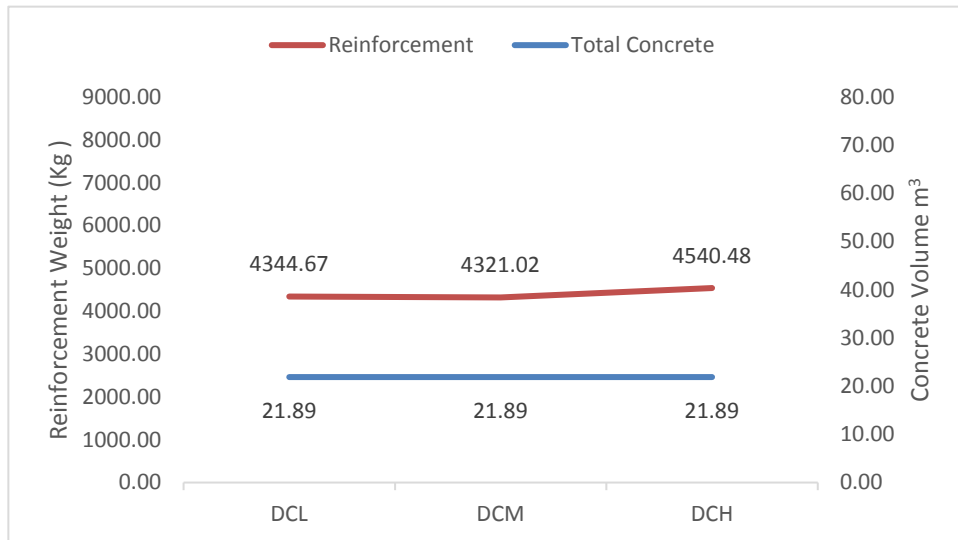


Figure 5.19 Total Concrete and reinforcement quantities for DC L, D M and DC H frames in Seismic zone 2.5

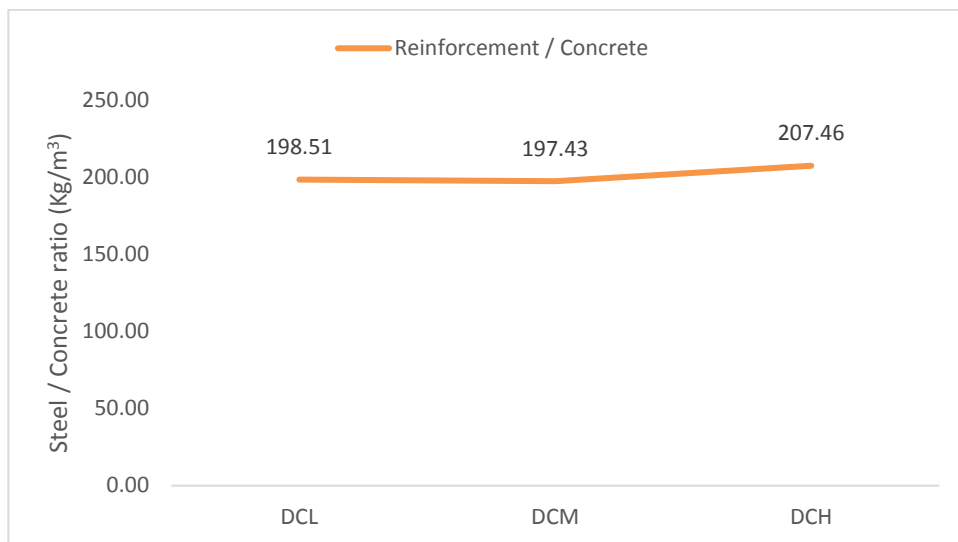


Figure 5.20 Reinforcement / concrete ratio (Kg/m³) for DC L, D M and DC H frames in Seismic zone 2.4

In seismic zone 2.5, the required quantities of concrete and reinforcement are almost the same for three ductility classes, also, the difference in reinforcement / concrete ratio is for ductility classes is small.

Figures from 5.21 to 5.24 show the results of seismic design for DC L, DC M, and DC H in seismic zones Type 1, when frame ductility class increased the total quantities of concrete and reinforcement decreased. However, reinforcement-concrete ratio is increasing with the higher ductility class, it is noticeable that the results from DC M and DC H, are so close to each other in high seismic zones, however DC H needs special criteria according to the workmanship, due to the reinforcement details, comparing with DC M.

Seismic Zones Type 1

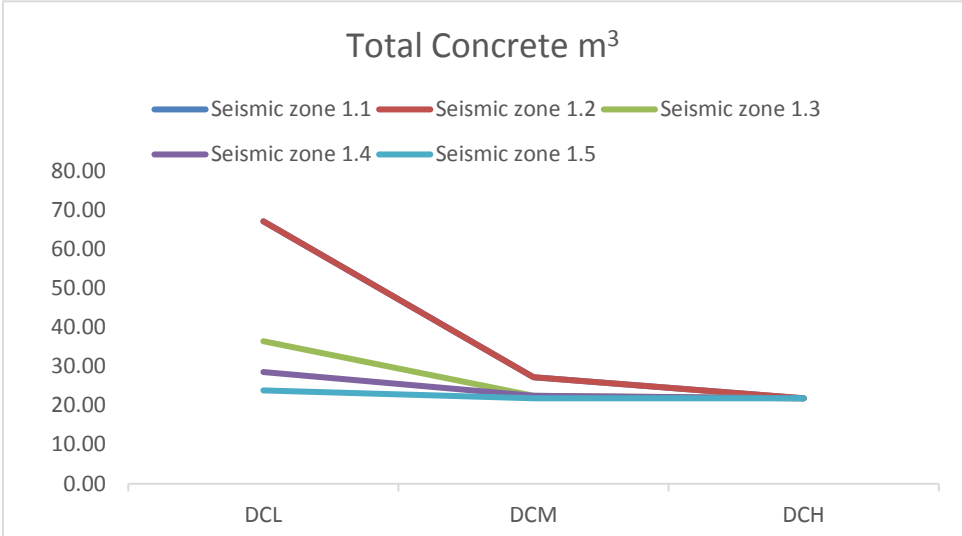


Figure 5.21 Total required concrete (m³) for DC L, DC M and DC H frames for seismic zones type 1

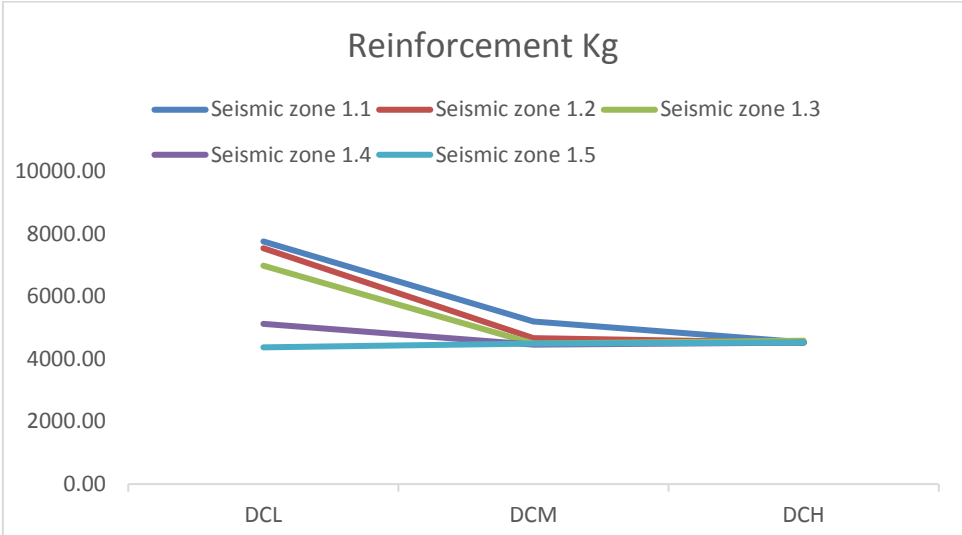


Figure 5.22 Total required reinforcement (kg) for DC L, DC M and DC H frames for seismic zones type 1

Seismic Zones Type 2

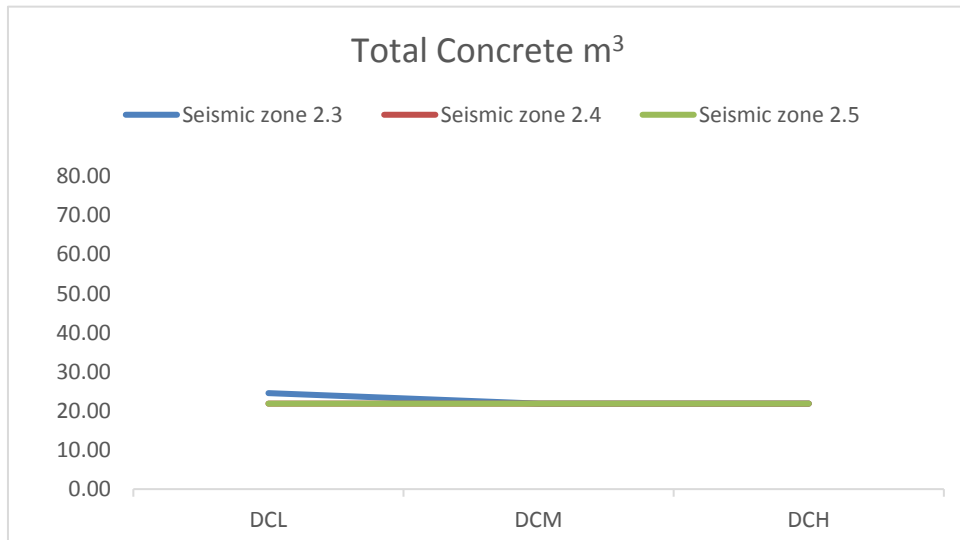


Figure 5.23 Total required concrete (m³) for DC L, DC M and DC H frames for seismic zones type 2

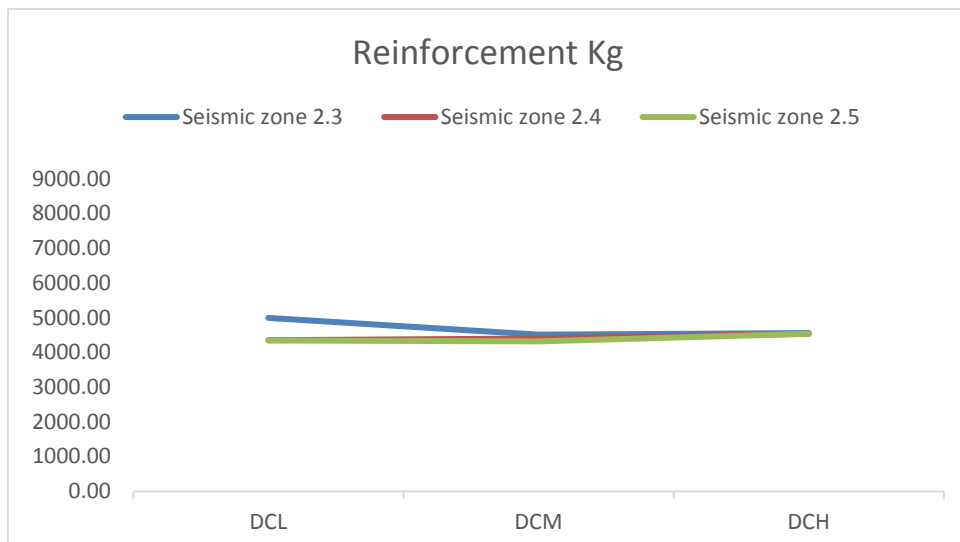


Figure 5.24 Total required reinforcement (kg) for DC L, DC M and DC H frames for seismic zones type 1

5.5. Total cost for the frame

Based on the total concrete and reinforcement for the frame referred in figure 5.1, considering material prices mentioned in table 5.1, to be able to calculate the cost of the material required for the frame for DC L, DC M and DC H in all seismic zones in Portugal. Figure from 5.25 to 5.33, show the total cost for the designed frame, workmanship was not included.

The estimated cost of seismic design for DC L, DC M, and DC H in seismic zones, is showing that, the DC L frame cost in high seismic zone like Seismic zone 1.1 and 1.2, are so

high comparing with the cost of the DC M and DC H frames, and the cost of DC M and DC H are very close to each other.

Seismic Zone 1.1

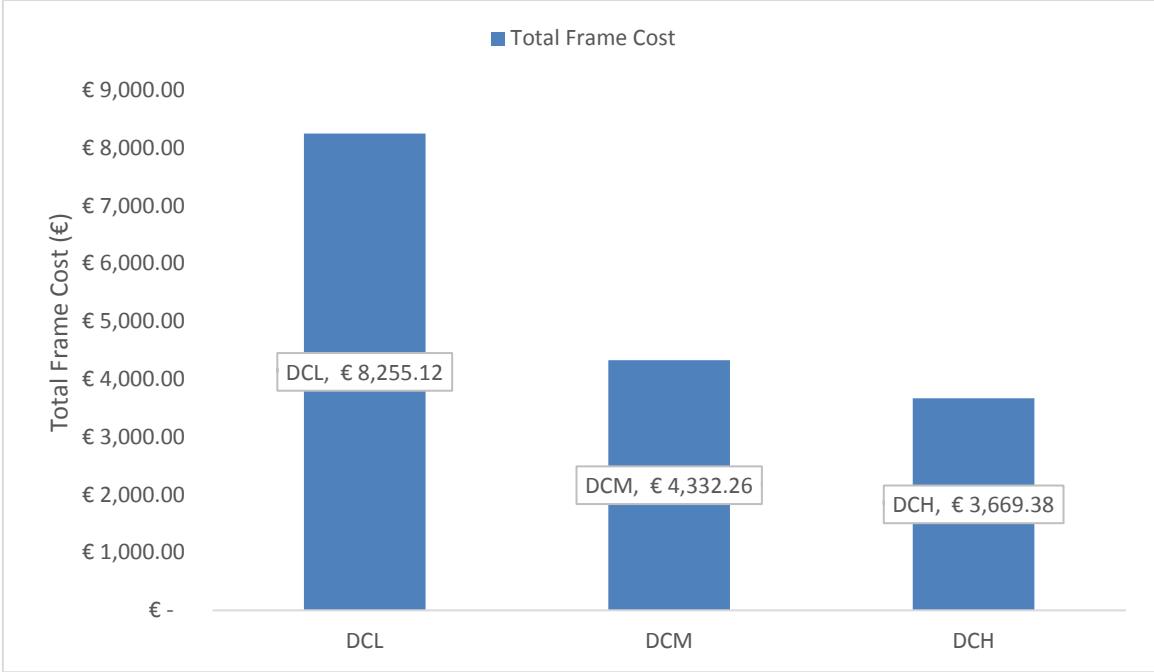


Figure 5 25 Total material costs for DC L, DC M and DC H frames in Seismic zone 1.1

Seismic Zone 1.2

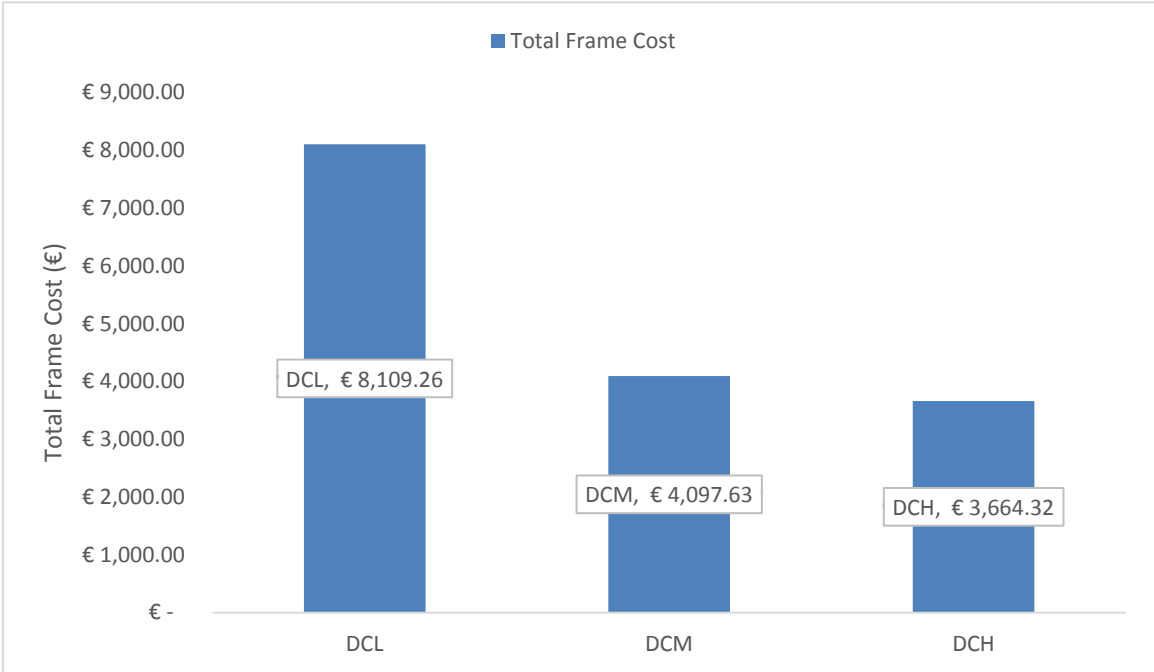


Figure 5 26 Total material costs for DC L, DC M and DC H frames in Seismic zone 1.2

Seismic Zone 1.3

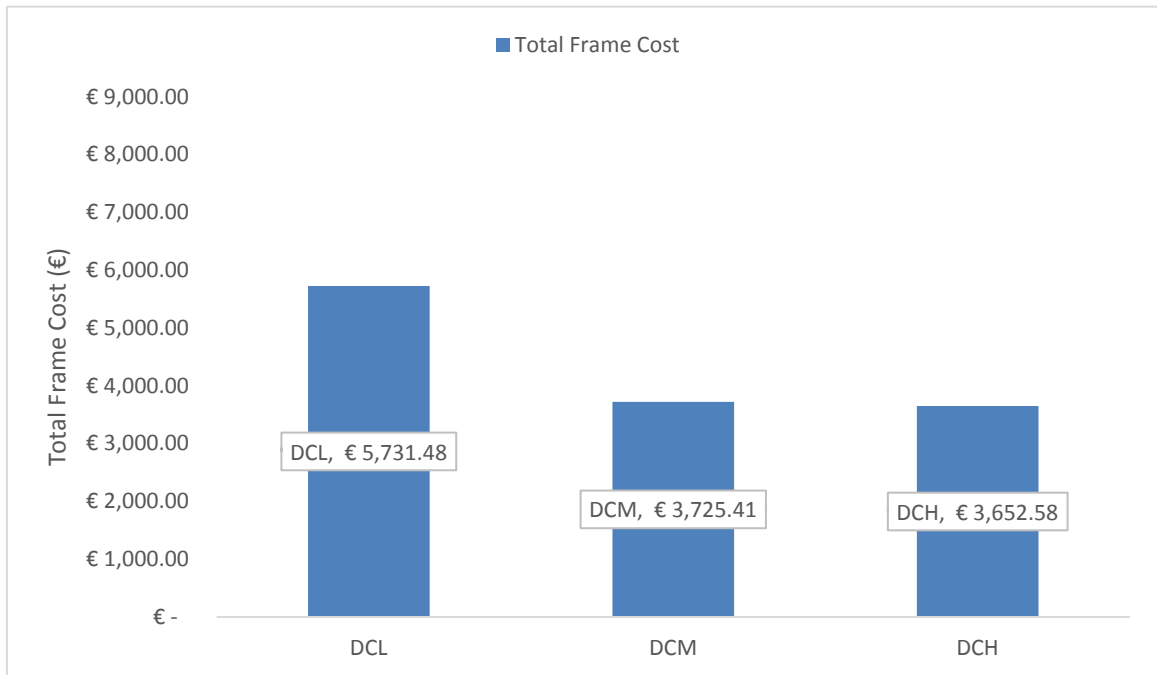


Figure 5 27 Total material costs for DC L, DC M and DC H frames in Seismic zone 1.3

Seismic Zone 1.4

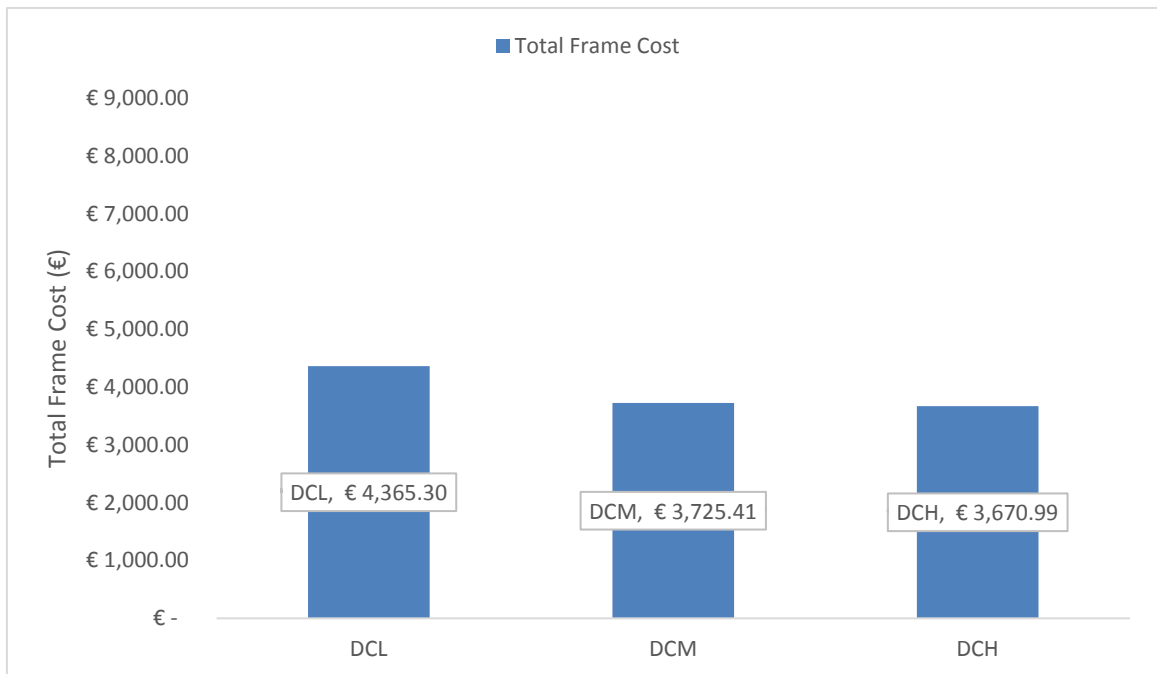


Figure 5 28 Total material costs for DC L, DC M and DC H frames in Seismic zone 1.4

Seismic Zone 1.5

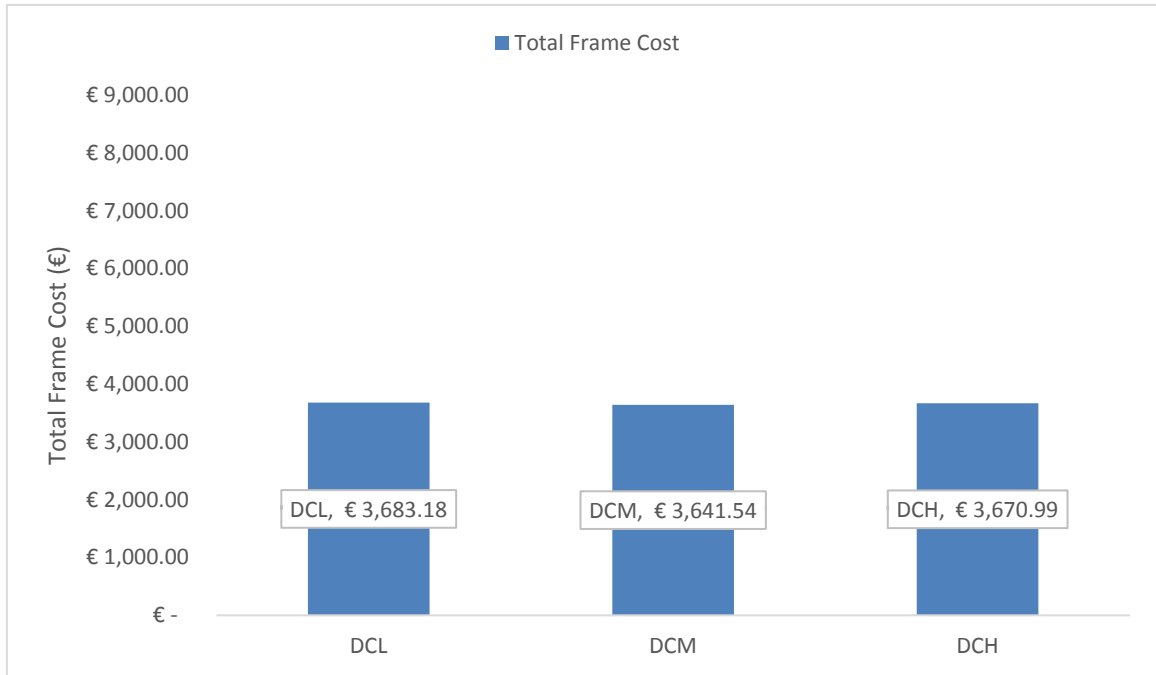


Figure 5 29 Total material costs for DC L, DC M and DC H frames in Seismic zone 1.5

Seismic Zone 2.3

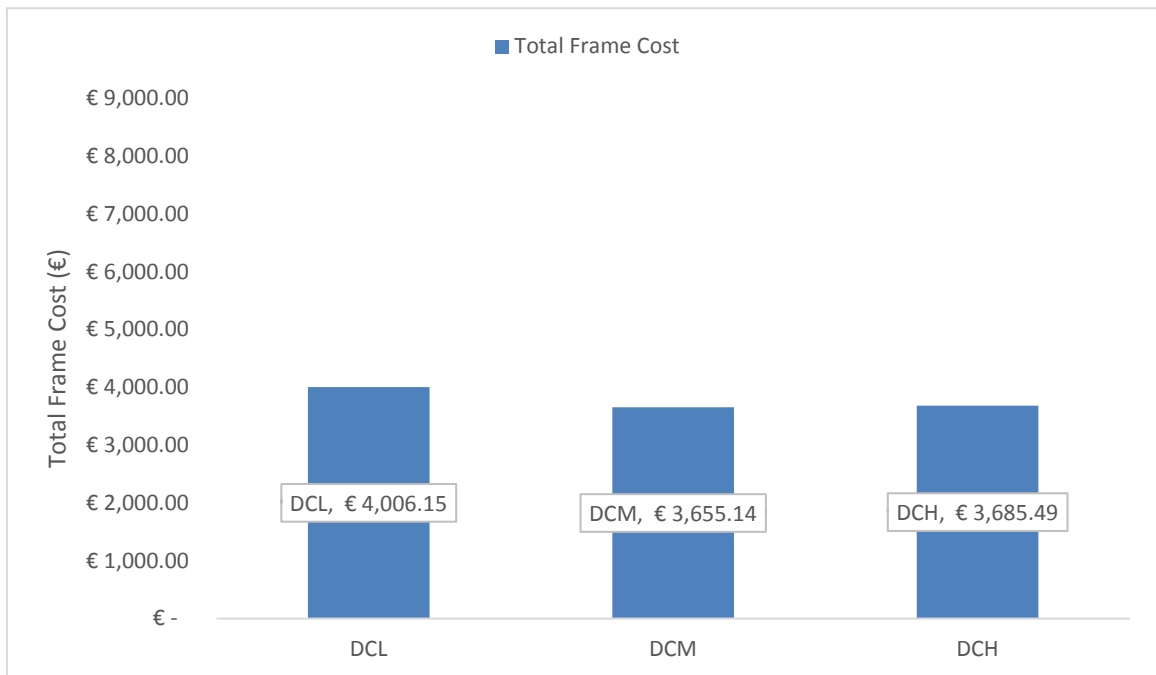


Figure 5 30 Total material costs for DC L, DC M and DC H frames in Seismic zone 2.3

Seismic Zone 2.4

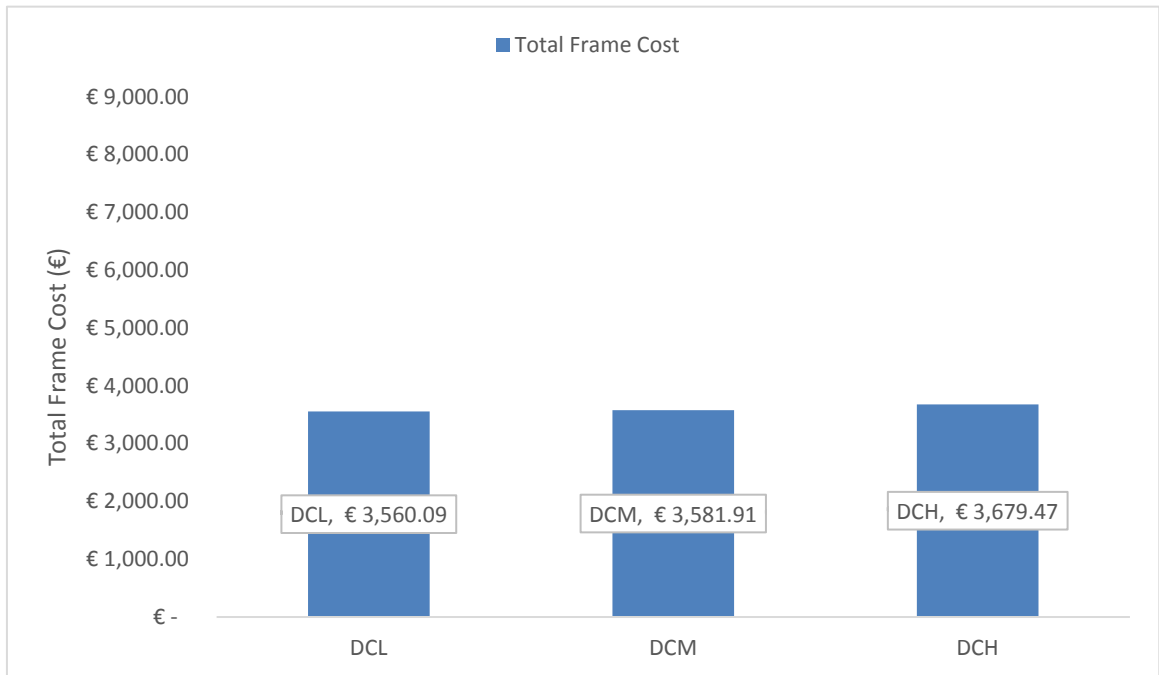


Figure 5 31 Total material costs for DC L, DC M and DC H frames in Seismic zone 2.4

Seismic Zone 2.5

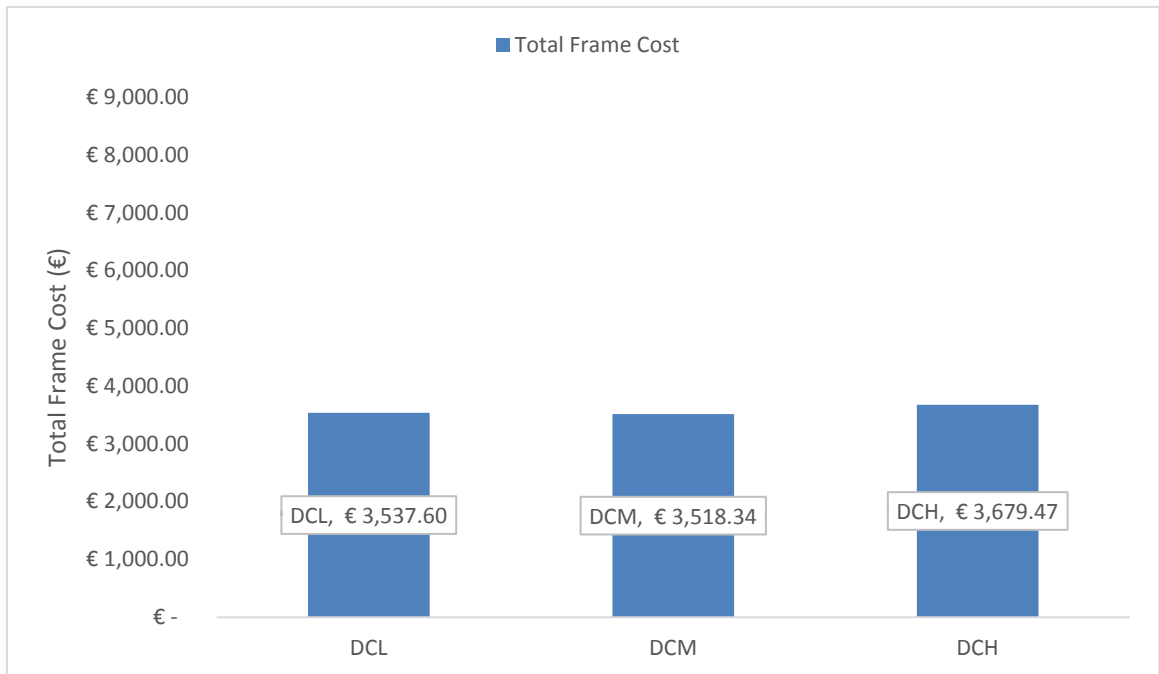


Figure 5 32 Total material costs for DC L, DC M and DC H frames in Seismic zone 2.5

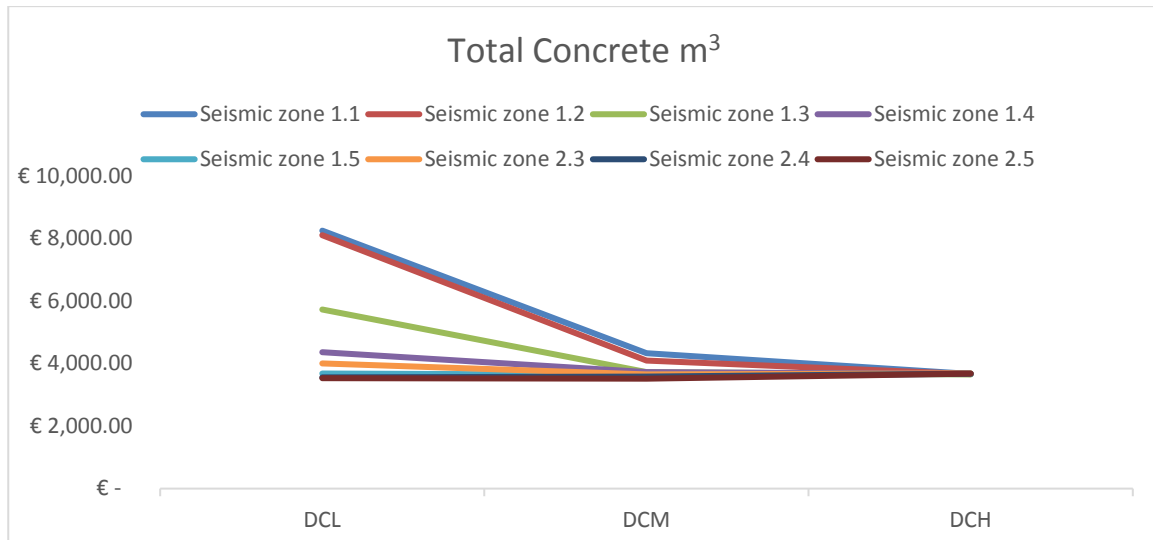


Figure 5.33 Comparison between total frame cost for DC L, DC M and DC H in different seismic zones

Several previous studies have comparatively assessed the performance of buildings designed to EC8 for different ductility classes (Fardis 2009; Booth 2012; Fardis et al. 2012) often further investigating the associated construction cost. The majority of these studies assess the inelastic behaviour of frame (Panagiotakos and Fardis 2004; Athanassiadou 2008) or both frame and dual (Kappos 1998; Kappos and Antoniadis 2007) earthquake resisting structural systems. This assessment is typically performed in-plane with only few exceptions (Anagnostopoulou et al. 2012). The main outcome of this research is that even in cases where large differences are indeed observed in material quantities and detailing for the alternative design approaches, this did not translate into remarkable differences in structural performance. Other studies (Carvalho et al. 1996) revealed that a difference was detected in the longitudinal-to-transverse steel ratio, which was found to depend primarily on the structural system, with a ratio fluctuation from 75%-25% for DCM, 60%-40% for DCH frame systems and a rather stable ratio for wall systems. However, the total material quantities (steel and concrete) required were approximately identical independently of the ductility class adopted. This observation was also verified by other researchers (Kappos 1998), which examined the effect of ductility class (DCL, DCM, DCH) on the in-plane performance of two, symmetric, ten-storey RC buildings with frame and dual structural systems. It was again concluded that the effect of ductility class on building cost is rather negligible and that the seismic performance of all buildings studied was equally satisfactory, with the anticipated exception of relatively extended column hinging and inadequate shear capacity of walls in the lower ductility systems.

In this study, the aim was to compare between the design results of three ductility classes; DC L, DC M and DC H, but in several seismic zones, the costs of frames for DC L, DC M, and DC H, are different for each seismic zone, figure 5.29 compares between the costs of frame costs for DC L, DC M and DC H, in all studied seismic zones, figure 5.29 shows that the DC L frame costs have a big change depend on the seismic zones, however this change become more less in DC M, the costs were close from seismic zone to another, in DC H, the frame cost in all seismic zone were more close, not a noticeable different between the costs in DC H.

6. Assessment of Designed Frames

After seismic design procedure of building frame, it is important to assess the behaviour of designed frame structures with a non-linear analysis procedure, there are two types of non-linear analysis, non-linear static analysis (Pushover static analysis) which is take place in this work because it is more faster and easier and have good results, and non-linear dynamic analysis (Time History non-linear analysis), which it is out of the work scoop, because of it is complexity and loss too much time for the analysis, however it is possible to get almost the same results with static pushover non-linear analysis procedure.

Performance-based earthquake engineering practice arose from the realisation that the problems in the seismic behaviour of structures had to do with the approach of designing them explicitly for life safety, Figure 6.1, thus not attempting to reduce damage in a structure, and minimise economic losses. It was suggested that performance goals should be defined in order to account for all the three previously mentioned factors, Table 6.1: structural damage, loss of life and economic losses. Figure 6.1 shows that there has been an attempt to define in a clear manner the performance objectives.

Table 6.1 Structural performance levels definition according to NEHRP Guidelines

Performance Level	Description
Operational	No significant damage has occurred to structural and non-structural components. Building is suitable for normal intended occupancy and use.
Immediate Occupancy	No significant damage has occurred to structure, which retains nearly all its pre-earthquake strength and stiffness. Nonstructural elements are secure and most would function, if utilities were available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safety	Significant damage to structural elements with substantial reduction in stiffness, however margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted.
Collapse Prevention	Substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may have occurred.

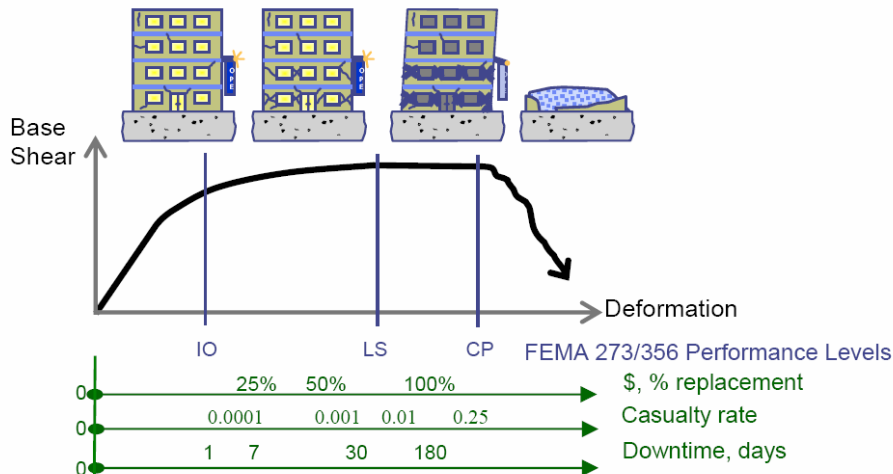


Figure 6.1 FEMA 356 Performance levels (taken from Fajfar et al. 2004)

6.1. Types of Analysis Procedures

There are four analytical procedures for design and assessment purposes recommended in the guidelines of FEMA, ATC, and EC8. These are the Linear Static Procedure, LSP, Linear Dynamic Procedure, LDP, Nonlinear Static Procedure, NSP, and the Nonlinear Dynamic Procedure, NDP, with ascending order of complexity.

1. Linear Static Analysis Procedure, LSP

The LSP procedure uses a pseudo-lateral static load pattern in order to compute the force and displacement demands on each element of the structure resulting from strong ground motion. These demands are compared with the capacities of the structural elements. The LSP however cannot be used if the structure is irregular, in terms of stiffness, strength, mass distribution, etc, if the elements have large ductility demands or the lateral force resisting system is non-orthogonal (Gupta 1998).

2. Linear Dynamic Analysis Procedure, LDP

The LDP procedure involves the computation of force and displacement demands using a modal analysis, or a time-history analysis. Usually the response spectrum analysis is favored compared to the modal analysis because it avoids the time-history analysis of a number of SDOF systems that correspond to each mode of vibration of interest. Instead the demands are computed directly by obtaining the maximum ground acceleration from the response spectrum of the ground motion or from the response spectrum of the ensemble of the ground motions.

3. Nonlinear Static Analysis Procedure or Pushover Analysis

The Nonlinear Static Analysis Procedure (NSP) procedure normally called Pushover Analysis (POA), is a technique in which a computer model of a structure is subjected to a predetermined lateral load pattern, which approximately represents the relative inertia forces generated at locations of substantial mass. The intensity of the load is increased, i.e. the

structure is ‘pushed’, and the sequence of cracks, yielding, plastic hinge formations, and the load at which failure of the various structural components occurs is recorded as function of the increasing lateral load. This incremental process continues until a predetermined displacement limit. This method will be explained in more detail later in this Chapter.

4. Nonlinear Dynamic Analysis Procedure, NDP

The non-linear time history analysis is the most sophisticated analysis method. It is usually considered to provide ‘exact’ solutions to assessment problems. The accuracy of the method depends on the modeling of the structure, the ground motion characteristics and the nonlinear material models used in the analyses, something that is true for any method of analysis.

6.2. Nonlinear Static analysis Procedure

Pushover analysis is an approximate analysis method in which the structure is subjected to monotonically increasing lateral forces with an invariant height-wise distribution until a target displacement is reached. Pushover analysis consists of a series of sequential elastic analysis, superimposed to approximate a force-displacement curve or capacity curve of the overall structure (Shinde, V and M, 2014).

There are some performance based methodology, that based on the pushover results, namely the capacity curve and with the determination of a performance point from the two estimated quantities namely; seismic demand and seismic capacity allow to check the performance of the structures. Seismic demand gives a description of the earthquake effects whereas seismic capacity shows the ability of the structure to resist these earthquake effects. POA can be done using software packages such as SAP2000, ETABS and STAAD Pro, which provides the facility to conduct the procedure. In general, POA is a technique in which the structure is subjected to a monotonic incremental lateral load which approximately represents the relative inertia forces generated at centers of masses for each storey (Leslie *et al.*, 2015).

Pushover analysis is the preferred tool for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is conceptually and computationally simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure as per (Girgin and Darılmaz, 2007).

Pushover analysis can be performed as force-controlled or displacement-controlled. In force- controlled pushover procedure, full load combination is applied as specified, that is, force- controlled procedure should be used when the load is known. Also, in force-controlled pushover procedure some numerical. Problems that affect the accuracy of results occur since target displacement may be associated with a very small positive or even a negative lateral stiffness because of the development of mechanisms and P delta effects (Shinde, V and M, 2014).

The pushover analysis is mainly based on the assumption that the response of the structure is controlled by the first mode of vibration and mode shape, or by the first few modes of vibration, and that this shape remains constant throughout the elastic and inelastic response of the structure. This provides the basis for transforming a dynamic problem to a static problem which is theoretically flawed. Furthermore, the response of a MDOF structure is related to the response of an equivalent SDOF system, ESDOF. This concept is illustrated in Figure 6.2 (Themelis, 2008).

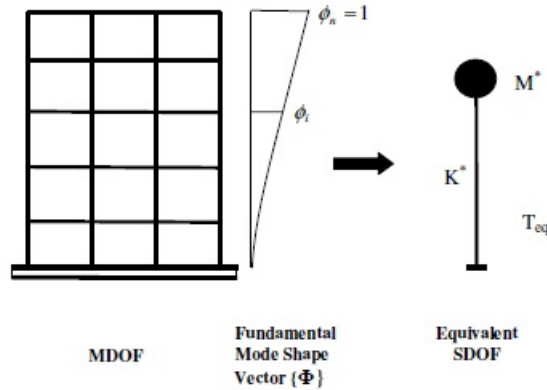


Figure 6.2 Conceptual diagram for transformation of MDOF to SDOF system (Themelis, 2008).

The earthquake induced motion of an elastic or inelastic MDOF system can be derived from its governing differential equation:

$$[M]\{\ddot{U}\} + [C]\{\dot{U}\} + \{F\} = -[M]\{1\}\ddot{u}_g \quad (6.1)$$

where $[M]$ is the mass matrix, $[C]$ is the damping matrix, $\{F\}$ is the storey force vector, $\{1\}$ is an influence vector characterising the displacements of the masses when a unit ground displacement is statically applied, and \ddot{u}_g is the ground acceleration history.

By assuming a single shape vector, $\{\Phi\}$, which is not a function of time and defining a relative displacement vector, U , of the MDOF system as $U = \{\Phi\}u_t$, where u_t denotes the roof/top displacement, the governing differential equation of the MDOF system will be transformed to:

$$[M]\{\Phi\}\ddot{u}_t + [C]\{\Phi\}\dot{u}_t + \{F\} = -[M]\{1\}\ddot{u}_g \quad (6.2)$$

If the reference displacement u^* of the SDOF system is defined as

$$U^* = \frac{\{\Phi\}^T[M]\{\Phi\}}{\{\Phi\}^T[M]\{1\}}u_t \quad (6.3)$$

Pre-multiplying equation (6.2) by $\{\Phi\}^T$ and substituting for u_t using equation (6.3) the following differential equation describes the response of the ESDOF system:

$$M^*\ddot{u}^* + C^*\dot{u}^* + F^* = -M^*\ddot{u}_g \quad (6.4)$$

where

$$M^* = \{\Phi\}^T [M] \{1\} \quad (6.5)$$

$$U^* = \{\Phi\}^T [C] \{\Phi\} \frac{\{\Phi\}^T [M] \{\Phi\}}{\{\Phi\}^T [M] \{1\}} \quad (6.6)$$

$$F^* = \{\Phi\}^T \{F\} \quad (6.7)$$

A nonlinear incremental static analysis of the MDOF structure can now be carried out from which it is possible to determine the force-deformation characteristics of the ESDOF system. The outcome of the analysis of the MDOF structure is a Base Shear, V_b , - Roof Displacement, u_t , diagram, the global force-displacement curve or capacity curve of the structure, Fig.6.3(a). This capacity curve provides valuable information about the response of the structure because it approximates how it will behave after exceeding its elastic limit. Some uncertainty exists about the post-elastic stage of the capacity curve and the information it can provide since the results are dependent on the material models used (Pankaj and Lin, 2005) and the modelling assumptions (Deierlein G., 1990; Wight J.K. *et al.*, 1997).

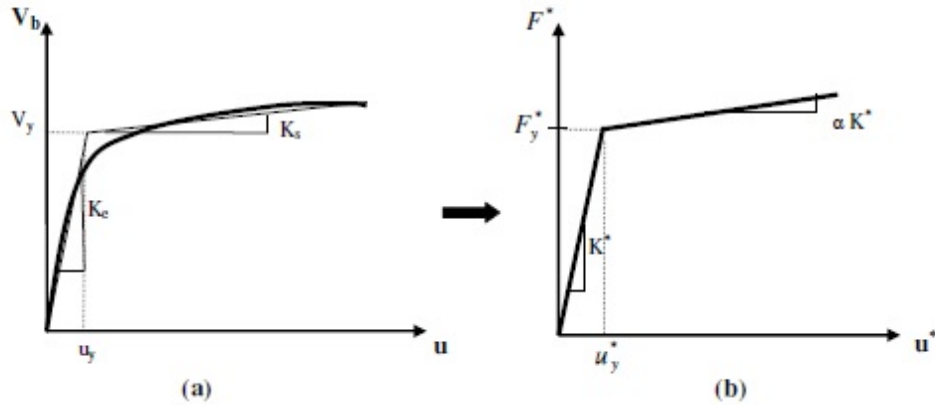


Figure 6. 3 (a) Capacity curve for MDOF structure, (b) bilinear idealization for the equivalent SDOF system.

For simplicity, the curve is idealised as bilinear from which the yield strength V_y , an effective elastic stiffness $K_e = V_y/u_y$ and a hardening/softening stiffness $K_s = \alpha K_e$ are defined. The idealised curve can then be used together with equations (6.3) and (6.7) to define the properties of the equivalent SDOF system, Figure 6.3(b).

Thus the initial period T_{eq} of the equivalent SDOF system will be:

$$T_{eq} = 2\pi \sqrt{\frac{M^*}{K^*}} \quad (6.8)$$

where K^* defines the elastic stiffness of the equivalent SDOF system and is given by:

$$K^* = \frac{F_y^*}{u_y^*} \quad (6.9)$$

The strain-hardening ratio, α , of the base shear – roof displacement relationship of the ESDOF system is taken as the same as for the MDOF structure.

The maximum displacement of the SDOF system subjected to a given ground motion can be found from either elastic or inelastic spectra or a time-history analysis. Then the corresponding displacement of the MDOF system can be estimated by re-arranging eq. 6.3 as follows:

$$u_t = \frac{\{\Phi\}^T [M] \{1\}}{\{\Phi\}^T [M] \{\Phi\}} u^* \quad (6.10)$$

The formulation of the equivalent SDOF system should not introduce much sensitivity in the results (Krawinkler et al. 1998) unless the design spectrum is sensitive to small period variations. It is also common in the pushover method that the deflected shape of the MDOF system can be represented by a single and constant shape vector $\{\Phi\}$ regardless of the level of deformation (Krawinkler et al. 1998).

The target displacement u_t is dependent on the choice of the mode shape vector $\{\Phi\}$. Previous studies of pushover analysis have shown that the first mode-shape can provide accurate predictions of the target displacement if the response of the structure is dominated by its fundamental mode ((Lawson, et al. 1994 , Fajfar et al. 1996 , Krawinkler et al.1998 , Antoniou, 2002, and many others).

6.2.1. Performance assessment based on Pushover analysis methods

The POA methods that are used can be divided into three general groups: the Conventional POA methods, the Adaptive POA methods, and the Energy-Based POA methods. Some other pushover procedures exist in the literature. The Conventional assessment procedures based on POA methods are the following:

1. Capacity Spectrum Method, CSM, (ATC 40, 1996)

The Capacity Spectrum Method, CSM, was first presented by Freeman et al. (1975) as a rapid seismic assessment tool for buildings. Subsequently, the method was accepted as a seismic design tool.

2. Improved Capacity Spectrum Method, ICSM

The ICSM method was proposed by Chopra et al. (2000) with the purpose of introducing the constant-ductility inelastic design spectra in the CSM method instead of the elastic damped spectra.

3. N2 method

The method which used in this work, section 6.2.2 is discussing the method criteria and the steps.

4. Displacement Coefficient Method, DCM

The DCM method differs to the CSM and N2 methods in the estimation of the target displacement, which does not require the conversion of the capacity curve to a capacity spectrum.

6.2.2. N2 Method

The N2 method was firstly presented by Fajfar et al. (1988) as an alternative to the CSM method. The basic idea of the N2 method stems from the Q-model developed by Saïdi et al. (1981) which in turn is based on the work of Gulkan et al. (1974). The main difference of the method with respect to the CSM method is the type of demand spectra used for the estimation of the target displacement. The steps of the method are given in the following sections.

6.2.2.1. DESCRIPTION OF THE METHOD

The basic steps of the method according to (Fajfar and Gašperšič, 1996) are:

I. Structural Data

- a) Structure (including moment-rotation relationships and low-cycle fatigue parameters β for members)
- b) Elastic (pseudo)acceleration spectrum A_e , (including peak ground velocity v_g) and either duration of strong ground motion t_D or $\int \ddot{u}_g^2 dt$

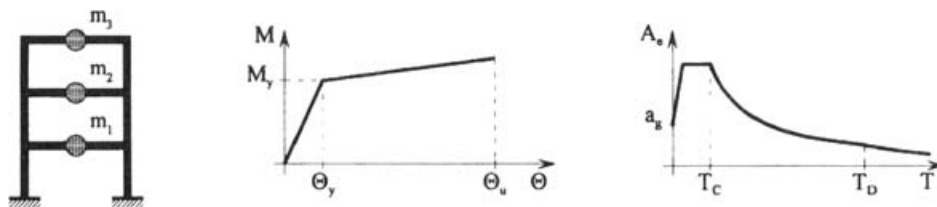


Figure 6.1 Structure data (Fajfar and Gašperšič, 1996)

II. Nonlinear static ("push-over") analysis of MDOF model under increasing lateral loading

- a) Assume displacement shape $\{\Phi\}$
- b) Determine vertical distribution of lateral loading $\{P\} = [M] \{\Phi\}$
- c) Determine base-shear (V) - top-displacement (D_t) relationship by push-over analysis

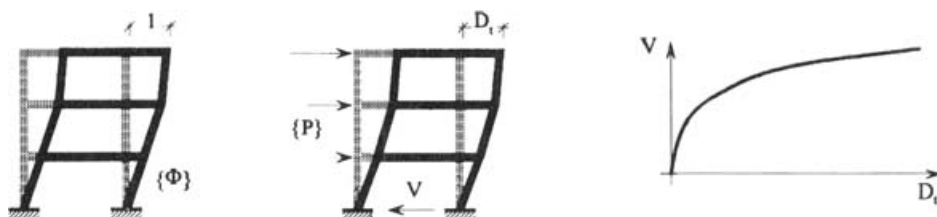


Figure 6.2 Nonlinear Static Pushover analysis of MDOF (Fajfar and Gašperšič, 1996)

III. Equivalent SDOF model

- Transform MDOF quantities to SDOF quantities
- Assume an approximate bilinear force-displacement relationship
- Determine strength F_y^* , displacement D_y^* , and period T^* of SDOF model

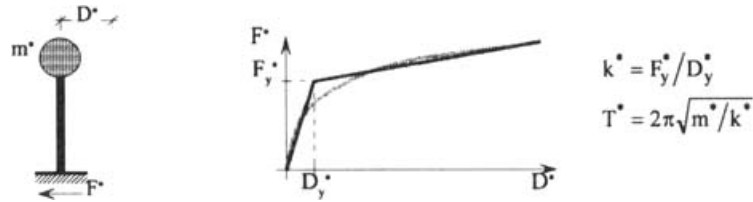
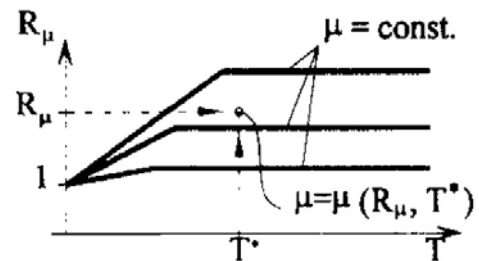


Figure 6.3 Equivalent SDOF model (Fajfar and Gašperšič, 1996)

IV. Seismic demand for the equivalent SDOF system

- Determine ductility dependent reduction factor R_μ
- Determine displacement ductility demand μ from R_μ – spectrum
- Determine displacement demand D^*
- Determine parameter γ
- Determine dissipated hysteretic energy E_H^*

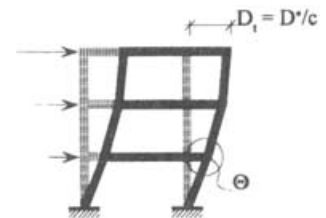


V. Global seismic demand for MDOF model

- Transform SDOF displacement to the top displacement of the MDOF model
- Transform hysteretic energy

VI. Local seismic demand (the results obtained in step II. can be used)

- Perform push-over analysis of MDOF model up to the top displacement D_t
- Determine all local quantities (rotations Θ , story drifts corresponding to D_t)
- Distribute energy demand among structural members



The results of base shear (V_b), target displacement (d_t) of the frame and the ductility (μ) of the frame which obtained from pushover analysis and N2 method, are presented in section 6.2.3.

6.2.3. Results of Pushover Analysis (N2 Method)

Using Pushover analysis as a design assessment for the frame building, to determine the base shear for the frame in each seismic zones with different ductility, with rules of N2 Method to get the Target displacement of the building and the ductility of frames.

To facilitate reading and comparing the results of pushover analyse, in the next charts, there are all the results for base shear, target displacement and frame ductility, for seismic actions in X and Y directions, for seismic zones Type 1 and Type 2.

6.2.3.1. Modelling Assumption

Sap 2000 is used to simulate the pushover analyse on the frame building, the analysis had done in the three dimension, take into the account the effect of all the building on the studied frame during the earthquake, following the steps of presented in (Surana, 2000) to get the pushover capacity curves, see figure 6.4.

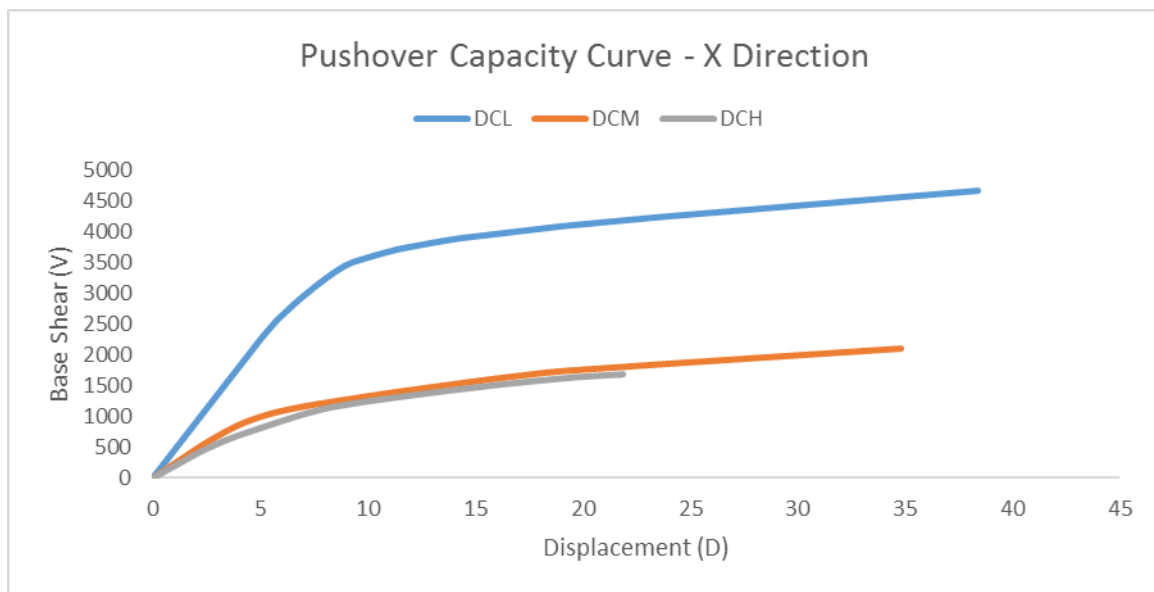


Figure 6.4 Pushover Capacity curves for DC L, DC M and DC H of seismic actions x Direction of Seismic zone 1.1

Transformed MDOF system to equivalent SDOF system, following the rules mentioned in this section 6.2, assuming an approximate bilinear force-displacement relationship (Idealized pushover capacity curve), see figure 6.5.

Determine strength F_y^* , displacement D_y^* , and period T^* of SDOF model, according to Annex b in Eurocode 8, and determine ductility dependent reduction factor R_μ and ductility μ .

$$R_\mu = \frac{S_{ae}}{S_{ay}} \quad (6.11)$$

$$S_{ay} = \frac{F_y^*}{m^*} \quad (6.12)$$

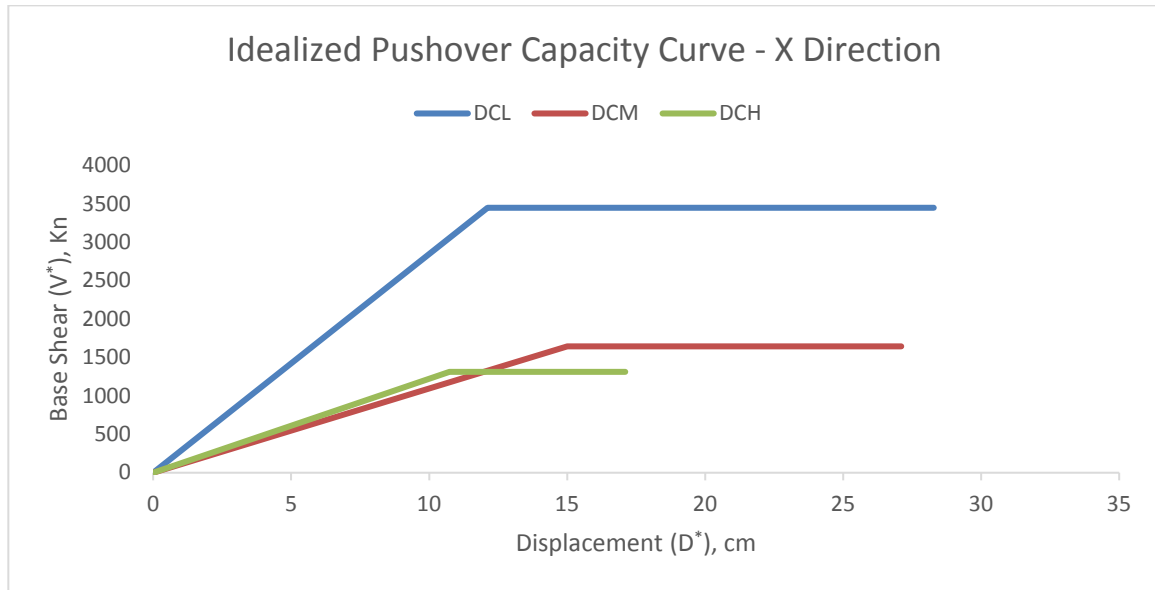


Figure 6.5 Idealized Pushover Capacity curves for DC L, DC M and DC H of seismic actions x Direction of Seismic zone 1.1

Determine demand displacement d_t^* , from the intersection of idealized bilinear pushover curve and the response spectrum capacity curve, Figure 6.6, shows the short period range where

$T^* < T_c$, and Medium and long period range where $T^* > T_c$.

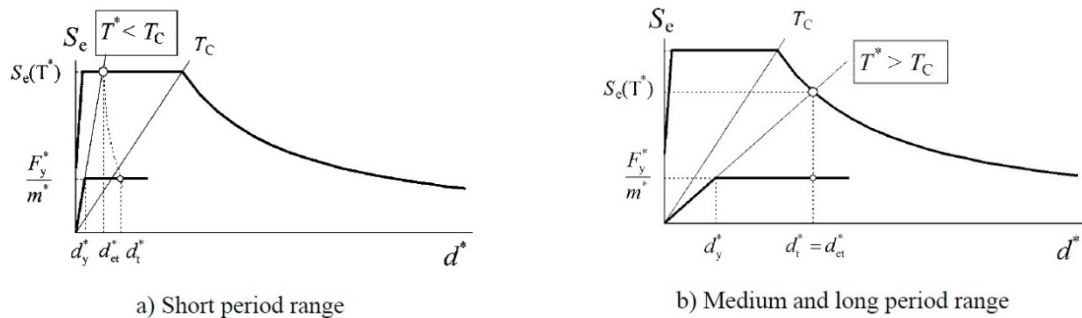


Figure 6.6 Determination of the target displacement for the equivalent SDOF system, Eurocode 8 Annex B

Transform the displacement d_t^* of SDOF to target top displacement in MDOF;

$$d_t = d_t^* * \Gamma \quad (6.12)$$

The target displacement corresponds to the control node.

6.2.3.2. Base Shear (V_b) of frame building

After getting the results of base shear (V_b), from Pushover analysis curves, figure from 6.7 to 6.10 are comparing between the base shears for the three ductility classes in each seismic zone, for seismic actions in x and y direction.

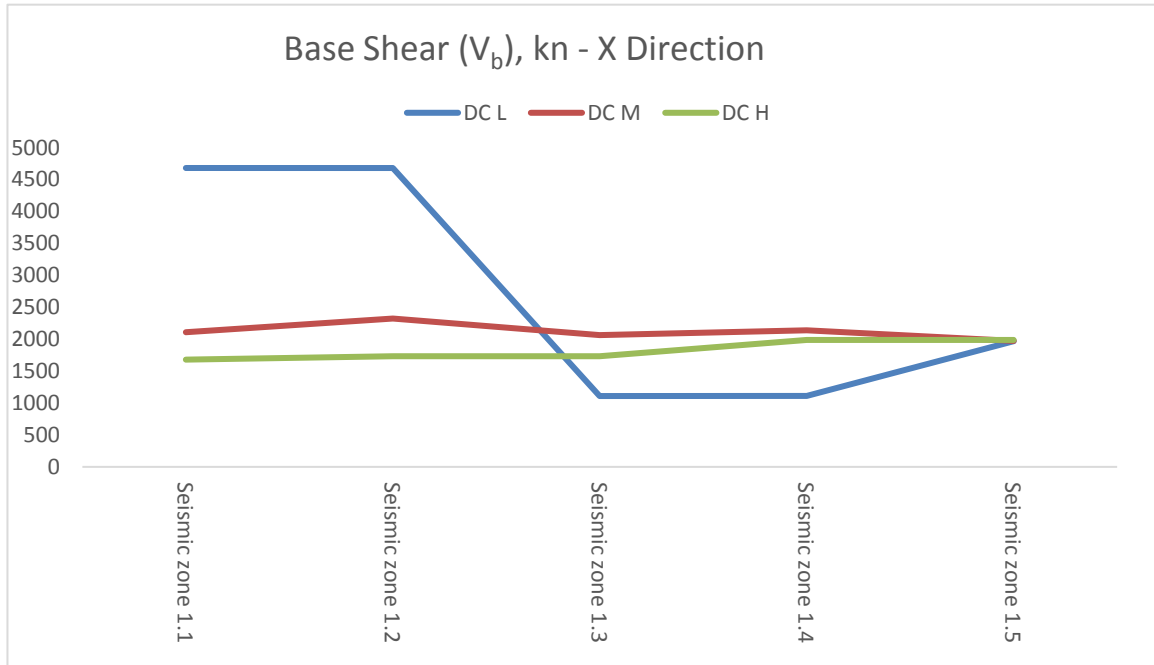


Figure 6.7 Base shear (V_b) of seismic actions in X direction, for DC L, DC M and DC H in seismic zones Type 1

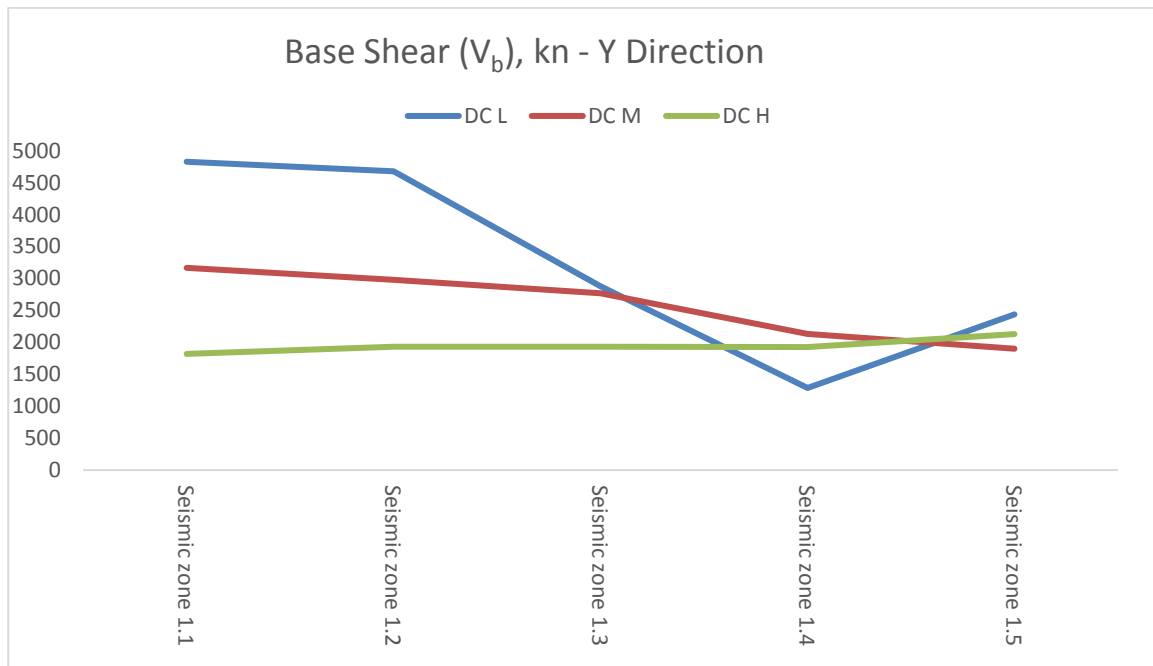


Figure 6.8 Base shear (V_b) of seismic actions in Y direction, for DC L, DC M and DC H in seismic zones Type 1

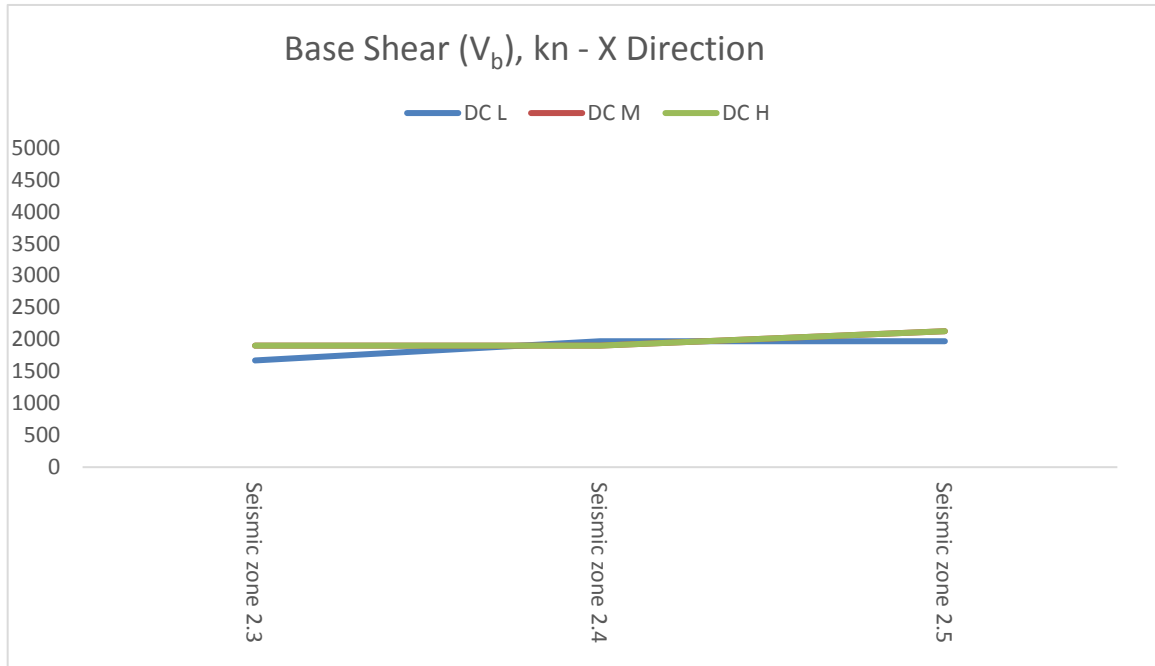


Figure 6.9 Base shear (V_b) of seismic actions in X direction, for DC L, DC M and DC H in seismic zones Type 2

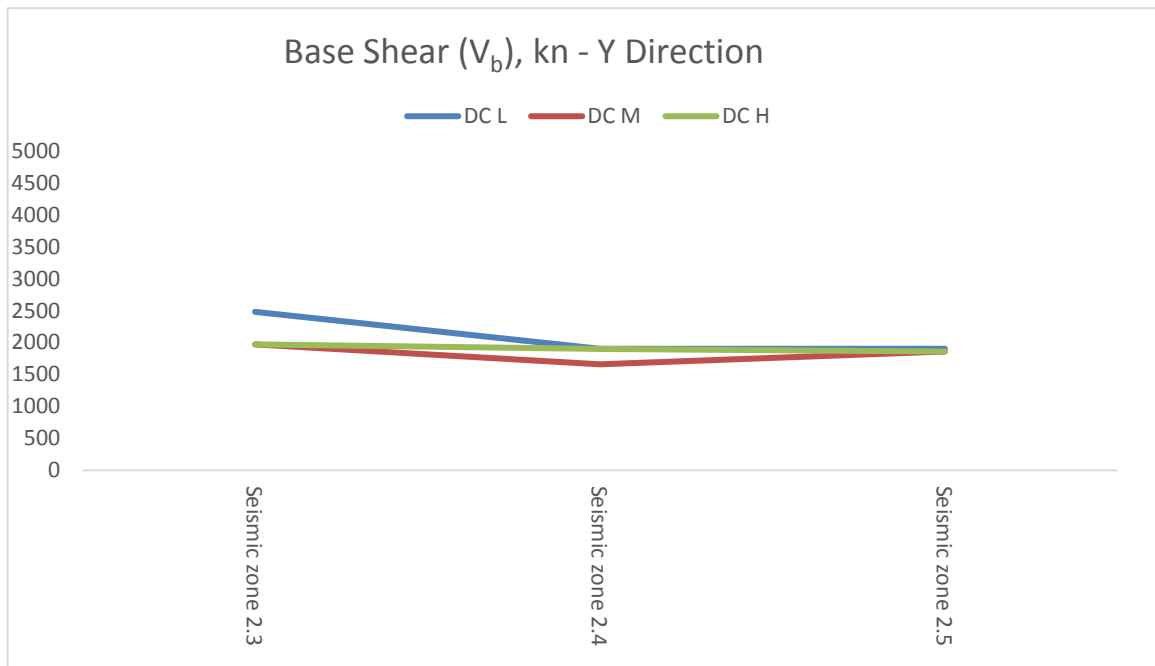


Figure 6.10 Base shear (V_b) of seismic actions in Y direction, for DC L, DC M and DC H in seismic zones Type 2

The figures above show base shear of the frame building in seismic zones type 1 and type 2, for the three ductility classes, it shows that the difference between base shears of DC M and DC H are very small, however, in high seismic zone there is a high difference for DC L comparing with DC M and DC H. In DC L curve it is noticeable that high drop in base shear after seismic zone 1.1 and 1.2, which it can be result of the difference in the cross section of columns.

6.2.3.3. Target Displacement (d_t) of frame building

N2 method was used to determine the target displacement of the building, using the idealized bilinear pushover curve and response spectrum curve. The figure from 6.11 to 6.14 are showing the target displacement of DC L, DC M and DC H, in each seismic zone for seismic actions in x and y directions.

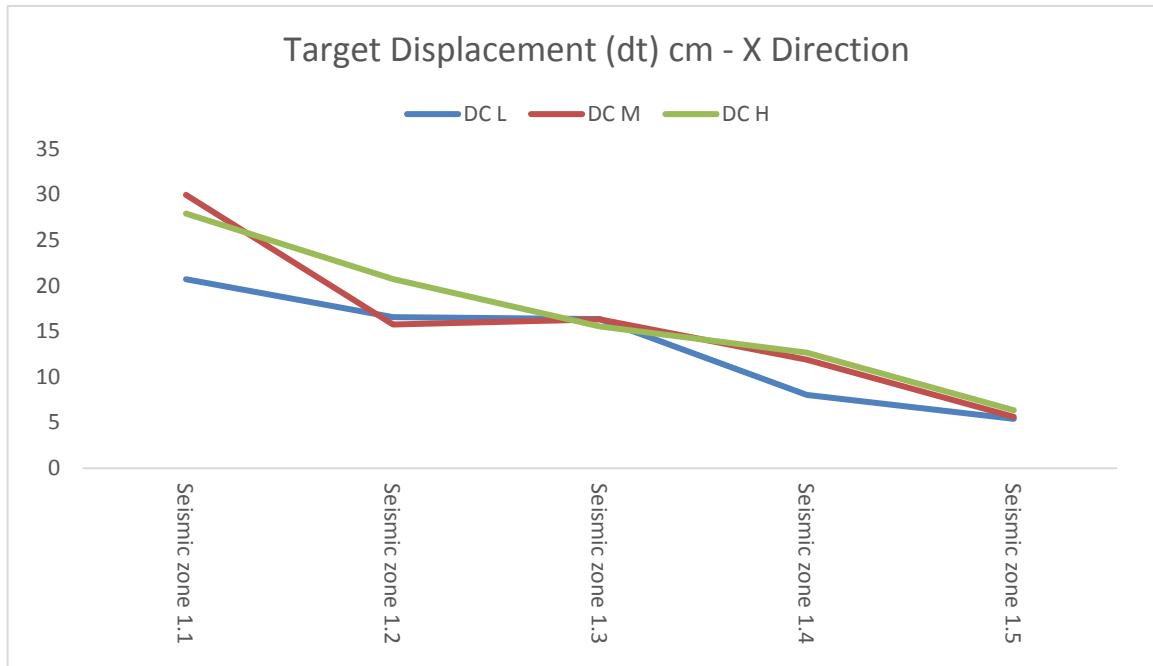


Figure 6.11 Target Displacement (d_t) of seismic actions in X direction, for DC L, DC M and DC H in seismic zones Type 1

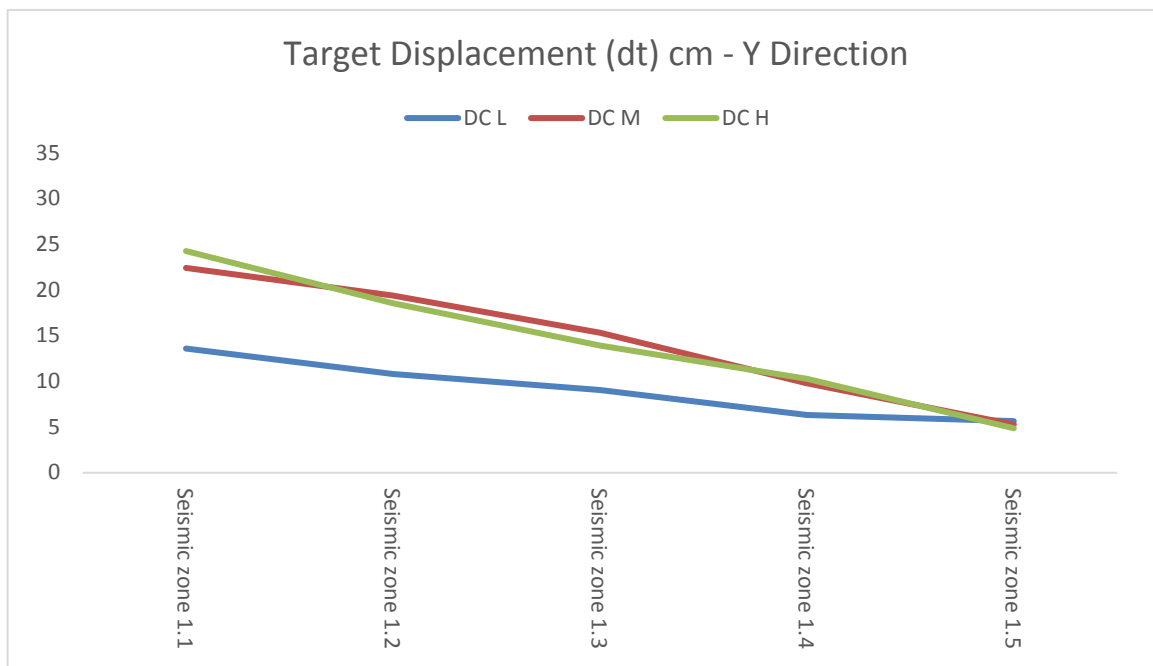


Figure 6.12 Target Displacement (d_t) of seismic actions in Y direction, for DC L, DC M and DC H in seismic zones Type 1

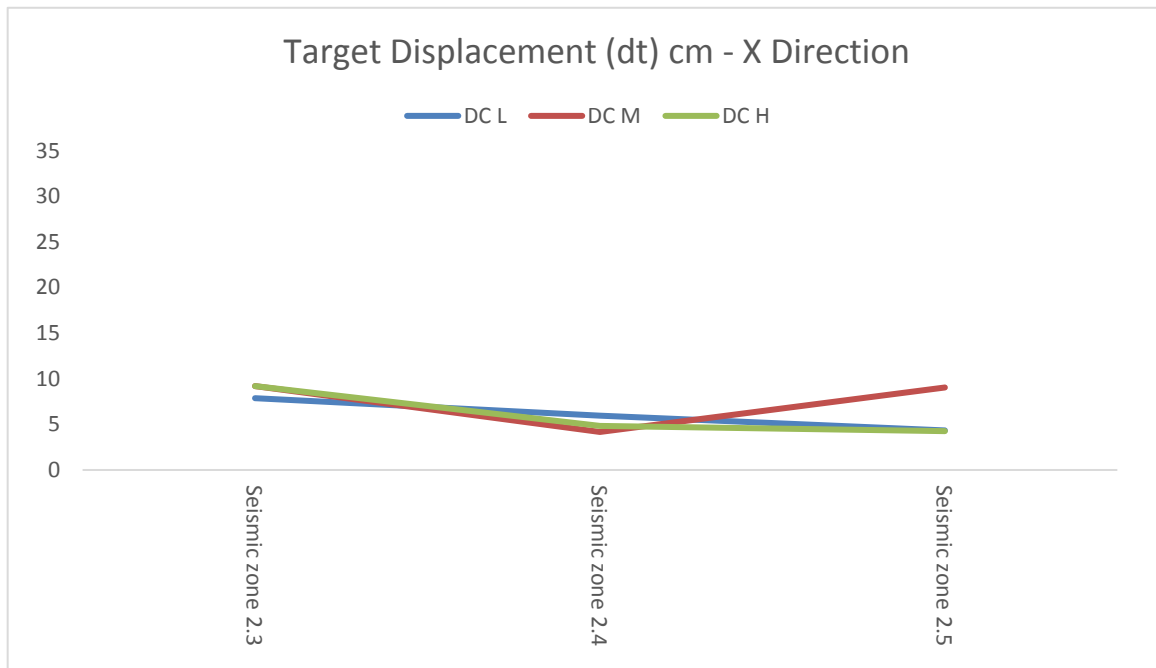


Figure 6.13 Target Displacement (d_t) of seismic actions in X direction, for DC L, DC M and DC H in Seismic zones Type 2

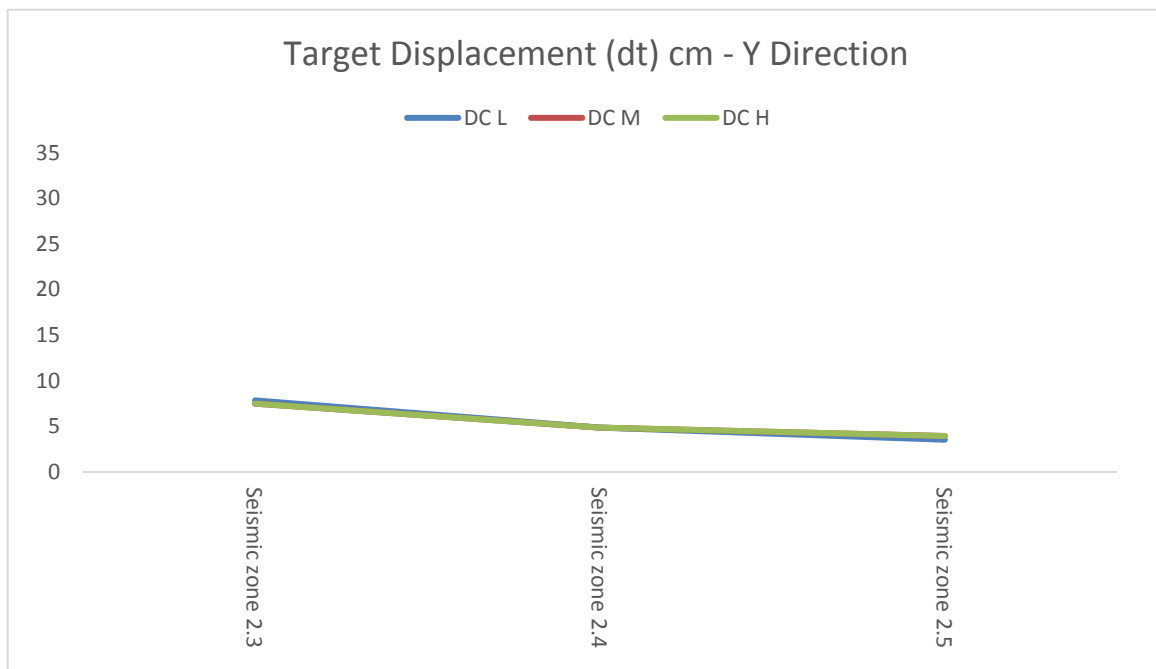


Figure 6.14 Target Displacement (d_t) of seismic actions in Y direction, for DC L, DC M and DC H in Seismic zones Type 2

Based on N2 method to calculate the target displacement d_t , of the frame building, in the charts it is noticeable that the target displacement is decrease according to the peak ground acceleration (a_g) of Seismic zones., the target displacement for DC M is so close the DC H curve.

6.2.3.4. Ductility (μ) of frame building

Ductility μ is the ability of a component or an assembly of components to deform beyond the elastic limit, and is expressed as the ratio between a maximum value of a deformation quantity and the same quantity at the yield limit state. Figure from 6.15 to 6.18, compare between the designed frames ductility in different seismic zones.

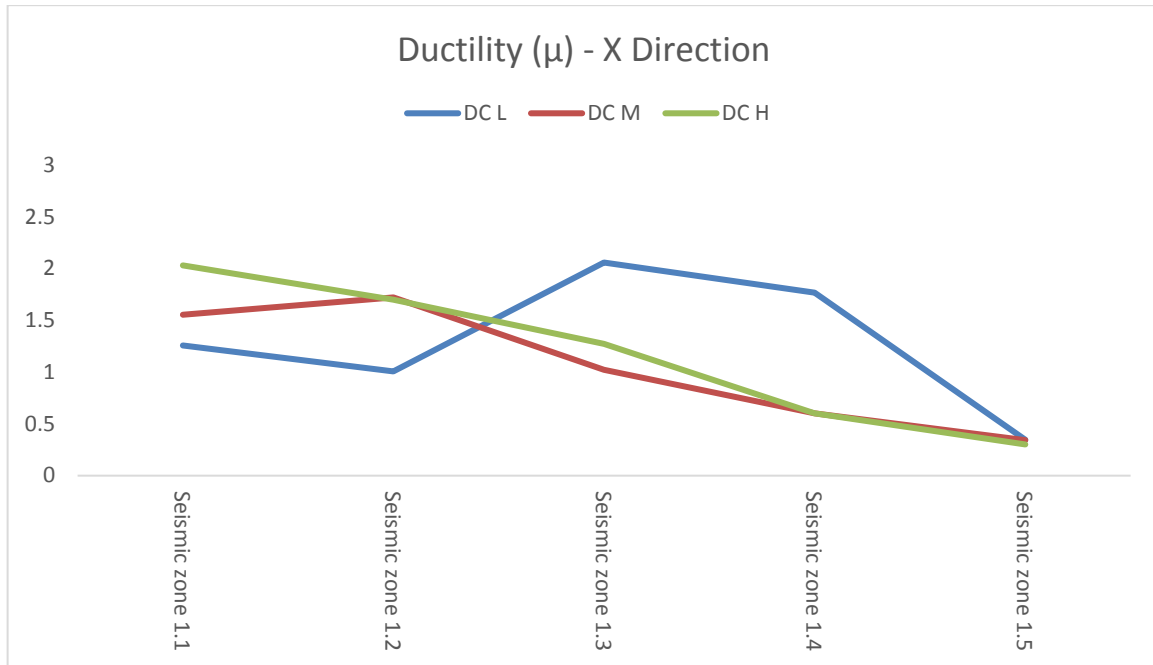


Figure 6.15 Ductility (μ) of seismic actions in X direction, for DC L, DC M and DC H in Seismic zones Type I

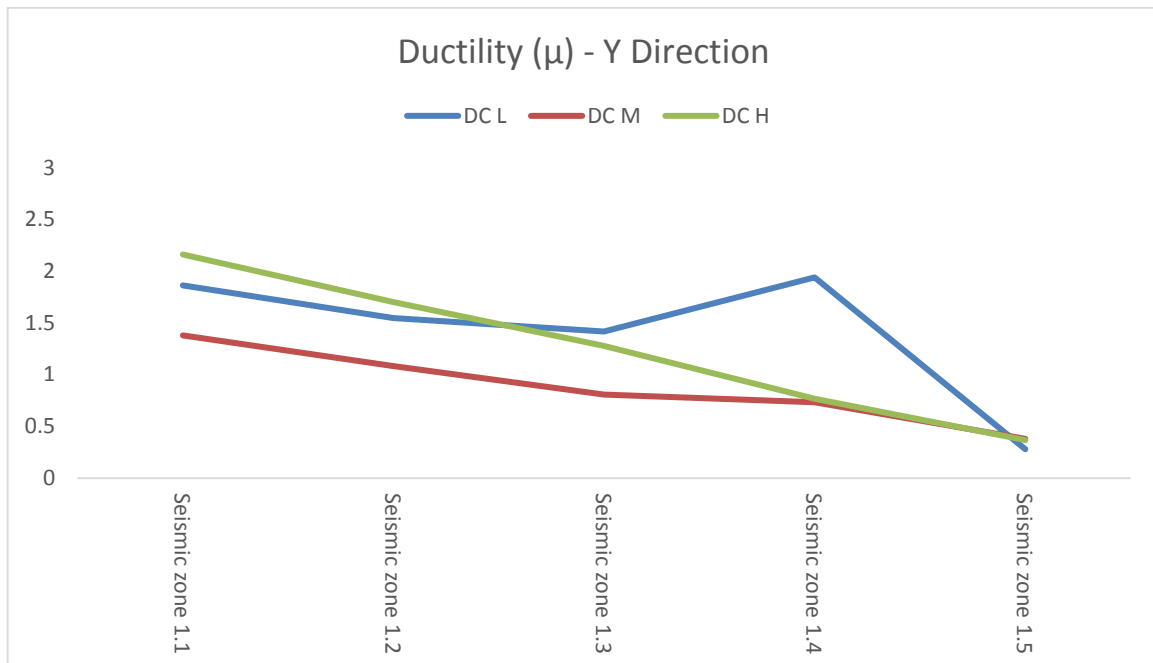


Figure 6.16 Ductility (μ) of seismic actions in Y direction, for DC L, DC M and DC H in Seismic zones Type I



Figure 6.17 Ductility (μ) of seismic actions in X direction, for DC L, DC M and DC H in Seismic zones Type 2

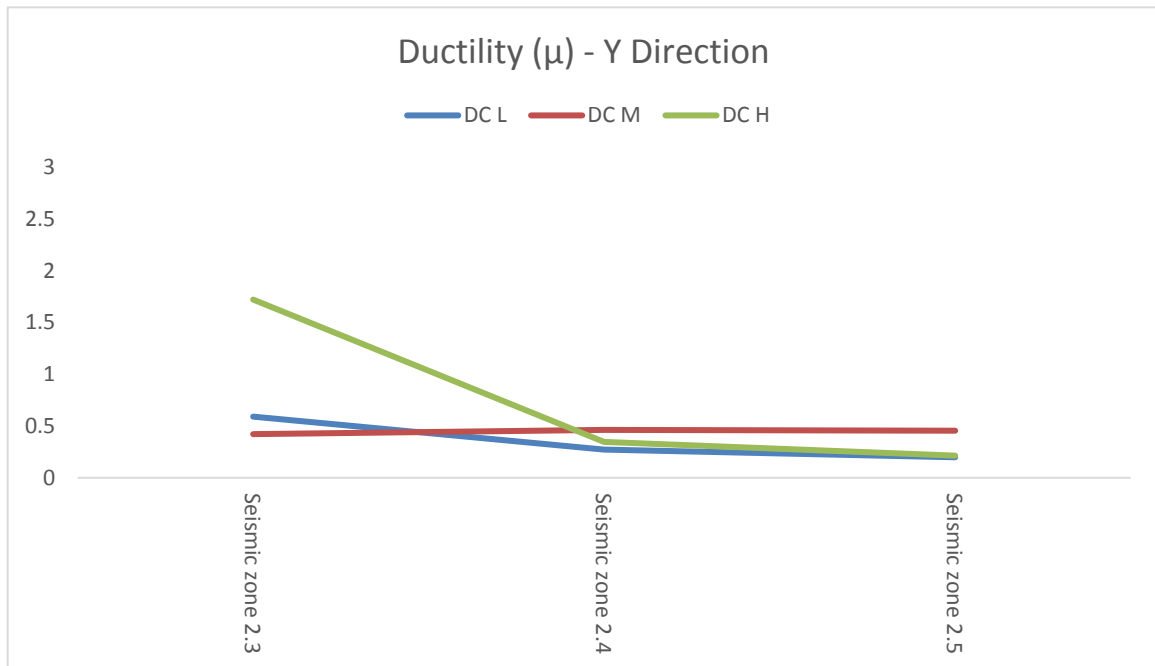


Figure 6.18 Ductility (μ) of seismic actions in Y direction, for DC L, DC M and DC H in Seismic zones Type 2

For Ductility of the frame building during the seismic actions, we could notice that for high and medium seismic zones (Seismic zone 1.1, 1.2 and 1.3) the ductility $\mu \geq 1$, but for other seismic zones the ductility decrease $\mu \leq 1$, and the explanation is for the low seismic zones have a low a_g , therefore, the target displacement d_t is less than the yield displacement d_y of the building, and according to pushover and N2 methods rules, $\mu = d_t^* / d_y^*$, therefore the result of ductility show less than 1.

Figures 6.19 to 6.20, are showing the distribution of plastic hinges for DC L, DC M and DC H frame respectively for Seismic zone 1.1, when the frame reach the target displacement, however in DC H, the frame reach the maximum displacement without reaching the determined target displacement.

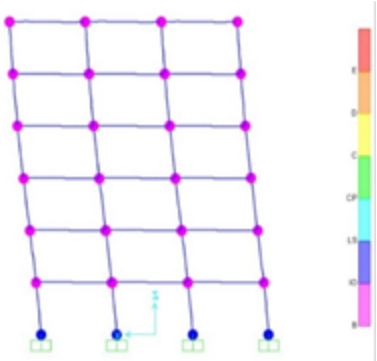


Figure 6.19 Distribution of plastic hinges on DC L frame on the target displacement in Seismic zone 1.1, $d_t = 20.72$ cm

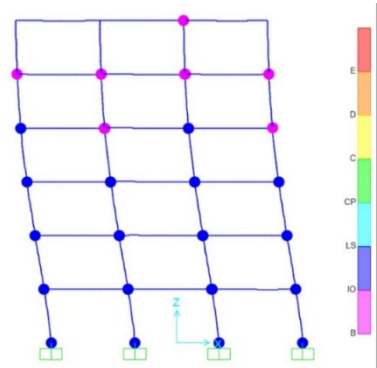


Figure 6.20 Distribution of plastic hinges on DC M frame on the target displacement in Seismic zone 1.1, $d_t = 29.9$ cm

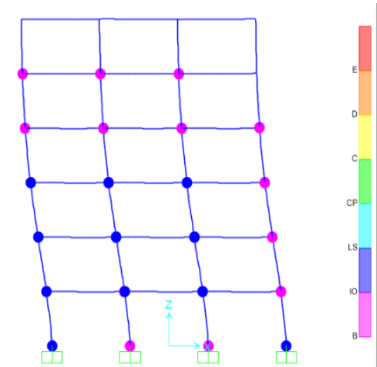


Figure 6.21 Distribution of plastic hinges on DC H frame on the maximum displacement in Seismic zone 1.1, $d = 21.9$ cm.

As we can notice plastic hinges are almost take place for the whole the frames joints for the three ductility classes in seismic zoon 1.1 when it reach the target displacement of the frame.

7. Conclusion and future research

7.1. Main conclusion

To conclude the research, the aim of this research was to clear the importance of ductility of building structure and its influence on the sizing and dimensioning of the elements, which affect on the total cost of the building, and the performance of frame building structures under the seismic actions in X and Y directions, in different seismic zones with high, medium and low ground peak acceleration (a_g), consider that the building will be in the three ductility classes mentioned in Eurocode 8, for the seismic design, for each seismic zone.

The initial design of the building runned out according to Eurocode 2, for the vertical uniaxial forces (dead and live loads), and estimate the initial cross section size for frame elements, and the required longitudinal and transverse reinforcement.

The Eurocode 8 is the European provision code that was used in this research for the seismic design of the frame building, to resist lateral seismic actions of the earthquake, to dissipate the energy of the earthquake and reduce the damages on the building and avoid building collapse during earthquakes. Dissipation of earthquake energy depends on the ductility of the designed frame, the Eurocode 8, recommends three ductility classes for the seismic design to dissipate earthquake energy (DC L, DC M and DC H). It is not clear in Eurocode 8, that how the designer can determine the suitable ductility for structure design!

In this work, a regular frame building has been designed and detailed for the three ductility classes mentioned in Eurocode 8, in all seismic zones exist in the mainland of Portugal, the results of the design used to estimate the total material concrete and reinforcement quantities, and the total cost of the frame, except the workmanship cost, was not included in the costs.

Using charts to facilitate the comparison between the quantities, reinforcement / concrete ratio, and frame costs, the charts obtained that, in high seismic zones like seismic zone 1.1 and 1.2, the quantities and costs of DC M and DC H are close to each other, however, the cost of workmanship for DC H is expected to be more higher than DC M, because of the reinforcement details of DC H, and in medium seismic zones like seismic zone 1.3, 1.4 and 2.3, frame with DC M is more economic than the other ductility classes, in low seismic zones, it is more economic always to use frame with DC L, it is more economic.

Using Nonlinear Pushover static analysis to assess the frame building, during the earthquake, and calculate the base shear (V_b), with aid of N2 method to transform the system from MDOF system to SDOF system, to be more easier to calculate the ductility (μ) and target displacement (d_t) of the building. Figure in chapter 6 are comparing the results of base shear, ductility and target displacement of the frame for three ductility classes (DC L, DC M, and DC H) in all seismic zones in the mainland of Portugal. The assessment results illustrate that, seismic zones type 1, are always more critical for the design, comparing with the seismic zones

type 2, base shear and target ductility and target displacement of DC M frames is so close to the results of DC H frames, in high seismic zones so, it could be more reasonable to design for DC M, even in high seismic zones, instead of design for DC H, because of its complexity.

7.2. Future research

The plan for future research, is to study more regular and irregular frames buildings, to study the influence of the irregularity in plane and in elevation on the results. The building will be considered not just in Portugal, will distribute in different hazard seismic zones in Europe. Using a nonlinear dynamic analysis (Time History analysis) to get more accurate results, and simulate different earthquakes. In the future research, using more sophisticated softwares, to be able to simulate the building with all the designed details, and run the nonlinear analysis to have high accurate assessment for the building. According to the future research plan it will be much more clear, if there is need to have three different ductility classes in Eurocode 8, or it could be enough to include two ductility classes, for all hazard seismic zones.

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Appendices

Appendix A: Seismic Design example for DC L Frame as per EC8

According to seismic design rules of Eurocode 8, mentioned in chapter 3, a full design example for seismic design of DC L frame ($q_o = 1.5$) located in seismic zone 1.3 ($a_g = 1.5 \text{ m}^2/\text{s}$), to explain the way that the results of chapter 5 was obtained. The example is designing frame in figure 5.1.

Design of Beams

Detailed Design of beams in Flexure

Sizing of the beam

Beam width (b_{beam})= 0.25m,

Beam depth (h_{beam}) = $L_{span}/10 = 0.5\text{m}$,

$d = 0.9 * h_{beam} = 0.9 * 0.5 = 0.45$

$z = d - d_2 = 0.9 * d = 0.9 * 0.45 = 0.405$

$$f_{yd} = \frac{f_{yk}}{1.15} = \frac{400}{1.15} = 347.8 \text{ Mpa}$$

$$f_{cd} = \frac{f_{ck}}{1.5} = \frac{25}{1.5} = 16.67 \text{ Mpa}$$

Dimensioning of the beam longitudinal reinforcement for the ULS in flexure

Figure A.1 below is showing the bending moment diagram on the beams, according to combinations a, b in section 3.4.1.1. The moment has redistributed with columns-beams joints 10% cracked, to reduce the moments on the beam-column joints, and increase the sag moment in the middle of the span.

According to equation 3.2 to calculate the longitudinal reinforcements of the beam sections;

- Beam-1, Section 1-1:

$$A_{s,1} = |M_{Ed,1}| / (f_{yd}(d - d_2))$$

$$|M_{Ed,1}| = 135.54 \text{ Kn.m}$$

$$A_{s,1} = \frac{135.54}{347.8 * 1000 * 0.405} * 10^4 = 9.62 \text{ cm}^2$$

$$\text{Use } 5 \text{ } \emptyset 16 \rightarrow A_{s,1} = 10.05 \text{ cm}^2$$

$$|M_{Rd,1}| = A_{s,1} * (f_{yd}(d - d_2))$$

$$|M_{Rd,1}| = 10.05 * 10^{-4} * (347.8 * 1000 * 0.405) = 141.58 \text{ Kn.m}$$

- Beam-1, Section 2-2:

$$A_{s,2} = |M_{Ed,2}| / (f_{yd}(d - d_2))$$

$$|M_{Ed,2}| = 94 \text{ Kn.m}$$

$$A_{s,2} = \frac{94}{347.8 * 1000 * 0.405} * 10^4 = 5.7 \text{ cm}^2$$

$$\text{Use } 4 \text{ } \emptyset 16 \rightarrow A_{s,1} = 8.04 \text{ cm}^2$$

$$|M_{Rd,2}| = A_{s,1} * (f_{yd} (d - d_2))$$

$$|M_{Rd,2}| = 8.04 * 10^{-4} * (347.8 * 1000 * 0.405) = 113.26 \text{ Kn.m}$$

- Beam-1, Section 3-3:

$$\text{Use } 6 \text{ } \emptyset 16 \rightarrow A_{s,3} = 12.06 \text{ cm}^2$$

$$|M_{Rd,3}| = 169.9 \text{ Kn.m}$$

- Beam-2, Section 4-4:

$$\text{Use } 6 \text{ } \emptyset 16 \rightarrow A_{s,4} = 12.06 \text{ cm}^2$$

$$|M_{Rd,4}| = 169.9 \text{ Kn.m}$$

- Beam-2, Section 5-5:

$$\text{Use } 3 \text{ } \emptyset 16 \rightarrow A_{s,1} = 6.03 \text{ cm}^2$$

$$|M_{Rd,5}| = 85 \text{ Kn.m}$$

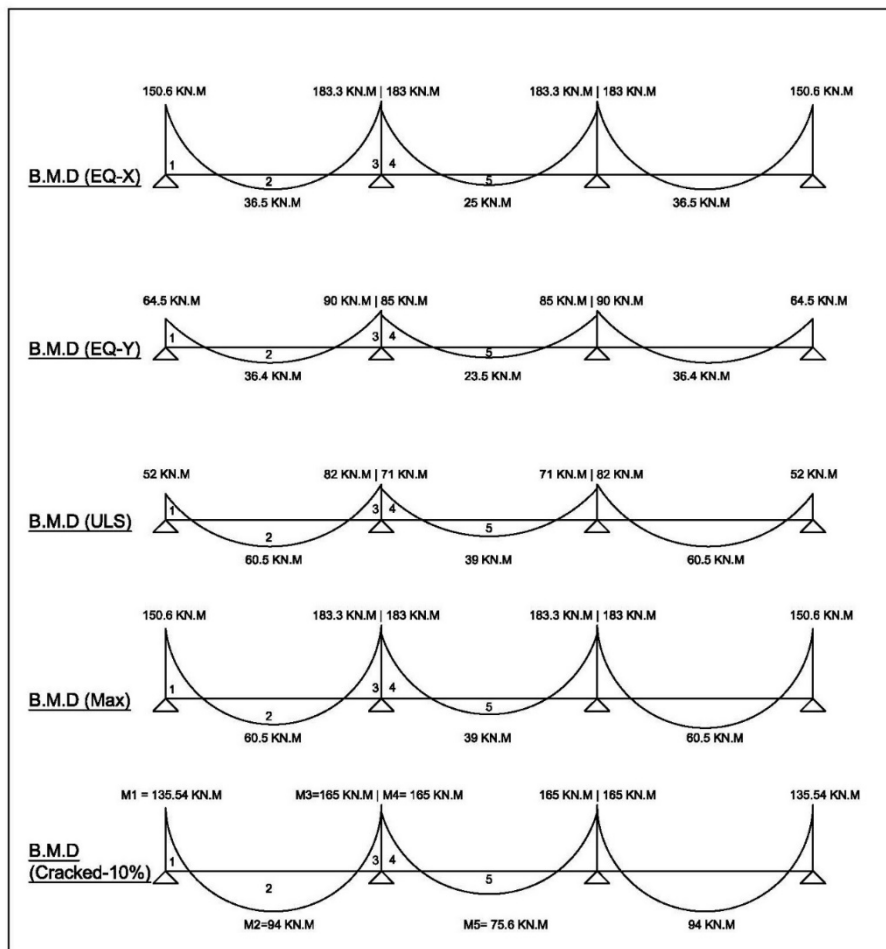


Figure A. 1 Bending moment diagrams for the beams

Detailing rules for the longitudinal reinforcement

- Critical Region Length
 - $L_{cr} = h_b = 0.5 \text{ m}$.
- Minimum steel ratio at the tensile side
 - $\rho_{min} = \max \left\{ \begin{array}{l} 0.26 * \frac{f_{ctm}}{f_{yk}} = 0.26 * \frac{2.6}{400} * 100 = 0.194 \% \\ 0.13 \% \end{array} \right. = 0.194 \%$
 - $A_{s,min} = \rho_{min} * b * d = 0.194 * 10^{-2} * 0.25 * 0.45 = 2.18 * 10^{-4} \text{ m}^2$
- Maximum A_s in critical region
 - $\rho_{max} = 0.04$
 - $A_s = \rho_{max} * b * d = 0.04 * 0.25 * 0.45 = 4.5 * 10^{-3} \text{ m}^2$
- Minimum A_s bottom bars at supports
 - $A_{s,min} = 0.25 A_{s,bottom-span}$

For Exterior Beam

 - $A_{s,min} = 0.25 A_{s,bottom-span} = 0.25 * 8.04 * 10^{-4} = 2.01 * 10^{-4} \text{ m}^2$
 - Use 2 $\emptyset 12 \rightarrow A_s = 2.26 * 10^{-4} \text{ m}^2$

For Middle Beam 2

 - $A_{s,min} = 0.25 A_{s,bottom-span} = 0.25 * 6.03 * 10^{-4} = 1.51 * 10^{-4} \text{ m}^2$
 - Use 2 $\emptyset 12 \rightarrow A_s = 2.26 * 10^{-4} \text{ m}^2$

Detailed Design of beams in Shear

$$V_{Rd,max} = 0.3 \times b_w \times Z \times \left[1 - \frac{f_{ck}}{250} \right] \times f_{cd} \times \sin 2\theta$$

$$[b_w = 0.25\text{m}], [Z = 0.405], [\theta = 45^\circ]$$

$$V_{Rd,max} = 0.3 * 0.25 * 0.405 * \left[1 - \frac{25}{250} \right] * 16.67 * 10^3 * \sin 90 = 455.6 \text{ Kn}$$

$$[V_{Ed,max} = 107 \text{ Kn}] < [V_{Rd,max} = 455.6]$$

\therefore The cross section of the beam is safe in shear.

$$V_{Rd} = \frac{A_{sh}}{s_h} * Z * f_{ywd} * \cot \theta$$

$$107 = \frac{1.01 * 10^{-4}}{s_w} * 0.405 * 348 * 10^3 * \cot 45$$

$$s_h = 0.133 \text{ m} \rightarrow^{use} s_h = 0.125 \text{ m}$$

Use 8 $\emptyset 8 / \text{m}$

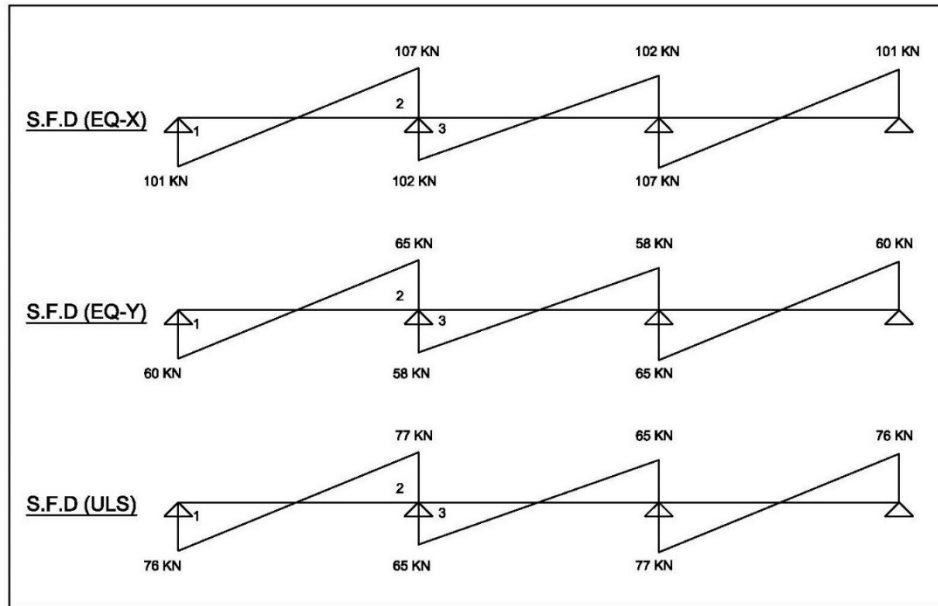


Figure A. 2 Shear force diagram for DC Lframe building

Detailing rules for the transverse reinforcement

- Outside Critical Regions

- $s_h \leq 0.75 d = 0.75 * 0.45 = 0.3375 \text{ m.}$

- $\rho_w = \frac{A_{sh}}{b_w * s_h} \geq 0.08 * \sqrt{f_{ck}} / f_{yk} \rightarrow s_h = \frac{1.01 * 10^{-4} * 400}{0.25 * 0.08 * \sqrt{25}} = 0.404 \text{ m.}$

- $s_h =$

$$\min \left\{ \begin{array}{l} 0.75 d = 0.3375 \text{ m.} \\ \frac{A_{sh}}{b_w * \rho_w} = 0.404 \text{ m.} \end{array} \right. \therefore s_h = 0.3375 \text{ m} \rightarrow^{use} s_h = 0.3333 \text{ m} = 333.33 \text{ mm}$$

- In Critical Regions

- $d_{bw} \geq 6 \text{ mm.}$

- Anchorage Length

- $L_{bd} = a_{tr} * [1 - 0.15((c_d/d_{bl}) - 1)(d_{bl}/4) f_{yd} / (2.25 f_{ctd} * a_{poor})]$

- $a_{tr} = [1 - k(n_w * A_{sw} - A_{s,t,min}) / A_s] \geq 0.7$

- $c_d = \min \left\{ \begin{array}{l} a_{cover} \\ \frac{1}{2} * [\{b - 2 * a_{cover} - d_{bl}\} / n_{bars}] \end{array} \right.$

- $f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$

- Section 1-1

For top corner reinforcement bars $5\phi 16 \rightarrow$

$$[k = 0.1], [a_{poor} = 0.7], [n_w = 10], [A_{sw} = 0.79 * 10^{-4} \text{ m}^2], [A_s = 2.01 * 10^{-4} \text{ m}^2], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4} \text{ m}^2]$$

- $c_d = \min \left\{ \begin{array}{l} a_{cover} = 0.025 + 0.01 = 0.035 \text{ m} \\ \frac{1}{2} * [\{0.25 - (2 * (0.025 + 0.008)) - (5 * 0.016)\} / 4] = 0.013 \text{ m} \end{array} \right. \rightarrow$

$$c_d = 0.013 \text{ m}$$

- $a_{tr} = 1 - \left[\frac{0.1(8*0.05*10^{-4} - 0.5025*10^{-4})}{2.01*10^{-4}} \right] = 0.826 > 0.7$.
If $a_{tr} < 0.7 \rightarrow$ Use $a_{tr} = 0.7$
- $L_{bd} = 0.826 * [1 - 0.15((0.013/0.016) - 1)] * \frac{(0.016/4) * 348*10^3}{(2.25*1.213*10^3*0.7)} = 0.618 \text{ m} \cong 0.65 \text{ m}$.
- For $L_{bd} \geq 5d_{bl}$, beyond a bend $\geq 90^\circ$, anchorage length in tension is reduced 30 %
- $L_{bd, reduced} = 0.7 * 0.618 = 0.43 \cong 45$

Using the same equation to calculate the anchorage length for all other longitudinal bars.

Design of Columns

Detailed Design of Columns in Flexure

Design of Exterior Columns for bending forces

$$N_{Ed} = 623 \text{ Kn}$$

$$b_{\text{column}} \geq b_{\text{beam}} + (2 * 0.050) = 0.25 + 0.1 = 0.35 \text{ m}$$

$$h_{\text{column}} \geq b_{\text{beam}} + (2 * 0.050) = 0.25 + 0.1 \geq 0.35 \text{ m}$$

$$a_{\text{cover}} = 0.025 \text{ m}$$

$$d_1 = a_{\text{cover}} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m}$$

$$d = h_c - d_1 = 0.35 - 0.039 = 0.311 \text{ m}$$

$$V_d = \frac{N_{Ed}}{b * d * f_{cd}} = \frac{623}{0.35 * 0.311 * \frac{25 * 10^3}{1.5}} = 0.343$$

$$M_{Rc} \geq 1.3 * M_{Rb}$$

$$M_{Rb, m} = 141.6 \text{ Kn.m} \rightarrow M_{Rc} = 1.3 * 141.6 = 184.1 \text{ Kn.m}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{184.1}{0.35 * 0.311^2 * \frac{25 * 10^3}{1.5}} = 0.326$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.311} = 0.125$$

$$V_1 = \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} + \varepsilon_{yd}} = \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) + \frac{348 * 10^3}{200 * 10^6}} = 0.54$$

$$V_2 = \delta_1 * \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} - \varepsilon_{yd}} = 0.125 * \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) - \frac{348 * 10^3}{200 * 10^6}} = 0.20$$

$$[V_2 = 0.20] \leq [V_d = 0.343] \leq [V_1 = 0.54] \rightarrow \text{Case 1}$$

$$\xi = \frac{x}{d} = \frac{V_d}{1 - (\varepsilon_{c2}/3\varepsilon_{cu2})} = \frac{0.343}{1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}} = 0.424$$

$$\begin{aligned}
(1 - \delta_1) * \omega_{1d} &= \mu_d - \xi * \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}} \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4\varepsilon_{cu2}} * \xi \right) \right] \rightarrow (1 - 0.125) * \omega_{1d} \\
&= 0.326 - 0.424 \\
&* \left[\frac{1 - 0.424}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \left(\frac{1}{2} - 0.424 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.424 \right) \right] \\
&\rightarrow \omega_{1d} = 0.246
\end{aligned}$$

$$\omega_{1d} = \frac{A_{s1}}{b*d} * \frac{f_{yd}}{f_{cd}} \rightarrow A_{s1} = 0.246 * 0.35 * 0.311 * \frac{16.667}{348} = 13.4 * 10^{-4} m^2$$

$$A_{s,total} = 2 * A_{s1} = 2 * 13.4 * 10^{-4} = 26.8 * 10^{-4} \rightarrow^{Use} 14\emptyset16 = 28.14 * 10^{-4} m^2$$

$$\rho = \frac{A_s}{b * d} * 100 = 2.3$$

$$\rho_{min} = \max \begin{cases} 0.2 \% = 2 * 10^{-3} \\ \frac{0.1N_d}{A_c f_{yd}} = 1.461 * 10^{-3} = 2 * 10^{-3} \end{cases}$$

$$[\rho_{max} = 4 \%] \geq [\rho = 2.3 \%] \geq [\rho_{min} = 0.2 \%] \therefore Ok$$

- Distribution of Longitudinal bars

$$A_{s1} = 12.06 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.231$$

$$A_{s2} = 12.06 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.231$$

$$A_{sv} = 4.02 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.077$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\varepsilon_{cu2} + \varepsilon_{yd}}{\varepsilon_{cu2} - \varepsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\varepsilon_{cu2} + \frac{\varepsilon_{c2}}{3}}{\varepsilon_{cu2} - \varepsilon_{yd}} \right)$$

$$\begin{aligned}
V_2 &= \frac{0.077}{1 - 0.125} * \left(0.125 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) \\
&+ \left(0.125 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right) = 0.146.
\end{aligned}$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\varepsilon_{cu2} - \varepsilon_{yd}}{\varepsilon_{cu2} + \varepsilon_{yd}} - \delta_1 \right) + \left(\frac{\varepsilon_{cu2} - \left(\frac{\varepsilon_{c2}}{3} \right)}{\varepsilon_{cu2} + \varepsilon_{yd}} \right)$$

$$\begin{aligned}
&= \frac{0.077}{1 - 0.125} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.125 \right) \\
&+ \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right)
\end{aligned}$$

$$V_1 = 0.56$$

$[V_2 = 0.146] \leq [V_d = 0.343] \leq [V_1 = 0.56] \rightarrow \text{Case 1}$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}}\right) + 2\omega_{vd}}$$

$$= \frac{((1 - 0.125) * (0.343)) + ((1 + 0.125) * 0.077)}{(1 - 0.125) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}\right) + (2 * 0.077)}$$

$$\xi = 0.45$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} \right]$$

$$* \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right]$$

$$M_{Rd,c} = 0.35 * 0.311^2 * \frac{25 * 10^3}{1.5}$$

$$* 0.45 \left[\frac{1 - 0.45}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * \left(\frac{1}{2} - 0.44 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.44 \right) \right]$$

$$+ \frac{(1 - 0.125)(0.231 + 0.231)}{2} + \frac{0.077}{1 - 0.125}$$

$$* \left[(0.45 - 0.125)(1 - 0.45) - \frac{1}{3} * \left(0.45 * \frac{348 * 10^3}{3.5 * 10^{-3}} \right)^2 \right]$$

$$= 118.62 \text{ Kn.m}$$

$[M_{Rd,c} = 118.62 \text{ Kn.m}] < [1.3 * M_{Rb} = 1.3 * 141.6 = 184.1 \text{ Kn.m}] \therefore \text{Not.Ok}$
 $\rightarrow \text{Increase column cross - section}$

Increase column cross-section and repeat the previous steps, until the column moment resistance becomes 30 % stronger than the beam moment resistance,

We have done many iterations, until we could achieve the equation $M_{RC} \geq 1.3 * M_{Rb}$ with the following characteristics:

$$b_c = 0.35m, h_c = 0.55m$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.55 - 0.039 = 0.411 \text{ m.}$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.411} = 0.076$$

$$v_d = 0.21$$

$$A_{s,total} = 12\emptyset16 = 24.12 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * h} * 100 = 1.25 \%$$

$[\rho_{max} = 4 \%] \geq [\rho = 1.25 \%] \leq [\rho_{min} = 0.2 \%] \therefore \text{Ok}$

Spacing along the perimeter of unrestrained bar to nearest restrained one ≤ 150 mm.

For side h = 0.55 m, $n_{bars} = 5 \text{ bars } \emptyset 16 \geq 2 \text{ bars}$

$$s_{max,unrestrainer} = \frac{[h-(2*d_1)]}{(n_{bars}-1)} = \frac{0.55-(2*0.039)}{(5-1)} = 0.118 \text{ m} = 118 \text{ mm} \leq 150 \text{ mm} \therefore$$

Ok

For side b = 0.35 m, $n_{bars} = 3 \text{ bars } \emptyset 16 + \geq 2 \text{ bars}$

$$s_{max,unrestrainer} = \frac{[b-(2*d_1)]}{(n_{bars}-1)} = \frac{0.35-(2*0.039)}{(3-1)} = 0.136 \text{ m} = 136 \text{ mm} \leq 150 \text{ mm} \therefore$$

Ok

- Distribution of Longitudinal bars

$$A_{s1} = 10.05 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.117$$

$$A_{s2} = 10.05 * 10^{-4} \rightarrow \omega_{2d} = \frac{A_{s2}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.117$$

$$A_{sv} = 4.02 * 10^{-4} \rightarrow \omega_{vd} = \frac{A_{sv}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.047$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\epsilon_{cu2} + \epsilon_{yd}}{\epsilon_{cu2} - \epsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\epsilon_{cu2} + \frac{\epsilon_{c2}}{3}}{\epsilon_{cu2} - \epsilon_{yd}} \right)$$

$$V_2 = \frac{0.047}{1 - 0.076} * \left(0.076 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) + \left(0.076 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right) = 0.083.$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\epsilon_{cu2} - \epsilon_{yd}}{\epsilon_{cu2} + \epsilon_{yd}} - \delta_1 \right) + \left(\frac{\epsilon_{cu2} - \left(\frac{\epsilon_{c2}}{3} \right)}{\epsilon_{cu2} + \epsilon_{yd}} \right)$$

$$= \frac{0.047}{1 - 0.076} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.076 \right) + \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right) = 0.55$$

$$[V_2 = 0.083] \leq [V_d = 0.21] \leq [V_1 = 0.55] \rightarrow \text{Case 1}$$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}}\right) + 2\omega_{vd}}$$

$$= \frac{(1 - 0.076) * (0.21) + ((1 + 0.076) * 0.047)}{(1 - 0.076) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}\right) + (2 * 0.047)}$$

$$\xi = 0.29.$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} \right]$$

$$* \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right]$$

$$M_{Rd,c} = 0.35 * 0.511^2 * \frac{25 * 10^3}{1.5}$$

$$* 0.29 \left[\frac{1 - 0.29}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * \left(\frac{1}{2} - 0.29 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.29 \right) \right]$$

$$+ \frac{(1 - 0.076)(0.117 + 0.117)}{2} + \frac{0.047}{1 - 0.076}$$

$$* \left[(0.29 - 0.076)(1 - 0.29) - \frac{1}{3} * \left(0.29 * \frac{348 * 10^3}{3.5 * 10^{-3}} \right)^2 \right]$$

$$= 186.2 \text{ Kn.m}$$

$$[M_{Rd,c} = 186.2] > [1.3 * M_{Rb} = 1.3 * 141.6 = 184.1 \text{ Kn.m}] \therefore \text{Ok}$$

Using CSIColumn software, to check the column for all biaxial equation for all the building, the results shown that the column was not safe and need to increase column cross-section and reinforcement,

$$b_c = 35 \text{ cm}, h_c = 80 \text{ cm}$$

$$A_s = 43.96 \text{ cm}^2 \rightarrow 14 \text{ } \emptyset \text{ 20}$$

Design of Interior Columns for bending forces

$$N_{Ed} = 1442 \text{ Kn}$$

$$b_{\text{column}} \geq b_{\text{beam}} + (2 * 0.050) = 0.25 + 0.1 = 0.35 \text{ m}$$

$$h_{\text{column}} = 0.4 \text{ m.}$$

$$a_{\text{cover}} = 0.025 \text{ m}$$

$$d_1 = a_{\text{cover}} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.40 - 0.039 = 0.361 \text{ m.}$$

$$V_d = \frac{N_{Ed}}{b * d * f_{cd}} = \frac{1442}{0.35 * 0.361 * \frac{25 * 10^3}{1.5}} = 0.685$$

$$M_{Rc} \geq 1.3 * M_{Rb}$$

$$M_{Rb,m} = 170 \text{ Kn.m} \rightarrow M_{Rc} = 1.3 * 170 = 221 \text{ Kn.m}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{211}{0.35 * 0.361^2 * \frac{25 * 10^3}{1.5}} = 0.3$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.361} = 0.108$$

$$V_1 = \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} + \varepsilon_{yd}} = \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) + \frac{348 * 10^3}{200 * 10^6}} = 0.54$$

$$V_2 = \delta_1 * \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} - \varepsilon_{yd}} = 0.108 * \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) - \frac{348 * 10^3}{200 * 10^6}} = 0.173$$

$$[V_2 = 0.173] \leq [V_d = 0.685] \geq [V_1 = 0.54] \rightarrow \text{Case 3}$$

We need to increase column cross section to get the value of V_d less than V_1 and to keep the column under the first case conditions,

Consider $[V_d = V_1 = 0.54]$

$$d = \frac{N}{f_{cd} * b * V_d} = \frac{1442}{\frac{25 * 10^3}{1.5} * 0.35 * (0.54)} = 0.457 \text{ m}$$

$$h_c = d + d_1 = 0.457 + 0.039 = 0.496 \text{ m} \cong 0.50 \text{ m}$$

$$d = h_c - d_1 = 0.50 - 0.039 = 0.461 \text{ m}$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.461} = 0.084$$

$$V_d = \frac{N_{Ed}}{b * d * f_{cd}} = \frac{1442}{0.35 * 0.461 * \frac{25 * 10^3}{1.5}} = 0.536$$

$$V_1 = 0.54, V_2 = 0.136$$

$$[V_2 = 0.136] \leq [V_d = 0.536] \leq [V_1 = 0.54] \rightarrow \text{Case 1}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{221}{0.35 * 0.461^2 * \frac{25 * 10^3}{1.5}} = 0.18$$

$$\xi = \frac{x}{d} = \frac{V_d}{1 - (\varepsilon_{c2}/3\varepsilon_{cu2})} = \frac{0.536}{1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}} = 0.66$$

$$(1 - \delta_1) * \omega_{1d} = \mu_d - \xi * \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}} \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4\varepsilon_{cu2}} * \xi \right) \right]$$

$$(1 - 0.084) * \omega_{1d} = 0.178 - 0.66 * \left[\frac{1 - 0.66}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \left(\frac{1}{2} - 0.66 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.66 \right) \right]$$

$$\omega_{1d} = 0.063 \rightarrow A_{s1} = 5.03 * 10^{-4} \text{ m}^2$$

$$A_{s,total} = 2 * A_{s1} = 2 * 8.8 * 10^{-4} \text{ m}^2 = 17.6 * 10^{-4} \text{ m}^2 \rightarrow \text{Use } 10 \text{ } \emptyset 16$$

$$A_{s,total} = 20.10 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * h} * 100 = \frac{20.10}{0.35 * 0.5} = 1.15 \%$$

$$\rho_{min} = \max \left\{ \begin{array}{l} 0.1N_d \\ \frac{0.2\% = 2 * 10^{-3}}{A_c f_{yd}} = \frac{0.1 * 1442}{0.35 * 0.5 * 348 * 1000} \end{array} \right. = 2.36 * 10^{-3} = 2.36 * 10^{-3} = 0.236 \%$$

$$[\rho_{max} = 4\%] \geq [\rho = 1.15\%] \geq [\rho_{min} = 0.236\%]$$

Spacing along the perimeter of unrestrained bar to nearest restrained one ≤ 150 mm.

For side h = 0.50 m, $n_{bars} = 4 \text{ bars } \emptyset 16 \geq 2 \text{ bars}$

$$s_{max,unrestrainer} = \frac{[h - (2 * d_1)]}{(n_{bars} - 1)} = \frac{0.50 - (2 * 0.039)}{(4 - 1)} = 0.14 \text{ m} = 140 \text{ mm} \leq 150 \text{ mm}$$

$\therefore Ok$

For side b = 0.35 m, $n_{bars} = 3 \text{ bars } \emptyset 16 + \geq 2 \text{ bars}$

$$s_{max,unrestrainer} = \frac{[b - (2 * d_1)]}{(n_{bars} - 1)} = \frac{0.35 - (2 * 0.039)}{(3 - 1)} = 0.136 \text{ m} = 136 \text{ mm}$$

$\leq 150 \text{ mm } \therefore Ok$

- Distribution of Longitudinal bars

$$A_{s1} = 8.04 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.104$$

$$A_{s2} = 8.04 * 10^{-4} \rightarrow \omega_{2d} = \frac{A_{s2}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.104$$

$$A_{sv} = 4.02 * 10^{-4} \rightarrow \omega_{vd} = \frac{A_{sv}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.052$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\varepsilon_{cu2} + \varepsilon_{yd}}{\varepsilon_{cu2} - \varepsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\varepsilon_{cu2} + \frac{\varepsilon_{c2}}{3}}{\varepsilon_{cu2} - \varepsilon_{yd}} \right)$$

$$V_2 = \frac{0.052}{1 - 0.0845} * \left(0.0845 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) + \left(0.0845 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right) = 0.1.$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\varepsilon_{cu2} - \varepsilon_{yd}}{\varepsilon_{cu2} + \varepsilon_{yd}} - \delta_1 \right) + \left(\frac{\varepsilon_{cu2} - \left(\frac{\varepsilon_{c2}}{3} \right)}{\varepsilon_{cu2} + \varepsilon_{yd}} \right) = \frac{0.052}{1 - 0.0845} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.0845 \right) + \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right) = 0.55$$

$$[V_2 = 0.1] \leq [V_d = 0.536] \leq [V_1 = 0.55] \rightarrow \text{Case 1}$$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} \right) + 2\omega_{vd}} = \frac{(1 - 0.0845) * (0.536) + (1 + 0.0845) * 0.052}{(1 - 0.0845) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \right) + 2 * 0.052} = 0.65$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} * \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right] \right]$$

$$M_{Rd,c} = 0.35 * 0.461^2 * \frac{25 * 10^3}{1.5} * 0.65 \left[\frac{1 - 0.65}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * \left(\frac{1}{2} - 0.65 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.65 \right) + \frac{(1 - 0.0845)(0.104 + 0.104)}{2} + \frac{0.052}{1 - 0.0845} * \left[(0.65 - 0.0845)(1 - 0.65) - \frac{1}{3} * \left(0.65 * \frac{348 * 10^3}{200 * 10^6} \right)^2 \right] \right]$$

$$= 233.15 \text{ Kn.m}$$

$$[M_{Rd,c} = 233.15 \text{ Kn.m}] > [1.3 * M_{Rb} = 1.3 * 170 = 220.1 \text{ Kn.m}] \therefore \text{Ok}$$

Using CSIColumn software, to check the column for all biaxial equation for all the building, the results shown that the column was not safe and need to increase column cross-section and reinforcement,

$$b_c = 35 \text{ cm}, h_c = 100 \text{ cm}$$

$$A_s = 60.04 \text{ cm}^2 \rightarrow 14 \text{ } \emptyset 20 + 8 \text{ } \emptyset 16$$

Detailed Design of Columns in Shear

Design of Exterior Columns for shear forces

$$V_{CD,Max} = 26.5 \text{ Kn}$$

$$V_{Rd,s} = \frac{Z}{h_{cl}} * N_{Ed} + \rho_w * b_w * Z * f_{ywd} * \cot \theta$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + v_d; \\ 2.5 * (1 - v_d). \end{array} \right) * b_w * Z * \left[1 - \frac{f_{ck}}{250} \right] * f_{cd} * \sin 2\theta$$

$$Z = 0.9 * d = 0.9 * 0.9 * 0.8 = 0.648 \text{ m}$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + 0.21 = 1.21; \\ 2.5 * (1 - 0.21) = 1.975. \end{array} \right) * 0.35 * 0.648 * \left[1 - \frac{25}{250} \right] * \frac{25 * 10^3}{1.5} * \sin(2 * 45)$$

$$V_{Rd,max} = 872.8 \text{ Kn.m}$$

$$h_{cl} = 3.2 - 0.5 = 2.7 \text{ m.}$$

$$\rho_w = \frac{A_{sh}}{b_w * s_h} \rightarrow \rho_w * b_w = \frac{A_{sh}}{s_h}$$

- Critical Region Length

$$L_{cr} \geq \max \left\{ \begin{array}{l} h_c = 0.80 \text{ m} \\ b_c = 0.35 \text{ m} \end{array} \right. = 0.80 \text{ m.}$$

- Detailing for Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \end{array} \right. \geq 6 \text{ mm.}$$

$$s_w \leq \min \left\{ \begin{array}{l} 20d_{bl} = 20 * 20 = 400 \text{ mm}; \\ h_c = 800 \text{ mm} \\ b_c = 350 \text{ mm} \\ 400 \text{ mm} \end{array} \right. \leq 350 \text{ mm.}$$

$$\text{Use } 5 \text{ } \emptyset 6 / \text{m} \rightarrow \emptyset 6 / 200 \text{ mm.}$$

$$V_{Rd.s} = \frac{0.648}{2.7} * 623 + \frac{4 * 0.28 * 10^{-4}}{0.2} * 0.648 * 348 * 10^3 * \cot 45 = 127.4 \text{ Kn}$$

- Detailing outside Critical Region

$$d_{bw} \geq \max \left\{ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \geq 6 \text{ mm} \right.$$

$$s_w \leq \min \left\{ \begin{array}{l} 20d_{bl} = 20 * 20 = 400 \text{ mm}; \\ h_c = 800 \text{ mm} \\ b_c = 350 \text{ mm} \\ 400 \text{ mm} \end{array} \right. \leq 350 \text{ mm}.$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd.s} = \frac{0.46}{2.7} * 623 + \frac{4 * 0.28 * 10^{-4}}{0.2} * 0.46 * 348 * 10^3 * \cot 45 = 127.4 \text{ Kn}$$

- Detailing for lap splices of bars

$$s_w \leq \min \left\{ \begin{array}{l} 12d_{bl} = 12 * 20 = 240 \text{ mm}; \\ 0.6h_c = 0.6 * 800 = 480 \text{ mm}; \\ 0.6b_c = 0.6 * 350 = 210 \text{ mm}; \\ 240 \text{ mm}. \end{array} \right. \leq 210 \text{ mm}.$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd.s} = \frac{0.648}{2.7} * 623 + \frac{4 * 0.28 * 10^{-4}}{0.66.67} * 0.648 * 348 * 10^3 * \cot 45 = 127.4 \text{ Kn}$$

- lap splices length

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{c_d}{d_{bl}} - 1 \right) \right] a_{tr} (d_{bl}/4) f_{yd} / (2.25 f_{ctd})$$

$$a_{tr} = 1 - k(2n_w A_{sw} - A_{s,t,min}) / A_s$$

$$c_d = \min \left\{ \begin{array}{l} \frac{1}{2} * [\{b - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \\ \frac{1}{2} * [\{h - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \\ 0.039 \text{ m} \end{array} \right.$$

$$c_d = \min \left\{ \begin{array}{l} \frac{1}{2} [\{0.35 - (2 * 0.025) - (0.006 * 2) - (0.016 * 3)\} / 2] = 0.06 \text{ m} \\ \frac{1}{2} [\{0.55 - (2 * 0.025) - (0.006 * 2) - (0.016 * 5)\} / 4] = 0.051 \text{ m} \end{array} \right.$$

For Corner \emptyset 16 bar

$$[d_{bl} = 16mm], [k = 0.1], [A_{sw} = 0.28 * 10^{-4} m^2], [n_w = 7 bars], [A_{s,t,min} = A_s = 2.01 * 10^{-4}].$$

$$c_d = 0.039m.$$

$$f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

$$a_{tr} = 1 - 0.1 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.905$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.039}{0.016} - 1 \right) \right] * 0.905 * \frac{\frac{0.016}{4} * 348 * 10^3}{(2.25 * 1.213 * 10^3)} = 0.54 \text{ m} \cong 0.55 \text{ m}$$

For Non – Corner \emptyset 16 bar

$$[d_{bl} = 16mm], [k = 0.05], [A_{sw} = 0.28 * 10^{-4} m^2], [n_w = 7 bars], [A_{s,t,min} = A_s = 2.01 * 10^{-4}].$$

$$c_d = 0.039m.$$

$$f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

$$a_{tr} = 1 - 0.05 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.952$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.039}{0.016} - 1 \right) \right] * 0.952 * \frac{\frac{0.016}{4} * 348 * 10^3}{(2.25 * 1.213 * 10^3)} = 0.57 \text{ m} \cong 0.60 \text{ m}$$

Design of Interior Columns for shear forces

$$V_{CD,Max} = 40 \text{ Kn}$$

$$V_{Rd,s} = \frac{Z}{h_{cl}} * N_{Ed} + \rho_w * b_w * Z * f_{ywd} * \cot \theta$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + v_d; \\ 2.5 * (1 - v_d). \end{array} \right) * b_w * Z * \left[1 - \frac{f_{ck}}{250} \right] * f_{cd} * \sin 2\theta$$

$$Z = 0.9 * d = 0.9 * 0.9 * 1 = 0.81 \text{ m}$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + 0.536 = 1.536; \\ 2.5 * (1 - 0.536) = 1.16. \end{array} \right) * 0.35 * 0.8 * \left[1 - \frac{25}{250} \right] * \frac{25 * 10^3}{1.5} * \sin(2 * 45)$$

$$V_{Rd,max} = 890.16 \text{ Kn.m}$$

$$h_{cl} = 3.2 - 0.5 = 2.7 \text{ m.}$$

$$\rho_w = \frac{A_{sh}}{b_w * s_h} \rightarrow \rho_w * b_w = \frac{A_{sh}}{s_h}$$

- Critical Region Length

$$L_{cr} \geq \max \begin{cases} h_c = 1.00m \\ b_c = 0.40m \end{cases} = 1.00 m.$$

- Detailing for Critical Region

$$d_{bw} \geq \max \left\{ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \geq 6 \text{ mm}. \right.$$

$$s_w \leq \min \begin{cases} 20d_{bl} = 20 * 16 = 320 \text{ mm}; \\ h_c = 1000 \text{ mm} \\ b_c = 350 \text{ mm} \\ 400\text{mm} \end{cases} \leq 320 \text{ mm}.$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd,s} = \frac{0.8}{2.7} * 1442 + \frac{4 * 0.28 * 10^{-4}}{0.25} * 0.8 * 348 * 10^3 * \cot 45 = 160 \text{ Kn}$$

- Detailing outside Critical Region

$$d_{bw} \geq \max \left\{ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \geq 6 \text{ mm}. \right.$$

$$s_w \leq \min \begin{cases} 20d_{bl} = 20 * 16 = 320 \text{ mm}; \\ h_c = 100 \text{ mm} \\ b_c = 400 \text{ mm} \\ 400\text{mm} \end{cases} \leq 320 \text{ mm}.$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd,s} = \frac{0.8}{2.7} * 1442 + \frac{4 * 0.28 * 10^{-4}}{0.2} * 0.8 * 348 * 10^3 * \cot 45 = 160 \text{ Kn}$$

- Detailing for lap splices of bars

$$s_w \leq \min \begin{cases} 12d_{bl} = 12 * 16 = 144 \text{ mm}; \\ 0.6h_c = 0.6 * 1000 = 600 \text{ mm}; \\ 0.6b_c = 0.6 * 400 = 240 \text{ mm}; \\ 240\text{mm}. \end{cases} \leq 144 \text{ mm}.$$

Use 7 $\emptyset 6 / m \rightarrow \emptyset 6 / 142.86 \text{ mm}$.

$$V_{Rd,s} = \frac{0.415}{2.7} * 1442 + \frac{4 * 0.28 * 10^{-4}}{0.14286} * 0.8 * 348 * 10^3 * \cot 45 = 223 \text{ Kn}$$

- lap splices length

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{c_d}{d_{bl}} - 1 \right) \right] a_{tr} (d_{bl}/4) f_{yd} / (2.25 f_{ctd})$$

$$a_{tr} = 1 - k (2n_w A_{sw} - A_{s,t,min}) / A_s$$

$$c_d = \min \begin{cases} \frac{1}{2} * [\{b - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \\ \frac{1}{2} * [\{h - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \\ 0.039 \text{ m} \end{cases}$$

$$c_d = \min \begin{cases} \frac{1}{2} [\{0.35 - (2 * 0.025) - (0.006 * 2) - (0.016 * 3)\} / 2] = 0.06 \text{ m} \\ \frac{1}{2} [\{0.50 - (2 * 0.025) - (0.006 * 2) - (0.016 * 4)\} / 4] = 0.046 \text{ m} \end{cases}$$

For Corner Ø 16 bar

$$[d_{bl} = 16\text{mm}], [k = 0.1], [A_{sw} = 0.28 * 10^{-4} \text{ m}^2], [n_w = 7 \text{ bars}], [A_{s,t,min} = A_s = 2.01 * 10^{-4}].$$

$$c_d = 0.039\text{m}.$$

$$f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

$$a_{tr} = 1 - 0.1 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.905$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.039}{0.016} - 1 \right) \right] * 0.905 * \frac{\frac{0.016}{4} * 348 * 10^3}{(2.25 * 1.213 * 10^3)} = 0.54 \text{ m} \cong 0.55 \text{ m}$$

For Non – Corner Ø 16 bar

$$[d_{bl} = 16\text{mm}], [k = 0.05], [A_{sw} = 0.28 * 10^{-4} \text{ m}^2], [n_w = 7 \text{ bars}], [A_{s,t,min} = A_s = 2.01 * 10^{-4}].$$

$$c_d = 0.039\text{m}.$$

$$f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

$$a_{tr} = 1 - 0.05 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.952$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.039}{0.016} - 1 \right) \right] * 0.952 * \frac{\frac{0.016}{4} * 348 * 10^3}{(2.25 * 1.213 * 10^3)} = 0.57 \text{ m} \cong 0.60 \text{ m}$$

Appendix B: Seismic Design example for DC M Frame as per EC8

According to seismic design rules of Eurocode 8, mentioned in chapter 3, a full design example for seismic design of DC M frame ($q_o = 3.9$) located in seismic zone 1.3 ($a_g = 1.5 \text{ m}^2/\text{s}$), to explain the way that the results of chapter 5 was obtained. The example is designing frame in figure 5.1.

Design of Beams

Detailed Design of beams in Flexure

Sizing of the beam

Beam width (b_{beam}) = 0.25m,

Beam depth (h_{beam}) = $L_{\text{span}}/10 = 0.5\text{m}$,

$d = 0.9 * h_{\text{beam}} = 0.9 * 0.5 = 0.45$

$z = d - d_2 = 0.9 * d = 0.9 * 0.45 = 0.405$

$$f_{yd} = \frac{f_{yk}}{1.15} = \frac{400}{1.15} = 347.8 \text{ Mpa}$$

$$f_{cd} = \frac{f_{ck}}{1.5} = \frac{25}{1.5} = 16.67 \text{ Mpa}$$

Dimensioning of the beam longitudinal reinforcement for the ULS in flexure

Figure B.1 below is showing the bending moment diagram on the beams, according to combinations a, b in section 3.4.1.1. The moment has redistributed with columns-beams joints 10% cracked, to reduce the moments on the beam-column joints, and increase the sag moment in the middle of the span.

Using equation 3.2 to calculate the longitudinal reinforcements of the beam sections

- Beam-1, Section 1-1:

$$A_{s,1} = |M_{Ed,1}| / (f_{yd} (d - d_2))$$

$$|M_{Ed,1}| = 67.5 \text{ Kn.m}$$

$$A_{s,1} = \frac{67.5}{347.8 * 1000 * 0.405} * 10^4 = 4.8 \text{ cm}^2$$

$$\text{Use } 3 \text{ } \emptyset 16 \rightarrow A_{s,1} = 6.03 \text{ cm}^2$$

$$|M_{Rd,1}| = A_{s,1} * (f_{yd} (d - d_2))$$

$$|M_{Rd,1}| = 6.03 * 10^{-4} * (347.8 * 1000 * 0.405) = 85 \text{ Kn.m}$$

- Beam-1, Section 2-2:

$$A_{s,2} = |M_{Ed,2}| / (f_{yd} (d - d_2))$$

$$|M_{Ed,2}| = 80 \text{ Kn.m}$$

$$A_{s,2} = \frac{80}{347.8 * 1000 * 0.405} * 10^4 = 5.8 \text{ cm}^2$$

$$\text{Use } 3 \text{ } \emptyset 16 \rightarrow A_{s,1} = 6.03 \text{ cm}^2$$

$$|M_{Rd,2}| = A_{s,1} * (f_{yd} (d - d_2))$$

$$|M_{Rd,2}| = 6.03 * 10^{-4} * (347.8 * 1000 * 0.405) = 85 \text{ Kn.m}$$

- Beam-1, Section 3-3:

$$\text{Use } 4 \text{ } \emptyset 16 \rightarrow A_{s,4} = 8.04 \text{ cm}^2$$

$$|M_{Rd,4}| = 113.6 \text{ Kn.m}$$

- Beam-2, Section 4-4:

$$\text{Use } 4 \text{ } \emptyset 16 \rightarrow A_{s,4} = 8.04 \text{ cm}^2$$

$$|M_{Rd,4}| = 113.6 \text{ Kn.m}$$

- Beam-2, Section 5-5:

$$\text{Use } 2 \text{ } \emptyset 16 \rightarrow A_{s,1} = 4.02 \text{ cm}^2$$

$$|M_{Rd,5}| = 56.6 \text{ Kn.m}$$

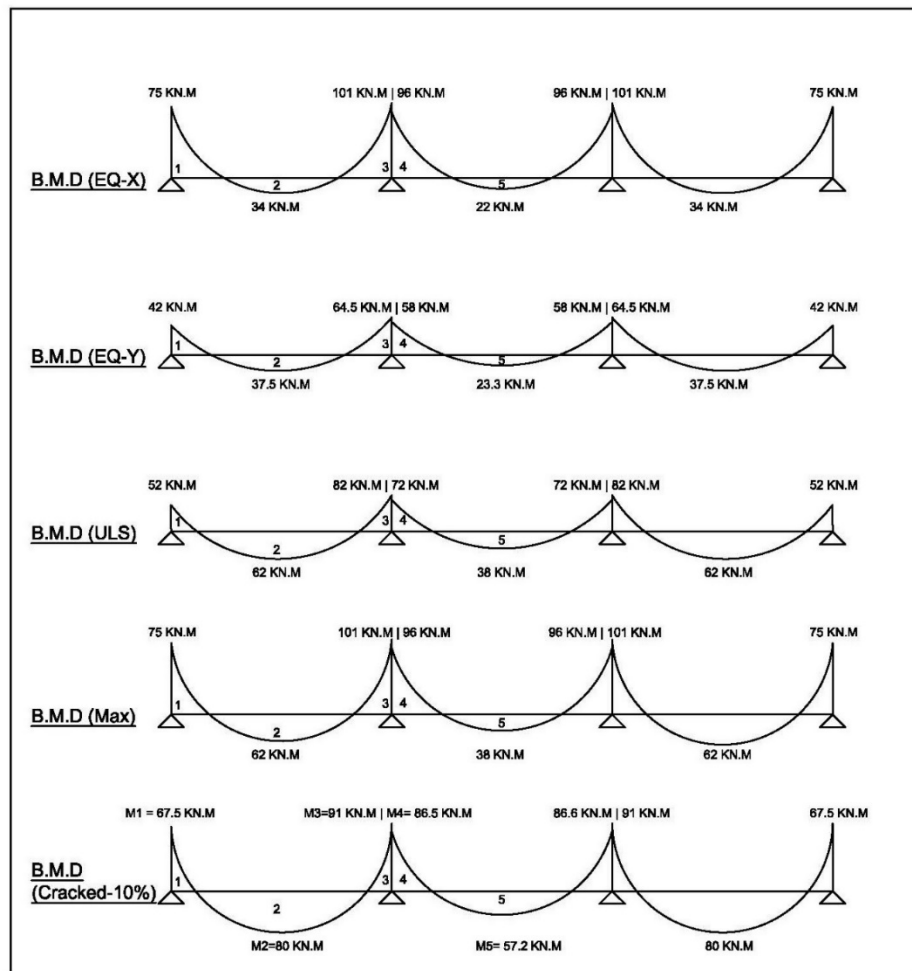


Figure B. 1 Bending moment diagrams for DC M frame.

$$h_{\text{column}} \geq b_{\text{beam}} + (2 * 0.050) = 0.25 + 0.1 \geq 0.35 \text{ m.}$$

$$a_{\text{cover}} = 0.025 \text{ m}$$

$$d_1 = a_{\text{cover}} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.35 - 0.039 = 0.311 \text{ m.}$$

$$\left[\begin{array}{l} V_{d,DCM} \leq 0.65; \\ V_{d,DCM} = \frac{N}{f_{cd} * b * d} = \frac{623}{\frac{25 * 10^3}{1.5} * (0.35 * 0.311)} = 0.343 \leq 0.65 \\ h_c = 0.3 \end{array} \right]$$

$$[h_{c(\text{exterior})} = 0.35\text{m}], [d_{bl} = 16 \text{ mm}], [f_{ctm} = 2.6 \text{ Mpa}], [f_{yd} = 348 \text{ Mpa}], [V_d = 0.343], [\gamma_{Rd,(DCM)} = 1], [k_{DCM} = 0.5], .$$

$$\frac{0.016}{h_{c(\text{exterior})}} \leq \left[\frac{7.5 * 2.6}{1 * 348} * (1 + (0.8 * 0.343)) \right] \rightarrow$$

$$h_{c(\text{exterior})} \geq 0.22 \text{ m} \rightarrow h_{c(\text{exterior})} = 0.35\text{m} \geq 0.22 \text{ m} \therefore Ok$$

- Critical Region Length

- $L_{cr} = h_b = 0.5 \text{ m.}$

- Minimum steel ratio at the tensile side

- $\rho_{min} = 0.5 * \frac{f_{ctm}}{f_{yk}} = 0.5 * \frac{2.6}{400} = 3.25 * 10^{-3}$

- $A_{s,min} = \rho_{min} * b * d = 3.25 * 10^{-3} * 0.25 * 0.45 = 3.65 * 10^{-4}$

- Minimum A_s bottom bars in critical regions

- $A_{s,min} = 0.5 * A_{s,top}$

For Beam 1 at Section 1-1

- $A_{s,min} = 0.5 * 5.15 * 10^{-4} = 2.575 * 10^{-4} \text{ m}^2 \rightarrow^{use} A_{s,min} = 1 \text{ } \emptyset 16 + 1 \text{ } \emptyset 12 = 3.14 * 10^{-4}$

For Beam 1 at Section 3-3

- $A_{s,min} = 0.5 * 8.04 * 10^{-4} = 4.02 * 10^{-4} \text{ m}^2 \rightarrow^{use} A_{s,min} = 2 \text{ } \emptyset 16 = 4.02 * 10^{-2} \text{ m}^2$

For Beam 2 at Section 4-4

- $A_{s,min} = 0.5 * 8.04 * 10^{-4} = 4.02 * 10^{-4} \text{ m}^2 \rightarrow^{use} A_{s,min} = 2 \text{ } \emptyset 16 = 4.02 * 10^{-2} \text{ m}^2$

- Maximum A_s in critical region

Section 1-1

- $\rho_{max} = \rho' + 0.0018 f_{cd} / (\mu_{\Phi} * \epsilon_{yd} * f_{yd})$

- $A_s' = 1 \text{ } \emptyset 16 + 1 \text{ } \emptyset 12 = 3.14 * 10^{-4}$

- $\rho' = \frac{A_s'}{b*d} = \frac{3.14*10^{-4}}{0.25*0.45} = 2.8 * 10^{-3}$
- $\mu_{\Phi} = 2q_o - 1 = (2 * 3.9) - 1 = 6.8$
- $\varepsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{348*10^3}{200*10^6} = 1.74 * 10^{-3}$
- $\rho_{max} = (2.8 * 10^{-3}) + [(0.0018 * 16.67 * 10^3)/(6.8 * 1.74 * 10^{-3} * 348 * 10^3)] = 10.086 * 10^{-3}$.
- $A_{s,max} = \rho_{max} * b * d = 10.086 * 10^{-3} * 0.25 * 0.45 = 11.34 * 10^{-4} m^2$.

Section 3-3 and Section 4-4

- $\rho_{max} = \rho' + 0.0018 f_{cd}/(\mu_{\Phi} * \varepsilon_{yd} * f_{yd})$
- Assume $\rightarrow A_s' = 2\emptyset 16 = 4.02 cm^2$
- $\rho' = \frac{A_s'}{b*d} = \frac{4.02*10^{-4}}{0.25*0.45} = 3.57 * 10^{-3}$
- $\mu_{\Phi} = 2q_o - 1 = (2 * 3.9) - 1 = 6.8$
- $\varepsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{348*10^3}{200*10^6} = 1.74 * 10^{-3}$
- $\rho_{max} = (3.57 * 10^{-3}) + [(0.0018 * 16.67 * 10^3)/(6.8 * 1.74 * 10^{-3} * 348 * 10^3)] = 10.866 * 10^{-3}$.
- $A_{s,max} = \rho_{max} * b * d = 10.866 * 10^{-3} * 0.25 * 0.45 = 12.22 * 10^{-4} m^2$.
-
- Anchorage Length
 - $L_{bd} = a_{tr} * [1 - 0.15((c_d/d_{bl}) - 1)(d_{bl}/4) f_{yd} / (2.25 f_{ctd} * a_{poor})]$
 - $a_{tr} = [1 - k(n_w * A_{sw} - A_{s,t,min}) / A_s] \geq 0.7$
 - $c_d = \min \left\{ \frac{1}{2} * \left[\frac{a_{cover}}{b - 2 * a_{cover} - d_{bl}} \right] / n \right\}$.
 - $f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 Mpa$

• Section 1-1

- For Top Reinforcement

For non-corner bars $\emptyset 12 \rightarrow [k = 0.05]$

- $[a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 1.13 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.2825 * 10^{-4} m^2]$

- $c_d =$

$$\min \left\{ \frac{1}{2} * \left[\frac{a_{cover} = 0.025 m}{\{0.25 - (2 * 0.025) - (0.016 + 0.016 + 0.012)\}} \right] / 2 \right\} = 0.0265 m \rightarrow c_d = 0.025 m$$

- $a_{tr} = 1 - \left[\frac{0.05(5*0.50*10^{-4} - 0.2825*10^{-4})}{1.13*10^{-4}} \right] = 0.924 \geq 0.7$.

- $L_{bd} = 0.924 * [1 - 0.15((0.025/0.012) - 1)] * \frac{(0.012/4) * 348*10^3}{(2.25*1.213*10^3*0.7)} = 0.423 m \cong 0.45 m$.

For corner bars $\emptyset 16 \rightarrow [k = 0.1]$

- $[a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 2.01 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4} m^2]$
- $a_{tr} = [1 - 0.1(5 * 0.5 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.9 \geq 0.7.$
- $L_{bd} = 0.9 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 0.7)} = 0.60m.$
- For Bottom Reinforcement

For corner bars $\emptyset 16 \rightarrow [k = 0.1]$

$$[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$$

- $c_d = \min \left\{ \begin{array}{l} a_{cover} = 0.025 m \\ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 m \end{array} \right. \rightarrow c_d = 0.025 m$
- $a_{tr} = [1 - 0.1(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.90 \geq 0.7.$
- $L_{bd} = 0.90 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.42m.$

For non-corner bars $\emptyset 16 \rightarrow [k = 0.05]$

$$[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$$

- $c_d = \min \left\{ \begin{array}{l} a_{cover} = 0.025 m \\ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 m \end{array} \right. \rightarrow c_d = 0.025 m$
- $a_{tr} = [1 - 0.05(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.95 \geq 0.7.$
- $L_{bd} = 0.95 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.45m.$

• Section 3-3, and Section 4-4

- For Top Reinforcement

For non-corner bars $\emptyset 16 \rightarrow [k = 0.05]$

- $[a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 2.01 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4} m^2]$
- $c_d = \min \left\{ \begin{array}{l} a_{cover} = 0.025 m \\ \frac{1}{2} * \left[\frac{\{0.25 - (2 * 0.025) - (0.016 + 0.016 + 0.016 + 0.016)\}}{3} \right] = 0.0226 m \end{array} \right.$
- $a_{tr} = 1 - \left[\frac{0.05(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4})}{2.01 * 10^{-4}} \right] = 0.95 \geq 0.7.$
- $L_{bd} = 0.95 * [1 - 0.15((0.0226/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 0.7)} = 0.65 m.$

For corner bars $\emptyset 16 \rightarrow [k = 0.1]$

- $[a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 2.01 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4} m^2]$
- $a_{tr} = [1 - 0.1(5 * 0.5 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.9 \geq 0.7.$
- $L_{bd} = 0.9 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 0.7)} = 0.60m.$
- For Bottom Reinforcement

For corner bars $\emptyset 16 \rightarrow [k = 0.1]$

$[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$

- $c_d = \min \left\{ \begin{array}{l} a_{cover} = 0.025 m \\ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 m \end{array} \right. \rightarrow c_d = 0.025 m$
- $a_{tr} = [1 - 0.1(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.90 \geq 0.7.$
- $L_{bd} = 0.90 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.42m.$

For non-corner bars $\emptyset 16 \rightarrow [k = 0.05]$

$[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$

- $c_d = \min \left\{ \begin{array}{l} a_{cover} = 0.025 m \\ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 m \end{array} \right. \rightarrow c_d = 0.025 m$
- $a_{tr} = [1 - 0.05(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.95 \geq 0.7.$
- $L_{bd} = 0.95 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.45m.$

Detailed Design of beams in Shear

$$V_{Rd,max} = 0.3 \times b_w \times Z \times \left[1 - \frac{f_{ck}}{250} \right] \times f_{cd} \times \sin 2\theta$$

$$[b_w = 0.25m], [Z = 0.405], [\theta = 45^\circ]$$

$$V_{Rd,max} = 0.3 * 0.25 * 0.405 * \left[1 - \frac{25}{250} \right] * 16.67 * 10^3 * \sin 90 = 455.6 Kn$$

$$[V_{Ed,max} = 77.8 Kn] < [V_{Rd,max} = 455.6]$$

\therefore The cross section of the beam is safe in shear.

$$V_{Rd} = \frac{A_{sh}}{S_h} * Z * f_{ywd} * \cot \theta$$

$$77.8 = \frac{0.56 * 10^{-4}}{S_w} * 0.405 * 348 * 10^3 * \cot 45$$

$$s_h = 0.101 \text{ m.} \rightarrow^{use} s_h = 0.1 \text{ m}$$

Use 10 \emptyset 6 / m

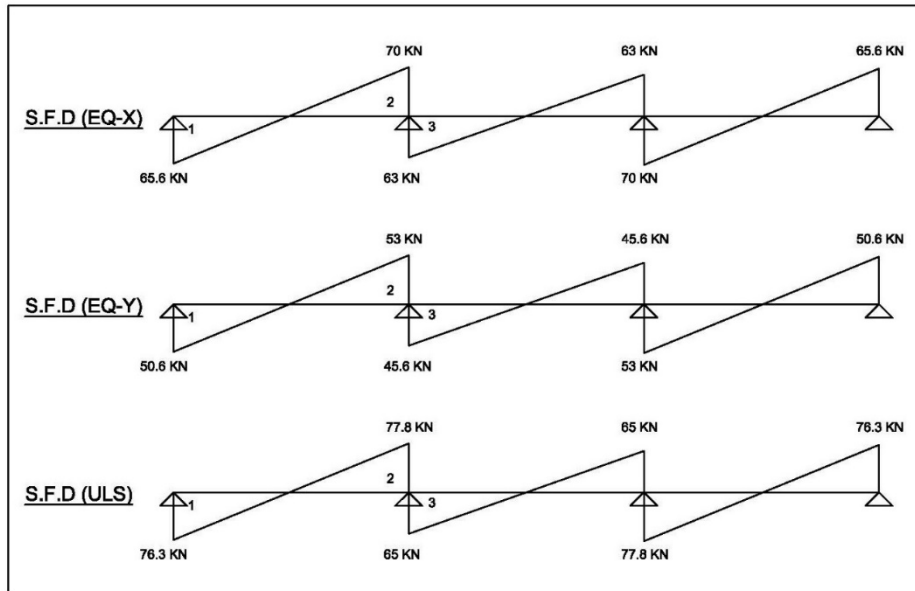


Figure B. 2 Shear force diagrams, for DC M frame in Seismic zone 1.3

Detailing rules for the transverse reinforcement

- Outside Critical Regions
 - $s_h \leq 0.75 d = 0.75 * 0.45 = 0.3375 \text{ m.}$
 - $\rho_w = \frac{A_{sh}}{b_w * s_h} \geq 0.08 * \sqrt{f_{ck}} / f_{yk} \rightarrow s_h = \frac{0.57 * 10^{-4} * 400}{0.25 * 0.08 * \sqrt{25}} = 0.228 \text{ m.}$
 - $s_h = \min \left\{ \begin{array}{l} 0.75 d = 0.3375 \text{ m.} \\ \frac{A_{sh}}{b_w * \rho_w} = 0.228 \text{ m.} \end{array} \right. \therefore s_h = 0.228 \text{ m} \rightarrow^{use} s_h = 0.2 \text{ m} = 200 \text{ mm}$
- In Critical Regions
 - $d_{bw} \geq 6 \text{ mm.}$
 - $s_h = \min \left\{ \begin{array}{l} 8d_{bl} = 8 * 12 = 96 \text{ mm} \\ h/4 = 125 \text{ mm} \\ 24d_{bw} = 144 \text{ mm} \\ 225 \text{ mm} \end{array} \right. \therefore s_h = 96 \text{ mm} \rightarrow^{use} s_h = 90.9 \text{ mm}$

Design of Columns

Detailed Design of Columns in Flexure

Design of Exterior Columns for bending

$$N_{Ed} = 623 \text{ Kn}$$

$$b_c = 0.35\text{m}, h_c = 0.35\text{m}$$

$$a_{cover} = 0.025 \text{ m}$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.35 - 0.039 = 0.311 \text{ m.}$$

$$V_d = \frac{N_{Ed}}{b * d * f_{cd}} = \frac{623}{0.35 * 0.311 * \frac{25 * 10^3}{1.5}} = 0.343 \leq 0.65$$

$$M_{Rc} \geq 1.3 * M_{Rb}$$

$$M_{Rb,m} = 72.5 \text{ Kn.m} \rightarrow M_{Rc} = 1.3 * 72.5 = 94.5 \text{ Kn.m}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{94.5}{0.35 * 0.311^2 * \frac{25 * 10^3}{1.5}} = 0.167$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.311} = 0.125$$

$$V_1 = \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} + \varepsilon_{yd}} = \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) + \frac{348 * 10^3}{200 * 10^6}} = 0.54$$

$$V_2 = \delta_1 * \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} - \varepsilon_{yd}} = 0.125 * \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) - \frac{348 * 10^3}{200 * 10^6}} = 0.20$$

$$[V_2 = 0.20] \leq [V_d = 0.343] \leq [V_1 = 0.54] \rightarrow \text{Case 1}$$

$$\xi = \frac{x}{d} = \frac{V_d}{1 - (\varepsilon_{c2}/3\varepsilon_{cu2})} = \frac{0.343}{1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}} = 0.424$$

$$\begin{aligned} (1 - \delta_1) * \omega_{1d} &= \mu_d - \xi * \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}} \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4\varepsilon_{cu2}} * \xi \right) \right] \rightarrow (1 - 0.125) * \omega_{1d} \\ &= 0.167 - 0.424 \\ &* \left[\frac{1 - 0.424}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \left(\frac{1}{2} - 0.424 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.424 \right) \right] \\ &\rightarrow \omega_{1d} = 0.064 \end{aligned}$$

$$\omega_{1d} = \frac{A_{s1}}{b*d} * \frac{f_{yd}}{f_{cd}} \rightarrow A_{s1} = 0.064 * 0.35 * 0.311 * \frac{16.667}{348} = 3.5 * 10^{-4} \text{ m}^2$$

$$A_{s,total} = 2 * A_{s1} = 2 * 3.5 * 10^{-4} = 7 * 10^{-4} \rightarrow \text{Use } 8\emptyset 12 = 9.04 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * d} * 100 = 0.737 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 0.737 \% A_c] \leq [\rho_{min} = 1 \% A_c] \therefore \text{Not. Ok} \rightarrow \text{Use } A_{s,min}$$

$$A_{s,min} = \rho_{min} * b * h = 0.01 * 0.35 * 0.35 = 12.25 * 10^{-4} \text{ m}^2 \rightarrow^{Use} 4\emptyset 16 + 4\emptyset 12$$

$$A_{s,total} = 12.56 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * h} * 100 = 1.02 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 1.02 \% A_c] \geq [\rho_{min} = 1 \% A_c] \therefore Ok$$

- Distribution of Longitudinal bars

$$A_{s1} = 5.15 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.1$$

$$A_{s2} = 5.15 * 10^{-4} \rightarrow \omega_{2d} = \frac{A_{s2}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.1$$

$$A_{sv} = 2.26 * 10^{-4} \rightarrow \omega_{vd} = \frac{A_{sv}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.043$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\varepsilon_{cu2} + \varepsilon_{yd}}{\varepsilon_{cu2} - \varepsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\varepsilon_{cu2} + \frac{\varepsilon_{c2}}{3}}{\varepsilon_{cu2} - \varepsilon_{yd}} \right)$$

$$V_2 = \frac{0.043}{1 - 0.125} * \left(0.125 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) + \left(0.125 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right) = 0.17.$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\varepsilon_{cu2} - \varepsilon_{yd}}{\varepsilon_{cu2} + \varepsilon_{yd}} - \delta_1 \right) + \left(\frac{\varepsilon_{cu2} - \left(\frac{\varepsilon_{c2}}{3} \right)}{\varepsilon_{cu2} + \varepsilon_{yd}} \right)$$

$$= \frac{0.043}{1 - 0.125} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.125 \right) + \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right)$$

$$V_1 = 0.55$$

$$[V_2 = 0.17] \leq [V_d = 0.343] \leq [V_1 = 0.55] \rightarrow Case 1$$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} \right) + 2\omega_{vd}}$$

$$= \frac{((1 - 0.125) * (0.343)) + ((1 + 0.125) * 0.043)}{(1 - 0.125) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \right) + (2 * 0.043)}$$

$$\xi = 0.44$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} \right. \\ \left. * \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right] \right]$$

$$M_{Rd,c} = 0.35 * 0.311^2 * \frac{25 * 10^3}{1.5} \\ * 0.44 \left[\frac{1 - 0.44}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * \left(\frac{1}{2} - 0.44 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.44 \right) \right. \\ \left. + \frac{(1 - 0.125)(0.1 + 0.1)}{2} + \frac{0.043}{1 - 0.125} \right. \\ \left. * \left[(0.44 - 0.125)(1 - 0.44) - \frac{1}{3} * \left(0.44 * \frac{348 * 10^3}{3.5 * 10^{-3}} \right)^2 \right] \right] = 86.8 \text{ Kn.m}$$

$$[M_{Rd,c} = 86.8 \text{ Kn.m}] < [1.3 * M_{Rb} = 1.3 * 72.55 = 94.3 \text{ Kn.m}] \therefore \text{Not. Ok}$$

→ Increase column cross – section

Increase column cross-section and repeat the previous steps, until the column moment resistance becomes 30 % stronger than the beam moment resistance,

We have done many iterations, until we could achieve the equation $M_{RC} \geq 1.3 * M_{Rb}$ with the following characteristics:

$$b_c = 0.35\text{m}, h_c = 0.4\text{m}$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.4 - 0.039 = 0.361 \text{ m.}$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.361} = 0.108$$

$$v_d = 0.3$$

$$A_{s1} = 1.5 * 10^{-4} \text{ m}^2$$

$$A_{s,total} = 2 * A_{s1} = 2 * 1.5 * 10^{-4} \text{ m}^2 = 3 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * h} * 100 = 0.214 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 0.214 \% A_c] \leq [\rho_{min} = 1 \% A_c] \therefore \text{Not. Ok} \rightarrow^{Use} A_{s,min}$$

$$A_{s,min} = \rho_{min} * b * h = 0.01 * 0.35 * 0.4 = 14 * 10^{-4} \text{ m}^2 \rightarrow^{Use} 4\emptyset 16 + 6\emptyset 12$$

$$A_{s,total} = 14.82 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * h} * 100 = 1.06 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 1.06 \% A_c] \geq [\rho_{min} = 1 \% A_c] \therefore \text{Ok}$$

Spacing along the perimeter of bars restrained by a tie corner or hook $\leq 200 \text{ mm}$.

For side $h = 0.40 \text{ m}$, $n_{bars} = 2 \text{ bars } \emptyset 16 + 2 \text{ bar } \emptyset 12 \geq 3 \text{ bars}$

$$s_{restrainer} = \frac{[h - (2 * d_1)]}{(n_{bars} - 1)} = \frac{0.40 - (2 * 0.039)}{(4 - 1)} = 0.107 \text{ m} = 107 \text{ mm} \leq 200 \text{ mm} \therefore \text{Ok}$$

For side $b = 0.35$ m, $n_{bars} = 2 \text{ bars } \emptyset 16 + 1 \text{ bar } \emptyset 12 \geq 3 \text{ bars}$

$$s_{restrainer} = \frac{[b-(2*d_1)]}{(n_{bars}-1)} = \frac{0.35-(2*0.039)}{(3-1)} = 0.136 \text{ m} = 136 \text{ mm} \leq 200 \text{ mm} \therefore Ok$$

All the column bars are restrained.

- Distribution of Longitudinal bars

$$A_{s1} = 5.15 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.085$$

$$A_{s2} = 5.15 * 10^{-4} \rightarrow \omega_{2d} = \frac{A_{s2}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.085$$

$$A_{sv} = 4.52 * 10^{-4} \rightarrow \omega_{vd} = \frac{A_{sv}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.074$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\varepsilon_{cu2} + \varepsilon_{yd}}{\varepsilon_{cu2} - \varepsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\varepsilon_{cu2} + \frac{\varepsilon_{c2}}{3}}{\varepsilon_{cu2} - \varepsilon_{yd}} \right)$$

$$V_2 = \frac{0.074}{1 - 0.108} * \left(0.108 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) + \left(0.108 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right) = 0.117.$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\varepsilon_{cu2} - \varepsilon_{yd}}{\varepsilon_{cu2} + \varepsilon_{yd}} - \delta_1 \right) + \left(\frac{\varepsilon_{cu2} - \left(\frac{\varepsilon_{c2}}{3} \right)}{\varepsilon_{cu2} + \varepsilon_{yd}} \right)$$

$$= \frac{0.074}{1 - 0.108} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.108 \right) + \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right) = 0.56$$

$[V_2 = 0.117] \leq [V_d = 0.30] \leq [V_1 = 0.56] \rightarrow \text{Case 1}$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} \right) + 2\omega_{vd}}$$

$$= \frac{(1 - 0.108) * (0.30) + ((1 + 0.108) * 0.074)}{(1 - 0.108) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \right) + (2 * 0.074)}$$

$$\xi = 0.40.$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} \right. \\ \left. * \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right] \right]$$

$$M_{Rd,c} = 0.35 * 0.361^2 * \frac{25 * 10^3}{1.5} \\ * 0.40 \left[\frac{1 - 0.40}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * \left(\frac{1}{2} - 0.40 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.40 \right) \right. \\ \left. + \frac{(1 - 0.108)(0.085 + 0.085)}{2} + \frac{0.074}{1 - 0.108} \right. \\ \left. * \left[(0.40 - 0.108)(1 - 0.40) - \frac{1}{3} * \left(0.40 * \frac{348 * 10^3}{200 * 10^6} \right)^2 \right] \right] = 108.5 \text{ Kn.m}$$

$$[M_{Rd,c} = 101] > [1.3 * M_{Rb} = 1.3 * 72.55 = 94.3 \text{ Kn.m}] \therefore \text{Ok}$$

Using CSIColumn software, to check the column for all biaxial equation for all the building, the results shown that the column was not safe and need to increase the concrete cross-section and longitudinal reinforcement;

$$b_c = 35 \text{ cm}, h_c = 40 \text{ cm}$$

$$A_s = 24.62 \text{ cm}^2 \rightarrow 4\emptyset 20 + 6\emptyset 16$$

Design of Interior Columns for bending forces

$$N_{Ed} = 1442 \text{ Kn}$$

$$b_c = 0.35\text{m}, h_c = 0.45\text{m}$$

$$a_{cover} = 0.025 \text{ m}$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.45 - 0.039 = 0.411 \text{ m.}$$

$$V_d = 0.60 \leq 0.65$$

$$M_{Rc} \geq 1.3 * M_{Rb}$$

$$M_{Rb,m} = 100.86 \text{ Kn.m} \rightarrow M_{Rc} = 1.3 * 100.86 = 131.12 \text{ Kn.m}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{131.12}{0.35 * 0.411^2 * \frac{25 * 10^3}{1.5}} = 0.133$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.411} = 0.095$$

$$V_1 = \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} + \varepsilon_{yd}} = \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) + \frac{348 * 10^3}{200 * 10^6}} = 0.54$$

$$V_2 = \delta_1 * \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} - \varepsilon_{yd}} = 0.084 * \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) - \frac{348 * 10^3}{200 * 10^6}} = 0.15$$

$$[V_2 = 0.15] \leq [V_d = 0.60] \geq [V_1 = 0.54] \rightarrow \text{Case 3}$$

We need to increase column cross section to get the value of V_d less than V_1 and to keep the column under the first case conditions,

Consider $[V_d = V_1 = 0.54]$

$$d = \frac{N}{f_{cd} * b * V_d} = \frac{1442}{\frac{25 * 10^3}{1.5} * 0.35 * (0.54)} = 0.457 \text{ m}$$

$$h_c = d + d_1 = 0.457 + 0.039 = 0.496 \text{ m} \cong 0.50 \text{ m}$$

$$d = h_c - d_1 = 0.50 - 0.039 = 0.461 \text{ m}$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.461} = 0.084$$

$$V_d = \frac{N_{Ed}}{b * d * f_{cd}} = \frac{1442}{0.35 * 0.461 * \frac{25 * 10^3}{1.5}} = 0.536 \leq 0.65$$

$$V_1 = 0.54, V_2 = 0.136$$

$$[V_2 = 0.112] \leq [V_d = 0.536] \leq [V_1 = 0.54] \rightarrow \text{Case 1}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{131.12}{0.35 * 0.461^2 * \frac{25 * 10^3}{1.5}} = 0.106$$

$$\xi = \frac{x}{d} = \frac{V_d}{1 - (\varepsilon_{cu2}/3\varepsilon_{cu2})} = \frac{0.536}{1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}} = 0.66$$

$$(1 - \delta_1) * \omega_{1d} = \mu_d - \xi * \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{cu2}}{3\varepsilon_{cu2}} \left(\frac{1}{2} - \xi + \frac{\varepsilon_{cu2}}{4\varepsilon_{cu2}} * \xi \right) \right]$$

$$(1 - 0.084) * \omega_{1d} = 0.106 - 0.66 * \left[\frac{1 - 0.66}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \left(\frac{1}{2} - 0.66 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.66 \right) \right]$$

$$\omega_{1d} = 0.019 \rightarrow A_{s1} = 1.5 * 10^{-4} \text{ m}^2$$

$$A_{s,total} = 2 * A_{s1} = 2 * 1.5 * 10^{-4} \text{ m}^2 = 3 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * h} * 100 = 0.171 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 0.1714 \% A_c] \leq [\rho_{min} = 1 \% A_c] \therefore \text{Not. Ok} \rightarrow^{Use} A_{s,min}$$

$$A_{s,min} = \rho_{min} * b * h = 0.01 * 0.35 * 0.5 = 17.5 * 10^{-4} \text{ m}^2 \rightarrow^{Use} 8\emptyset 16 + 2\emptyset 12$$

$$A_{s,total} = 18.34 * 10^{-4} \text{ m}^2$$

$$\rho = \frac{A_s}{b * h} * 100 = 1.048 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 1.048 \% A_c] \geq [\rho_{min} = 1 \% A_c]$$

Spacing along the perimeter of bars restrained by a tie corner or hook ≤ 200 mm.

For side h = 0.50 m, $n_{bars} = 4 \text{ bars } \emptyset 16 \geq 3 \text{ bars}$

$$S_{restrainer} = \frac{[h - (2 * d_1)]}{(n_{bars} - 1)} = \frac{0.50 - (2 * 0.039)}{(4 - 1)} = 0.130 \text{ m} = 140 \text{ mm} \leq 200 \text{ mm} \therefore \text{Ok}$$

For side b = 0.35 m, $n_{bars} = 2 \text{ bars } \emptyset 16 + 1 \text{ bar } \emptyset 12 \geq 3 \text{ bars}$

$$S_{restrainer} = \frac{[b - (2 * d_1)]}{(n_{bars} - 1)} = \frac{0.35 - (2 * 0.039)}{(3 - 1)} = 0.136 \text{ m} = 136 \text{ mm} \leq 200 \text{ mm} \therefore \text{Ok}$$

All the column bars are restrained.

- Distribution of Longitudinal bars

$$A_{s1} = 8.04 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.104$$

$$A_{s2} = 8.04 * 10^{-4} \rightarrow \omega_{2d} = \frac{A_{s2}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.104$$

$$A_{sv} = 2.26 * 10^{-4} \rightarrow \omega_{vd} = \frac{A_{sv}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.029$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\varepsilon_{cu2} + \varepsilon_{yd}}{\varepsilon_{cu2} - \varepsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\varepsilon_{cu2} + \frac{\varepsilon_{cu2}}{3}}{\varepsilon_{cu2} - \varepsilon_{yd}} \right)$$

$$V_2 = \frac{0.029}{1 - 0.0845} * \left(0.0845 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) + \left(0.0845 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right) = 0.11.$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\varepsilon_{cu2} - \varepsilon_{yd}}{\varepsilon_{cu2} + \varepsilon_{yd}} - \delta_1 \right) + \left(\frac{\varepsilon_{cu2} - \left(\frac{\varepsilon_{c2}}{3}\right)}{\varepsilon_{cu2} + \varepsilon_{yd}} \right) = \frac{0.029}{1 - 0.0845} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.0845 \right) + \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right) = 0.55$$

$$[V_2 = 0.09] \leq [V_d = 0.536] \leq [V_1 = 0.55] \rightarrow \text{Case 1}$$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}}\right) + 2\omega_{vd}} = \frac{(1 - 0.0845) * (0.536) + (1 + 0.0845) * 0.029}{(1 - 0.0845) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}\right) + 2 * 0.029} = 0.653$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} * \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right] \right]$$

$$M_{Rd,c} = 0.35 * 0.461^2 * \frac{25 * 10^3}{1.5} * 0.653 \left[\frac{1 - 0.653}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * 0.653 \right] + \frac{(1 - 0.0845)(0.104 + 0.104)}{2} + \frac{0.029}{1 - 0.0845} * \left[(0.653 - 0.0845)(1 - 0.653) - \frac{1}{3} * \left(0.653 * \frac{348 * 10^3}{200 * 10^6} \right)^2 \right] = 230 \text{ Kn.m}$$

$$[M_{Rd,c} = 230 \text{ Kn.m}] > [1.3 * M_{Rb} = 1.3 * 100.08 = 131.12 \text{ Kn.m}] \therefore Ok$$

Using CSIColumn software, to check the column for all biaxial equation for all the building, the results shown that the column was not safe and need to increase longitudinal reinforcement;

$$b_c = 35 \text{ cm}, h_c = 75 \text{ cm}$$

$$A_s = 50.24 \text{ cm}^2 \rightarrow 16\emptyset 20$$

Detailed Design of Columns in Shear

Design of Exterior Columns for shear

$$V_{CD,Max} = 35 \text{ Kn}$$

$$V_{Rd,s} = \frac{Z}{h_{cl}} * N_{Ed} + \rho_w * b_w * Z * f_{ywd} * \cot \theta$$

$$Z = 0.9 * d = 0.9 * 0.9 * 0.5 = 0.40 \text{ m}$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{c} 1.25; \\ 1 + v_d; \\ 2.5 * (1 - v_d). \end{array} \right) * b_w * Z * \left[1 - \frac{f_{ck}}{250} \right] * f_{cd} * \sin 2\theta$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{c} 1.25; \\ 1 + 0.237 = 1.237; \\ 2.5 * (1 - 0.237) = 1.90. \end{array} \right) * 0.30 * 0.40 * \left[1 - \frac{25}{250} \right] * \frac{25 * 10^3}{1.5} * \sin(2 * 45)$$

$$V_{Rd,max} = 668.11 \text{ Kn.m}$$

$$h_{cl} = 3.2 - 0.5 = 2.7 \text{ m.}$$

$$\rho_w = \frac{A_{sh}}{b_w * s_h} \rightarrow \rho_w * b_w = \frac{A_{sh}}{s_h}$$

- Critical Region Length

$$L_{cr} \geq \max \left\{ \begin{array}{l} h_c = 0.50 \text{ m} \\ b_c = 0.35 \text{ m} \\ 0.45 \text{ m} \\ \frac{H_{cl}}{6} = \frac{3.2-0.5}{6} = 0.45 \text{ m} \end{array} \right. = 0.50 \text{ m.}$$

- Detailing for Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \end{array} \right. \geq 6 \text{ mm.}$$

$$s_w \leq \min \begin{cases} 8d_{bl} = 8 * 20 = 160 \text{ mm}; \\ b_o/2 = 300/2 = 150 \text{ mm}; \leq 150 \text{ mm}. \\ 175 \text{ mm}. \end{cases}$$

Use 7 $\emptyset 6$ /m $\rightarrow \emptyset 6$ / 142.86 mm.

$$V_{Rd.s} = \frac{0.4}{2.7} * 623 + \frac{4 * 0.28 * 10^{-4}}{0.142} * 0.235 * 348 * 10^3 * \cot 45 = 120 \text{ Kn}$$

- Detailing outside Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \geq 6 \text{ mm}. \end{array} \right.$$

$$s_w \leq \min \begin{cases} 20d_{bl} = 20 * 20 = 400 \text{ mm}; \\ h_c = 500 \text{ mm} \\ b_c = 350 \text{ mm} \\ 400 \text{ mm} \end{cases} \leq 350 \text{ mm}.$$

Use 5 $\emptyset 6$ /m $\rightarrow \emptyset 6$ / 200 mm.

$$V_{Rd.s} = \frac{0.40}{2.7} * 623 + \frac{4 * 0.28 * 10^{-4}}{0.20} * 0.40 * 348 * 10^3 * \cot 45 = 111 \text{ Kn}$$

- Detailing for lap splices of bars

$$s_w \leq \min \begin{cases} 12d_{bl} = 12 * 16 = 144 \text{ mm}; \\ 0.6h_c = 0.6 * 400 = 240 \text{ mm}; \\ 0.6b_c = 180 \text{ mm}; \\ 240 \text{ mm}. \end{cases} \leq 144 \text{ mm}.$$

Use 7 $\emptyset 6$ /m $\rightarrow \emptyset 6$ / 142.86 mm.

$$V_{Rd.s} = \frac{0.235}{2.7} * 623 + \frac{4 * 0.28 * 10^{-4}}{0.14286} * 0.235 * 348 * 10^3 * \cot 45 = 120 \text{ Kn}$$

- lap splices length

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{c_d}{d_{bl}} - 1 \right) \right] a_{tr} (d_{bl}/4) f_{yd} / (2.25 f_{ctd})$$

$$a_{tr} = 1 - k(2n_w A_{sw} - A_{s,t,min}) / A_s$$

$$[d_{bl} = 12 \text{ mm}; 16 \text{ mm}], [k = 0.05; 0.1], [A_{sw} = 0.28 * 10^{-4} \text{ m}^2], [n_w = 7 \text{ bars}], [A_{s,t,min} = A_s = 1.13 * 10^{-4}; 2.01 * 10^{-4}].$$

$$c_d = \min \begin{cases} \frac{1}{2} * [\{b - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \\ \frac{1}{2} * [\{h - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \end{cases}$$

c_d

$$= \min \begin{cases} 0.025 \text{ m} \\ \frac{1}{2} [\{0.35 - (2 * 0.025) - (0.006 * 2) - (0.016 + 0.016 + 0.012)\} / 2] = 0.0485 \text{ m} \\ \frac{1}{2} [\{0.40 - (2 * 0.025) - (0.006 * 2) - (0.016 + 0.016 + 0.012)\} / 2] = 0.075 \text{ m} \end{cases}$$

$$c_d = 0.025m.$$

$$f_{cta} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

$$\text{For } d_{bl} = 12mm \rightarrow k = 0.05$$

$$a_{tr} = 1 - 0.05 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (1.13 * 10^{-4})}{(1.13 * 10^{-4})} = 0.877$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.012} - 1 \right) \right] * 0.877 * \frac{\frac{0.012}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.42 \text{ m} \cong 0.45 \text{ m}$$

$$\text{For } d_{bl} = 16mm \rightarrow k = 0.1$$

$$a_{tr} = 1 - 0.1 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.905$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.016} - 1 \right) \right] * 0.905 * \frac{\frac{0.016}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.63 \text{ m} \cong 0.65 \text{ m}$$

- At the Connection to the foundation

$$a * \omega_d \geq 30 \mu_\phi * v_d * \varepsilon_{yd} * \frac{b_c}{b_o} - 0.035$$

$$a = \left(1 - \frac{s}{2b_o} \right) * \left(1 - \frac{s}{2h_o} \right) * \left(1 - \left\{ \frac{b_o}{[(n_h - 1)h_o]} + \frac{h_o}{[(n_b - 1)b_o]} \right\} / 3 \right)$$

$$b_o = (b_c - 2 a_{cover} - 2 d_{bw}) = 0.3 - (2 * 0.025) - (2 * 0.006) = 0.238 \text{ m.}$$

$$h_o = (h_c - 2 a_{cover} - 2 d_{bw}) = 0.4 - (2 * 0.025) - (2 * 0.006) = 0.338 \text{ m.}$$

$$[s = 0.125 \text{ m}], [b_o = 0.238 \text{ m}], [h_o = 0.338 \text{ m}], [n_b = 6], [n_h = 2]$$

$$a = \left(1 - \frac{0.09091}{2 * 0.238} \right) * \left(1 - \frac{0.09091}{2 * 0.338} \right) * \left(1 - \left\{ \frac{0.238}{[(2 - 1) * 0.338]} + \frac{0.338}{[(6 - 1)0.238]} \right\} / 3 \right)$$

$$= 0.47$$

$$0.47 * \omega_d \geq 30 * 6.8 * 0.356 * 1.74 * 10^{-3} * \frac{0.3}{0.238} - 0.035$$

$$\omega_d = 0.264 \geq 0.08 \therefore 0.k$$

Using CSIColumn software, to check the column for all biaxial equation for all the building, the results shown that the column was not safe and need to increase the concrete cross-section and longitudinal reinforcement;

$$b_c = 35 \text{ cm}, h_c = 50 \text{ cm}$$

$$A_s = 44 \text{ cm}^2 \rightarrow 14\emptyset 20$$

$$V_{CD,Max} = 70 \text{ Kn}$$

$$V_{Rd,s} = \frac{Z}{h_{cl}} * N_{Ed} + \rho_w * b_w * Z * f_{ywd} * \cot \theta$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + v_d; \\ 2.5 * (1 - v_d). \end{array} \right) * b_w * Z * \left[1 - \frac{f_{ck}}{250} \right] * f_{cd} * \sin 2\theta$$

$$Z = 0.9 * d = 0.9 * 0.9 * 0.75 = 0.625 \text{ m}$$

$$b_w = b_c - 2a_{cover} = 0.35 - (2 * 0.025) = 0.30 \text{ m}$$

$$h_{cl} = 3.2 - 0.5 = 2.7 \text{ m.}$$

$$\rho_w = \frac{A_{sh}}{b_w * s_h} \rightarrow \rho_w * b_w = \frac{A_{sh}}{s_h}$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + 0.36 = 1.36; \\ 2.5 * (1 - 0.36) = 1.6. \end{array} \right) * 0.30 * 0.625 * \left[1 - \frac{25}{250} \right] * \frac{25 * 10^3}{1.5} * \sin(2 * 45)$$

$$V_{Rd,max} = 1148 \text{ Kn. m}$$

- Critical Region Length

$$L_{cr} \geq \max \left\{ \begin{array}{l} h_c = 0.60 \text{ m} \\ b_c = 0.30 \text{ m} \\ 0.45 \text{ m} \\ \frac{H_{cl}}{6} = \frac{3.2-0.5}{6} = 0.45 \text{ m} \end{array} \right. = 0.60 \text{ m.}$$

- Detailing for Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \end{array} \right. \geq 6 \text{ mm.}$$

$$s_w \leq \min \left\{ \begin{array}{l} 8d_{bl} = 8 * 12 = 96 \text{ mm}; \\ b_o/2 = 238/2 = 119 \text{ mm}; \leq 96 \text{ mm.} \\ 175 \text{ mm.} \end{array} \right.$$

Use 11 $\emptyset 6 / \text{m} \rightarrow \emptyset 6 / 90.91 \text{ mm.}$

$$V_{Rd,s} = \frac{0.505}{2.7} * 1442 + \frac{3 * 0.28 * 10^{-4}}{0.09091} * 0.505 * 348 * 10^3 * \cot 45 = 432.1 \text{ Kn}$$

- Detailing outside Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ \frac{d_{bl}}{4} = \frac{16}{4} = 4 \text{ mm} \end{array} \right. \geq 6 \text{ mm.}$$

$$s_w \leq \min \begin{cases} 20d_{bl} = 20 * 12 = 240 \text{ mm}; \\ h_c = 600 \text{ mm}; \\ b_c = 300 \text{ mm}; \\ 400\text{mm}. \end{cases} \leq 240 \text{ mm}.$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd.s} = \frac{0.505}{2.7} * 1442 + \frac{3 * 0.28 * 10^{-4}}{0.20} * 0.505 * 348 * 10^3 * \cot 45 = 343.52 \text{ Kn}$$

- Detailing for lap splices of bars

$$s_w \leq \min \begin{cases} 12d_{bl} = 12 * 12 = 144 \text{ mm}; \\ 0.6h_c = 0.6 * 400 = 240 \text{ mm}; \\ 0.6b_c = 180 \text{ mm}; \\ 240\text{mm}. \end{cases} \leq 144 \text{ mm}.$$

Use 7 $\emptyset 6 / m \rightarrow \emptyset 6 / 142.86 \text{ mm}$.

$$V_{Rd.s} = \frac{0.505}{2.7} * 1442 + \frac{3 * 0.28 * 10^{-4}}{0.14286} * 0.505 * 348 * 10^3 * \cot 45 = 373 \text{ Kn}$$

- lap splices length

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{c_d}{d_{bl}} - 1 \right) \right] a_{tr} (d_{bl}/4) f_{yd} / (2.25 f_{ctd})$$

$$a_{tr} = 1 - k(2n_w A_{s_w} - A_{s,t,min}) / A_s$$

$$[d_{bl} = 12\text{mm}; 16\text{mm}], [k = 0.05; 0.1], [A_{s_w} = 0.28 * 10^{-4} \text{ m}^2], [n_w = 7 \text{ bars}], [A_{s,t,min} = A_s = 1.13 * 10^{-4}; 2.01 * 10^{-4}].$$

$$c_d = \min \begin{cases} \frac{1}{2} * [\{b - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \\ \frac{1}{2} * [\{h - 2 * a_{cover} - d_{bw} - d_{bl}\} / n] \end{cases}$$

c_d

$$= \min \begin{cases} 0.025 \text{ m} \\ \frac{1}{2} [\{0.30 - (2 * 0.025) - (0.006 * 2) - (0.016 + 0.016 + 0.012)\} / 2] = 0.0485 \text{ m} \\ \frac{1}{2} [\{0.60 - (2 * 0.025) - (0.006 * 2) - (0.016 + 0.016 + 0.016)\} / 2] = 0.1265 \text{ m} \end{cases}$$

$$c_d = 0.025\text{m}.$$

$$f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

For $d_{bl} = 12\text{mm} \rightarrow k = 0.05$

$$a_{tr} = 1 - 0.05 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (1.13 * 10^{-4})}{(1.13 * 10^{-4})} = 0.877$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.012} - 1 \right) \right] * 0.877 * \frac{\frac{0.012}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.42 \text{ m} \cong 0.45 \text{ m}$$

For $d_{bl} = 16mm \rightarrow k = 0.05$

$$a_{tr} = 1 - 0.05 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.9525$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.016} - 1 \right) \right] * 0.9525 * \frac{\frac{0.016}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.67 m \cong 0.70 m$$

For $d_{bl} = 16mm \rightarrow k = 0.1$

$$a_{tr} = 1 - 0.1 * \frac{(2 * 7 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.905$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.016} - 1 \right) \right] * 0.905 * \frac{\frac{0.016}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.63 m \cong 0.65 m$$

- At the Connection to the foundation

$$a * \omega_d \geq 30 \mu_\phi * v_d * \varepsilon_{yd} * \frac{b_c}{b_o} - 0.035$$

$$a = \left(1 - \frac{s}{2b_o} \right) * \left(1 - \frac{s}{2h_o} \right) * \left(1 - \left\{ \frac{b_o}{[(n_h - 1)h_o]} + \frac{h_o}{[(n_b - 1)b_o]} \right\} / 3 \right)$$

$$b_o = (b_c - 2 a_{cover} - 2 d_{bw}) = 0.3 - (2 * 0.025) - (2 * 0.006) = 0.238 m.$$

$$h_o = (h_c - 2 a_{cover} - 2 d_{bw}) = 0.6 - (2 * 0.025) - (2 * 0.006) = 0.538 m.$$

$$[s = 0.09091 m], [b_o = 0.238 m], [h_o = 0.538 m], [n_b = 6], [n_h = 2]$$

$$a = \left(1 - \frac{0.09091}{2 * 0.238} \right) * \left(1 - \frac{0.09091}{2 * 0.538} \right) * \left(1 - \left\{ \frac{0.238}{[(2 - 1) * 0.538]} + \frac{0.538}{[(6 - 1) * 0.238]} \right\} / 3 \right)$$

$$= 0.52$$

$$0.52 * \omega_d \geq 30 * 6.8 * 0.514 * 1.74 * 10^{-3} * \frac{0.3}{0.238} - 0.035$$

$$\omega_d = 0.375 \geq 0.08 \therefore 0. k$$

Appendix C: Seismic Design example for DC H Frame as per EC8

According to seismic design rules of Eurocode 8, mentioned in chapter 3, a full design example for seismic design of DC H frame ($q_o = 5.85$) located in seismic zone 1.3 ($a_g = 1.5 \text{ m}^2/\text{s}$), to explain the way that the results of chapter 5 was obtained. The example is designing frame in figure 5.1.

Design of Beams

Detailed Design of beams in Flexure

Sizing of the beam

Beam width (b_{beam}) = 0.25m,

Beam depth (h_{beam}) = $L_{\text{span}}/10 = 0.5\text{m}$,

$d = 0.9 * h_{\text{beam}} = 0.9 * 0.5 = 0.45$

$z = d - d_2 = 0.9 * d = 0.9 * 0.45 = 0.405$

$$f_{yd} = \frac{f_{yk}}{1.15} = \frac{400}{1.15} = 347.8 \text{ Mpa}$$

$$f_{cd} = \frac{f_{ck}}{1.5} = \frac{25}{1.5} = 16.67 \text{ Mpa}$$

Dimensioning of the beam longitudinal reinforcement for the ULS in flexure

Figure C.1 below is showing the bending moment diagram on the beams, according to combinations a, b in section 3.4.1.1. The moment has redistributed with columns-beams joints 10% cracked, to reduce the moments on the beam-column joints, and increase the sag moment in the middle of the span.

Using equation 3.2 to calculate the longitudinal reinforcements of the beam sections

- Beam-1, Section 1-1:

$$A_{s,1} = |M_{Ed,1}| / (f_{yd}(d - d_2))$$

$$|M_{Ed,1}| = 46.8 \text{ Kn.m}$$

$$A_{s,1} = \frac{46.8}{348 * 10^3 * 0.405} * 10^4 = 4.0625 \text{ cm}^2$$

$$\text{Use } 4 \text{ } \emptyset 12 \rightarrow A_{s,1} = 4.52 \text{ cm}^2$$

$$|M_{Rd,1}| = A_{s,1} * (f_{yd}(d - d_2))$$

$$|M_{Rd,1}| = 4.52 * 10^{-4} * (347.8 * 1000 * 0.405) = 63.67 \text{ Kn.m}$$

- Beam-1, Section 2-2:

$$A_{s,2} = |M_{Ed,2}| / (f_{yd}(d - d_2))$$

$$|M_{Ed,2}| = 74.4 \text{ Kn.m}$$

$$A_{s,2} = \frac{74.4}{348 * 10^3 * 0.405} * 10^4 = 5.28 \text{ cm}^2$$

$$\text{Use } 3 \text{ } \emptyset 16 \rightarrow A_{s,1} = 6.03 \text{ cm}^2$$

$$|M_{Rd,2}| = A_{s,1} * (f_{yd} (d - d_2))$$

$$|M_{Rd,2}| = 6.03 * 10^{-4} * (347.8 * 1000 * 0.405) = 85 \text{ Kn.m}$$

- Beam-1, Section 3-3:

$$\text{Use } 3 \text{ } \emptyset 16 \rightarrow A_{s,3} = 6.03 \text{ cm}^2$$

$$|M_{Rd,3}| = 85 \text{ Kn.m}$$

- Beam-2, Section 4-4:

$$\text{Use } 3 \text{ } \emptyset 16 \rightarrow A_{s,3} = 6.03 \text{ cm}^2$$

$$|M_{Rd,3}| = 85 \text{ Kn.m}$$

- Beam-2, Section 5-5:

$$\text{Use } 2 \text{ } \emptyset 16 \rightarrow A_{s,1} = 4.02 \text{ cm}^2$$

$$|M_{Rd,5}| = 56.6 \text{ Kn.m}$$

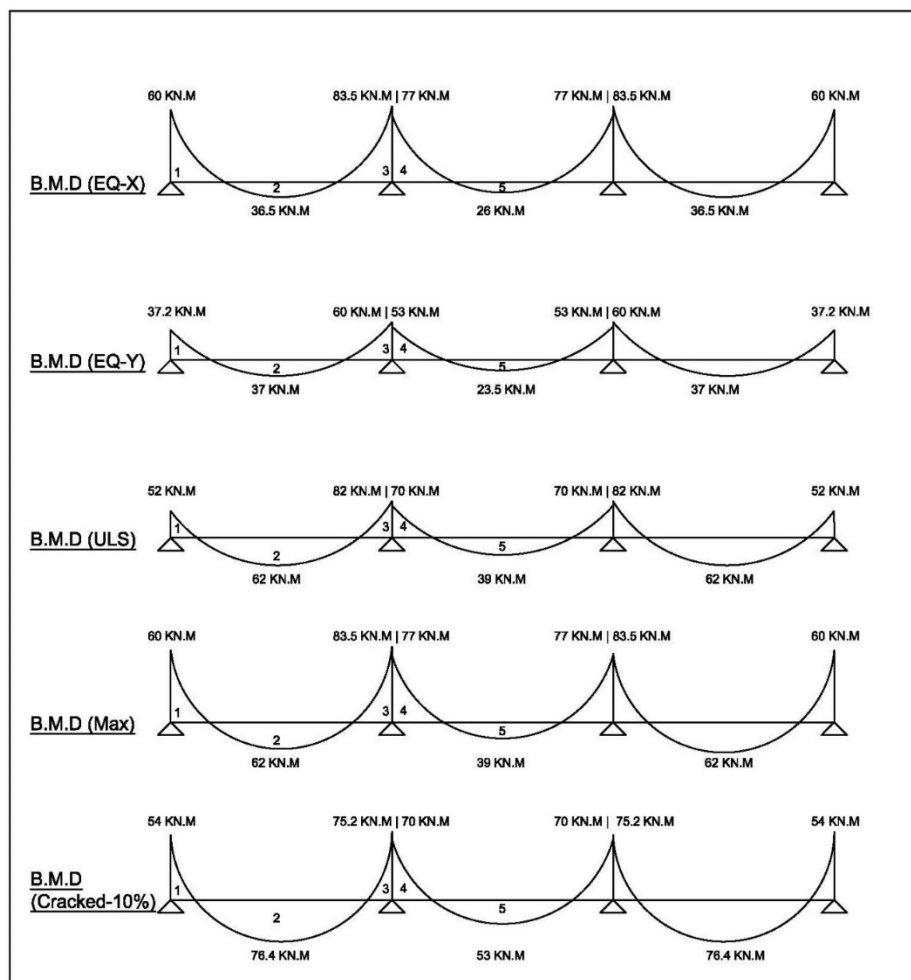


Figure C. 1 Bending moment diagrams for DC H frame building in Seismic zone 1.3.

Detailing rules for the longitudinal reinforcement

- Beam-Column Joint

- Interior Beam-Column Joint

$$\frac{d_{bl}}{h_c} \leq \frac{7.5 f_{ctm}}{\gamma_{Rd} * f_{yd}} * \frac{1 + 0.8V_d}{1 + K * \frac{\rho_2}{\rho_{1,max}}}$$

$$b_{column} = b_{beam} + (2 * 0.050) = 0.25 + 0.1 = 0.35 \text{ m}$$

$$h_{column} = 0.40 \text{ m} \geq b_{beam} + (2 * 0.050) = 0.25 + 0.1 \geq 0.35 \text{ m.}$$

$$a_{cover} = 0.025 \text{ m}$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.40 - 0.039 = 0.361 \text{ m.}$$

$$\left[\begin{array}{l} V_{d,DCH} \leq 0.55 \\ V_{d,DCH} = \frac{N}{f_{cd} * b * d} = \frac{1442}{\frac{25 * 10^3}{1.5} * (0.35 * 0.361)} = 0.684 \geq 0.55 \\ \therefore \text{Not.Ok} \rightarrow \text{Increase column cross section} \\ d = \frac{N}{f_{cd} * V_d} = \frac{1442}{\frac{25 * 10^3}{1.5} * (0.55) * 0.35} = 0.45 \text{ m} \\ h_c = d + d_1 = 0.45 + 0.039 = 0.49 \text{ m} \cong 0.50 \text{ m} \\ d = h_c - d_1 = 0.50 - 0.039 = 0.461 \text{ m.} \\ V_{d,DCH} = \frac{N}{f_{cd} * b * d} = \frac{1442}{\frac{25 * 10^3}{1.5} * (0.35 * 0.461)} = 0.536 \leq 0.55 \end{array} \right]$$

$$[h_{c(interior)} = 0.6 \text{ m}], [d_{bl} = 16 \text{ mm}], [f_{ctm} = 2.6 \text{ Mpa}], [f_{yd} = 348 \text{ Mpa}], [V_d = 0.536], [\gamma_{Rd,(DCH)} = 1], [k_{DCH} = 0.75], [\rho_{1,max} = \frac{A_{s,top,max}}{b*d} = \frac{6.03 * 10^{-4}}{0.25 * 0.45} = 5.36 * 10^{-3}], [\rho_2 = \frac{\rho_{1,max}}{2} = 2.68 * 10^{-3}].$$

$$\frac{0.016}{h_{c(interior)}} \leq \left[\frac{7.5 * 2.6}{1.2 * 348} * \frac{1 + 0.8 * 0.536}{1 + 0.75 * \frac{2.68 * 10^{-3}}{5.36 * 10^{-3}}} \right] \rightarrow$$

$$h_{c(interior)} \geq 0.33 \text{ m}, h_{c(interior)} = 0.6 \text{ m} \geq 0.33 \text{ m} \therefore \text{Ok}$$

- Exterior Beam-Column Joint

$$\frac{d_{bl}}{h_c} \leq \frac{7.5 f_{ctm}}{\gamma_{Rd} * f_{yd}} * (1 + 0.8V_d)$$

$$b_{column} \geq b_{beam} + (2 * 0.050) = 0.25 + 0.1 \geq 0.35 \text{ m}$$

$$h_{column} \geq b_{beam} + (2 * 0.050) = 0.25 + 0.1 \geq 0.35 \text{ m}$$

$$a_{cover} = 0.025 \text{ m}$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.35 - 0.039 = 0.311 \text{ m.}$$

$$\left[\begin{array}{l} V_{d,DCH} \leq 0.55; \\ V_{d,DCH} = \frac{N}{f_{cd} * b * d} = \frac{623}{\frac{25 * 10^3}{1.5} * (0.35 * 0.311)} = 0.343 \leq 0.55 \end{array} \right]$$

$$[h_{c(exterior)} = 0.35\text{m}], [f_{ctm} = 2.6 \text{ Mpa}], [f_{yd} = 348 \text{ Mpa}], [V_d = 0.343], [\gamma_{Rd,(DCH)} = 1.2].$$

$$\frac{0.016}{h_{c(interior)}} \leq \left[\frac{7.5 * 2.6}{1.2 * 348} * (1 + 0.8 * 0.343) \right] \rightarrow$$

$$h_{c(interior)} \geq 0.26\text{m}, h_{c(interior)} = 0.35\text{m} \geq 0.23\text{m} \therefore Ok$$

- Critical Region Length

- $L_{cr} = 1.5 * h_b = 1.5 * 0.5 = 0.75\text{m}$

- Minimum steel ratio at the tensile side

- $\rho_{min} = 0.5 * \frac{f_{ctm}}{f_{yk}} = 0.5 * \frac{2.6}{400} = 3.25 * 10^{-3}$

- $A_{s,min} = \rho_{min} * b * d = 3.25 * 10^{-3} * 0.25 * 0.45 = 3.65 * 10^{-4}$

- Minimum A_s top and bottom bars.

- $A_{s,min} = 2 \emptyset 14 \rightarrow (308 \text{ mm}^2)$

- Minimum A_s top bars in the span.

- $A_{s,min,top \text{ span}} = 0.25 A_{s,top \text{ supports}} = 0.25 * 6.03 * 10^{-4} = 1.50 * 10^{-4} \text{ m}^2 \rightarrow^{Use} 2\emptyset 12.$

- Minimum A_s bottom bars in critical regions

- $A_{s,min} = 0.5 * A_{s,top}$

For Beam 1 at Section [1-1]

- $A_{s,min} = 0.5 * 4.27 * 10^{-4} = 2.135 * 10^{-4} \text{ m}^2 \rightarrow^{use} A_{s,min} = 2 \emptyset 12 \cong 2.26 * 10^{-4} \text{ m}^2$

For Beam 1 at Section [3-3] and For Beam 2 at Section [4-4]

- $A_{s,min} = 0.5 * 6.03 * 10^{-4} = 3.015 * 10^{-4} \text{ m}^2 \rightarrow^{use} A_{s,min} = 1 \emptyset 16 + 1 \emptyset 12 \cong 3.14 * 10^{-4} \text{ m}^2$

- Maximum A_s in critical region

- $\rho_{max} = \rho' + 0.0018 f_{cd} / (\mu_{\Phi} * \epsilon_{yd} * f_{yd})$

- For Beam 1 at Section [1-1]

- $A_s' = 2 \emptyset 12 = 2.26 * 10^{-4}$

- $\rho' = \frac{A_s'}{b * d} = \frac{2.26 * 10^{-4}}{0.25 * 0.45} = 2 * 10^{-3}$

- $\mu_{\Phi} = 2q_o - 1 = (2 * 5.85) - 1 = 10.7$

- $\epsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{348 * 10^3}{200 * 10^6} = 1.74 * 10^{-3}$

- $\rho_{max} = (2 * 10^{-3}) + [(0.0018 * 16.667 * 10^3)/(10.7 * 1.74 * 10^{-3} * 348 * 10^3)] = 6.63 * 10^{-3}$.
- $A_{s,max} = \rho_{max} * b * d = 6.63 * 10^{-3} * 0.25 * 0.45 = 7.45 * 10^{-4} m^2$.
- For Beam 1 at Section [3-3] and For Beam 2 at Section [4-4]
 - $A_s = 1 \emptyset 16 + 1 \emptyset 12 = 3.14 * 10^{-4}$
 - $\rho = \frac{A_s}{b*d} = \frac{3.14*10^{-4}}{0.25*0.45} = 2.8 * 10^{-3}$
 - $\mu_{\Phi} = 2q_o - 1 = (2 * 5.85) - 1 = 10.7$
 - $\varepsilon_{yd} = \frac{f_{yd}}{E_s} = \frac{348*10^3}{200*10^6} = 1.74 * 10^{-3}$
 - $\rho_{max} = (2.8 * 10^{-3}) + [(0.0018 * 16.667 * 10^3)/(10.7 * 1.74 * 10^{-3} * 348 * 10^3)] = 7.43 * 10^{-3}$.
 - $A_{s,max} = \rho_{max} * b * d = 7.43 * 10^{-3} * 0.25 * 0.45 = 8.36 * 10^{-4} m^2$.
- Anchorage Length
 - $L_{bd} = a_{tr} * [1 - 0.15((c_d/d_{bl}) - 1)(d_{bl}/4) f_{yd} / (2.25 f_{ctd} * a_{poor})]$
 - $a_{tr} = [1 - k(n_w * A_{sw} - A_{s,t,min}) / A_s] \geq 0.7$
 - $c_d = \min \left\{ \frac{a_{cover}}{2} * [b - 2 * a_{cover} - d_{bl}] / n \right\}$
 - $f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 Mpa$
- Section 1-1
 - For Top Reinforcement
 - For non-corner bars $\emptyset 12 \rightarrow [k = 0.05]$
 - $[a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 1.13 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.2825 * 10^{-4} m^2]$
 - $c_d = \min \left\{ \frac{a_{cover} = 0.025 m}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016 + 0.012)] / 2 \right\} = 0.0265 m \rightarrow c_d = 0.025 m$
 - $a_{tr} = 1 - \left[\frac{0.05(5*0.50*10^{-4} - 0.2825*10^{-4})}{1.13*10^{-4}} \right] = 0.924 \geq 0.7$.
 - $L_{bd} = 0.924 * [1 - 0.15((0.025/0.012) - 1)] * \frac{(0.012/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 0.7)} = 0.423 m \cong 0.45 m$.
 - For corner bars $\emptyset 16 \rightarrow [k = 0.1]$
 - $[a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 2.01 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4} m^2]$
 - $a_{tr} = [1 - 0.1(5 * 0.5 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.9 \geq 0.7$.
 - $L_{bd} = 0.9 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 0.7)} = 0.60 m$.
 - For Bottom Reinforcement
 - For corner bars $\emptyset 16 \rightarrow [k = 0.1]$

$$[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$$

$$\circ c_d = \min \left\{ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 m \rightarrow c_d = 0.025 m \right.$$

$$\circ a_{tr} = [1 - 0.1(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.90 \geq 0.7.$$

$$\circ L_{bd} = 0.90 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.42m.$$

For non-corner bars $\emptyset 16 \rightarrow [k = 0.05]$

$$[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$$

$$\circ c_d = \min \left\{ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 m \rightarrow c_d = 0.025 m \right.$$

$$\circ a_{tr} = [1 - 0.05(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.95 \geq 0.7.$$

$$\circ L_{bd} = 0.95 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.45m.$$

- Section 3-3, and Section 4-4

- For Top Reinforcement

For non-corner bars $\emptyset 16 \rightarrow [k = 0.05]$

$$\circ [a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 2.01 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4} m^2]$$

$$\circ c_d = \min \left\{ \frac{1}{2} * \left[\frac{a_{cover} = 0.025 m}{3} * [0.25 - (2 * 0.025) - (0.016 + 0.016 + 0.016 + 0.016)] \right] = 0.0226 m \right.$$

$$\circ a_{tr} = 1 - \left[\frac{0.05(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4})}{2.01 * 10^{-4}} \right] = 0.95 \geq 0.7.$$

$$\circ L_{bd} = 0.95 * [1 - 0.15((0.0226/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 0.7)} = 0.65 m.$$

For corner bars $\emptyset 16 \rightarrow [k = 0.1]$

$$\circ [a_{poor} = 0.7], [n_w = 5], [A_{sw} = 0.50 * 10^{-4} m^2], [A_s = 2.01 * 10^{-4} m^2], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4} m^2]$$

$$\circ a_{tr} = [1 - 0.1(5 * 0.5 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.9 \geq 0.7.$$

$$\circ L_{bd} = 0.9 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 0.7)} = 0.60m.$$

- For Bottom Reinforcement

For corner bars $\emptyset 16 \rightarrow [k = 0.1]$

$$[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$$

- $c_d = \min \left\{ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 \text{ m} \rightarrow c_d = 0.025 \text{ m} \right.$
- $a_{tr} = [1 - 0.1(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.90 \geq 0.7.$
- $L_{bd} = 0.90 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.42 \text{ m}.$

For non-corner bars $\emptyset 16 \rightarrow [k = 0.05]$

- $[a_{poor} = 1], [n_w = 5], [A_{sw} = 0.5 * 10^{-4}], [A_s = 2.016 * 10^{-4}], [A_{s,t,min} = 0.25 * A_s = 0.5025 * 10^{-4}].$
- $c_d = \min \left\{ \frac{1}{2} * [0.25 - (2 * 0.025) - (0.016 + 0.016)] = 0.053 \text{ m} \rightarrow c_d = 0.025 \text{ m} \right.$
- $a_{tr} = [1 - 0.05(5 * 0.50 * 10^{-4} - 0.5025 * 10^{-4}) / 2.01 * 10^{-4}] = 0.95 \geq 0.7.$
- $L_{bd} = 0.95 * [1 - 0.15((0.025/0.016) - 1)] * \frac{(0.016/4) * 348 * 10^3}{(2.25 * 1.213 * 10^3 * 1)} = 0.45 \text{ m}.$

Detailed Design of beams in Shear

$$V_{Rd,max} = 0.3 \times b_w \times Z \times \left[1 - \frac{f_{ck}}{250} \right] \times f_{cd} \times \sin 2\theta$$

$$[b_w = 0.25 \text{ m}], [Z = 0.405], [\theta = 45^\circ]$$

$$V_{Rd,max} = 0.3 * 0.25 * 0.405 * \left[1 - \frac{25}{250} \right] * 16.67 * 10^3 * \sin 90 = 455.6 \text{ Kn}$$

$$[V_{Ed,max} = 77.8 \text{ Kn}] < [V_{Rd,max} = 455.6]$$

\therefore The cross section of the beam is safe in shear.

$$V_{Rd,s} = \frac{A_{sh}}{S_h} * Z * f_{ywd} * \cot \theta$$

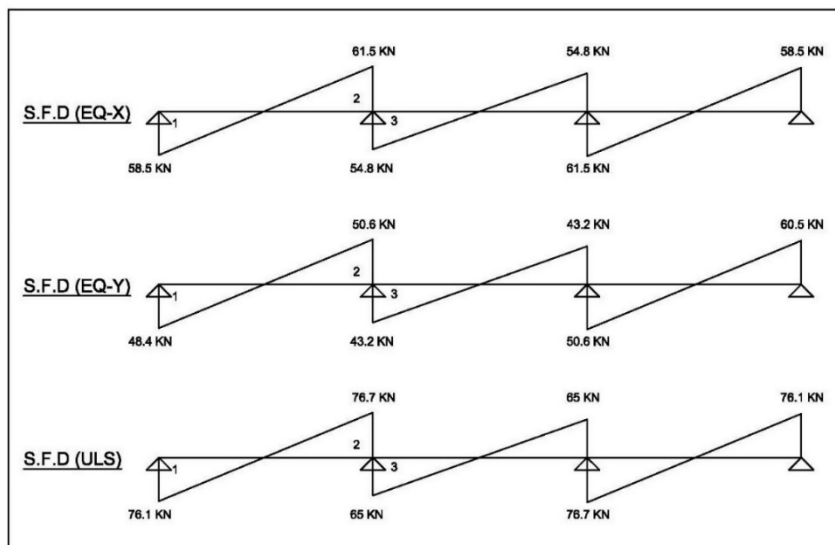


Figure C. 2 Shear force diagrams for DC H frame in seismic zone 1.3.

Detailing rules for the transverse reinforcement

- Outside Critical Regions
 - $s_h \leq 0.75 d = 0.75 * 0.45 = 0.3375 \text{ m.} \rightarrow^{use} s_h \leq 333.33 \text{ mm}$
 - $\rho_w = \frac{A_{sh}}{b_w * s_h} \geq [0.08 * \sqrt{f_{ck}} / f_{yk}]$.
 - $s_h \leq \frac{A_{sh}}{b_w * 0.08 * \frac{\sqrt{f_{ck}}}{f_{yk}}} \leq \frac{0.57 * 10^{-4}}{0.25 * 0.08 * \frac{\sqrt{25}}{400}} \leq 0.228 \text{ m.} \rightarrow^{use} s_h = 200 \text{ mm}$
 - $V_{Rd,s} = \frac{0.57 * 10^{-4}}{0.2} * 0.405 * \frac{400 * 10^3}{1.15} * \cot 45 = 40.16 \text{ kn.}$
- In Critical Regions
 - $d_{bw} \geq 6 \text{ mm.}$
 - $s_h = \min \begin{cases} 6d_{bl} = 6 * 12 = 72 \text{ mm} \\ \frac{h}{4} = 125 \text{ mm} \\ 24d_{bw} = 24 * 6 = 144 \text{ mm} \\ 175 \text{ mm} \end{cases} \therefore s_h = 72 \text{ mm} \rightarrow^{use} s_h = 71.43 \text{ mm}$
 - $V_{Rd,s} = \frac{0.57 * 10^{-4}}{0.07143} * 0.405 * \frac{400 * 10^3}{1.15} * \cot 45 = 112.46 \text{ kn.}$

Design of Columns DCH

Detailed Design of Columns in Flexure

Design of Exterior Columns in bending

$$N_{Ed} = 623 \text{ Kn}$$

$$b_c = 0.35 \text{ m}, h_c = 0.35 \text{ m}$$

$$a_{cover} = 0.025 \text{ m}$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.35 - 0.039 = 0.311 \text{ m.}$$

$$V_d = \frac{N_{Ed}}{b * d * f_{cd}} = \frac{623}{0.35 * 0.311 * \frac{25 * 10^3}{1.5}} = 0.343 \leq 0.55$$

$$M_{RC} \geq 1.3 * M_{Rb}$$

$$M_{Rb,m} = 60.15 \text{ Kn.m} \rightarrow M_{RC} = 1.3 * 60.15 = 78.2 \text{ Kn.m}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{78.2}{0.35 * 0.311^2 * \frac{25 * 10^3}{1.5}} = 0.138$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.311} = 0.125$$

$$V_1 = \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} + \varepsilon_{yd}} = \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) + \frac{348 * 10^3}{200 * 10^6}} = 0.54$$

$$V_2 = \delta_1 * \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} - \varepsilon_{yd}} = 0.125 * \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) - \frac{348 * 10^3}{200 * 10^6}} = 0.20$$

$$[V_2 = 0.20] \leq [V_d = 0.343] \leq [V_1 = 0.54] \rightarrow \text{Case 1}$$

$$\xi = \frac{x}{d} = \frac{V_d}{1 - (\varepsilon_{c2}/3\varepsilon_{cu2})} = \frac{0.343}{1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}} = 0.424$$

$$(1 - \delta_1) * \omega_{1d} = \mu_d - \xi * \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}} \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4\varepsilon_{cu2}} * \xi \right) \right]$$

$$\begin{aligned} (1 - 0.125) * \omega_{1d} &= 0.138 - 0.424 \\ &* \left[\frac{1 - 0.424}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \left(\frac{1}{2} - 0.424 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.424 \right) \right] \\ &\rightarrow \omega_{1d} = 0.0314 \end{aligned}$$

$$\omega_{1d} = \frac{A_{s1}}{b*d} * \frac{f_{yd}}{f_{cd}} \rightarrow A_{s1} = 0.0314 * 0.35 * 0.311 * \frac{16.667}{348} = 1.71 * 10^{-4} m^2$$

$$A_{s,total} = 2 * A_{s1} = 2 * 1.71 * 10^{-4} = 3.42 * 10^{-4} m^2$$

Number of bars per side ≥ 3 bars

$$\text{Use } A_{s,total} = 8 \emptyset 12 = 9.04$$

$$\rho = \frac{A_s}{b * h} * 100 = 0.37 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 0.37 \% A_c] \leq [\rho_{min} = 1 \% A_c] \therefore \text{Not. Ok} \rightarrow \text{Use } A_{s,min}$$

$$A_{s,min} = \rho_{min} * b * h = 0.01 * 0.35 * 0.35 = 12.25 * 10^{-4} m^2 \rightarrow \text{Use } 4 \emptyset 16 + 4 \emptyset 12$$

$$A_{s,total} = 12.56 * 10^{-4} m^2 \rightarrow \rho = 1.025 \% A_c$$

Spacing along the perimeter of bars restrained by a tie corner or hook ≤ 150 mm.

For side h = 0.35 m, $n_{bars} = 2 \text{ bars } \emptyset 16 + 1 \text{ bar } \emptyset 12 \geq 3 \text{ bars}$

$$s_{restrainer} = \frac{[h - (2*d_1)]}{(n_{bars} - 1)} = \frac{0.35 - (2*0.039)}{(3 - 1)} = 0.136 m = 136 mm \leq 150 mm \therefore \text{Ok}$$

For side b = 0.35 m, $n_{bars} = 2 \text{ bars } \emptyset 16 + 1 \text{ bar } \emptyset 12 \geq 3 \text{ bars}$

$$s_{restrainer} = \frac{[b - (2*d_1)]}{(n_{bars} - 1)} = \frac{0.35 - (2*0.039)}{(3 - 1)} = 0.136 m = 136 mm \leq 150 mm \therefore \text{Ok}$$

All the column bars are restrained.

- Distribution of Longitudinal bars

$$A_{s1} = 5.15 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.102$$

$$A_{s2} = 5.15 * 10^{-4} \rightarrow \omega_{2d} = \frac{A_{s2}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.102$$

$$A_{sv} = 2.26 * 10^{-4} \rightarrow \omega_{vd} = \frac{A_{sv}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.043$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\varepsilon_{cu2} + \varepsilon_{yd}}{\varepsilon_{cu2} - \varepsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\varepsilon_{cu2} + \frac{\varepsilon_{c2}}{3}}{\varepsilon_{cu2} - \varepsilon_{yd}} \right)$$

$$V_2 = \frac{0.043}{1 - 0.125} * \left(0.125 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) + \left(0.125 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right) = 0.17$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\varepsilon_{cu2} - \varepsilon_{yd}}{\varepsilon_{cu2} + \varepsilon_{yd}} - \delta_1 \right) + \left(\frac{\varepsilon_{cu2} - \left(\frac{\varepsilon_{c2}}{3}\right)}{\varepsilon_{cu2} + \varepsilon_{yd}} \right) = \frac{0.043}{1 - 0.125} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.125 \right) + \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right) = 0.55$$

$$[V_2 = 0.17] \leq [V_d = 0.343] \leq [V_1 = 0.55] \rightarrow \text{Case 1}$$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}}\right) + 2\omega_{vd}} = \frac{(1 - 0.125) * (0.343) + (1 + 0.125) * 0.043}{(1 - 0.125) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}\right) + 2 * 0.043}$$

$$\xi = 0.44$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} * \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right] \right]$$

$$M_{Rd,c} = 0.35 * 0.311^2 * \frac{25 * 10^3}{1.5} * 0.44 \left[\frac{1 - 0.44}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * \left(\frac{1}{2} - 0.44 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.44 \right) + \frac{(1 - 0.125)(0.102 + 0.102)}{2} + \frac{0.043}{1 - 0.125} * \left[(0.44 - 0.125)(1 - 0.44) - \frac{1}{3} * \left(0.44 * \frac{348 * 10^3}{200 * 10^6} \right)^2 \right] \right] = 87.5 \text{ Kn.m}$$

$$[M_{Rd,c} = 87.5 \text{ Kn.m}] > [1.3 * M_{Rb} = 1.3 * 60.15 = 78.2 \text{ Kn.m}] \therefore \text{Ok}$$

Using CSIColumn software, to check the column for all biaxial equation for all the building, the results shown that the column was not safe and need to increase column longitudinal reinforcement for the column,

$$b_c = 35 \text{ cm}, h_c = 50 \text{ cm}$$

$$A_s = 43.96 \text{ cm}^2 \rightarrow 14 \text{ } \emptyset 20$$

Design of Interior Columns in bending

$$N_{Ed} = 1442 \text{ Kn}$$

$$b_c = 0.35 \text{ m}, h_c = 0.50 \text{ m}$$

$$a_{cover} = 0.025 \text{ m}$$

$$d_1 = a_{cover} + d_{bw} + d_{bl} = 0.025 + 0.008 + \frac{0.016}{2} = 0.039 \text{ m.}$$

$$d = h_c - d_1 = 0.50 - 0.039 = 0.461 \text{ m.}$$

$$V_d = \frac{N_{Ed}}{b * d * f_{cd}} = \frac{1442}{0.35 * 0.461 * \frac{25 * 10^3}{1.5}} = 0.536 \leq 0.55$$

$$M_{Rc} \geq 1.3 * M_{Rb}$$

$$M_{Rb,m} = 85 \text{ Kn.m} \rightarrow M_{Rc} = 1.3 * 85 = 110.5 \text{ Kn.m}$$

$$\mu_d = \frac{M}{b * d^2 * f_{cd}} = \frac{110.5}{0.35 * 0.461^2 * \frac{25 * 10^3}{1.5}} = 0.09$$

$$\delta_1 = \frac{d_1}{d} = \frac{0.039}{0.461} = 0.0846$$

$$V_1 = \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} + \varepsilon_{yd}} = \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) + \frac{348 * 10^3}{200 * 10^6}} = 0.54$$

$$V_2 = \delta_1 * \frac{\varepsilon_{cu2} - (\varepsilon_{c2}/3)}{\varepsilon_{cu2} - \varepsilon_{yd}} = 0.0846 * \frac{(3.5 * 10^{-3}) - \frac{2 * 10^{-3}}{3}}{(3.5 * 10^{-3}) - \frac{348 * 10^3}{200 * 10^6}} = 0.136$$

$$[V_2 = 0.136] \leq [V_d = 0.536] \leq [V_1 = 0.54] \rightarrow \text{Case 1}$$

$$\xi = \frac{x}{d} = \frac{V_d}{1 - (\varepsilon_{c2}/3\varepsilon_{cu2})} = \frac{0.536}{1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}} = 0.66$$

$$(1 - \delta_1) * \omega_{1d} = \mu_d - \xi * \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3\varepsilon_{cu2}} \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4\varepsilon_{cu2}} * \xi \right) \right]$$

$$(1 - 0.0846) * \omega_{1d}$$

$$= 0.09 - 0.66$$

$$* \left[\frac{1 - 0.66}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} \left(\frac{1}{2} - 0.66 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.66 \right) \right]$$

$$\omega_{1d} = -0.034 \rightarrow \text{There is no need for reinforcement, use } A_{s,min}$$

$$\rho_{min} = 1 \% A_c$$

$$A_{s,min} = \rho_{min} * b * h = 0.01 * 0.35 * 0.5 = 17.50 * 10^{-4} m^2 \rightarrow^{Use} 8 \emptyset 16 + 2 \emptyset 12$$

$$A_{s,total} = 18.34 * 10^{-4} m^2 \rightarrow \rho = 1.048 \% A_c$$

$$[\rho_{max} = 4 \% A_c] \geq [\rho = 1.048 \% A_c] \geq [\rho_{min} = 1 \% A_c] \therefore Ok$$

Number of bars per side ≥ 3 bars

Spacing along the perimeter of bars restrained by a tie corner or hook ≤ 150 mm.

For side $h = 0.50$ m, $n_{bars} = 4$ bars $\emptyset 16 \geq 3$ bars

$$s_{restrainer} = \frac{[h-(2*d_1)]}{(n_{bars}-1)} = \frac{0.50-(2*0.039)}{(4-1)} = 0.140 m = 140 mm \leq 150 mm \therefore Ok$$

For side $b = 0.35$ m, $n_{bars} = 2$ bars $\emptyset 16 + 1$ bar $\emptyset 12 \geq 3$ bars

$$s_{restrainer} = \frac{[b-(2*d_1)]}{(n_{bars}-1)} = \frac{0.35-(2*0.039)}{(3-1)} = 0.136 m = 136 mm \leq 150 mm \therefore Ok$$

All the column bars are restrained.

- Distribution of Longitudinal bars

$$A_{s1} = 5.15 * 10^{-4} \rightarrow \omega_{1d} = \frac{A_{s1}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.068$$

$$A_{s2} = 5.15 * 10^{-4} \rightarrow \omega_{2d} = \frac{A_{s2}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.068$$

$$A_{sv} = 2.26 * 10^{-4} \rightarrow \omega_{vd} = \frac{A_{sv}}{b * d} * \frac{f_{yd}}{f_{cd}} = 0.106$$

$$V_2 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\delta_1 * \frac{\epsilon_{cu2} + \epsilon_{yd}}{\epsilon_{cu2} - \epsilon_{yd}} - 1 \right) + \left(\delta_1 * \frac{\epsilon_{cu2} + \frac{\epsilon_{c2}}{3}}{\epsilon_{cu2} - \epsilon_{yd}} \right)$$

$$V_2 = \frac{0.106}{1 - 0.0846} * \left(0.0846 * \frac{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} - 1 \right) + \left(0.0846 * \frac{3.5 * 10^{-3} + \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}} \right)$$

$$V_2 = 0.05$$

$$V_1 = \omega_{2d} - \omega_{1d} + \frac{\omega_{vd}}{1 - \delta_1} * \left(\frac{\epsilon_{cu2} - \epsilon_{yd}}{\epsilon_{cu2} + \epsilon_{yd}} - \delta_1 \right) + \left(\frac{\epsilon_{cu2} - \left(\frac{\epsilon_{c2}}{3} \right)}{\epsilon_{cu2} + \epsilon_{yd}} \right)$$

$$= \frac{0.106}{1 - 0.0846} * \left(\frac{3.5 * 10^{-3} - \frac{348 * 10^3}{200 * 10^6}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} - 0.0846 \right) + \left(\frac{3.5 * 10^{-3} - \frac{2 * 10^{-3}}{3}}{3.5 * 10^{-3} + \frac{348 * 10^3}{200 * 10^6}} \right)$$

$$V_1 = 0.57$$

$$[V_2 = 0.136] \leq [V_d = 0.536] \leq [V_1 = 0.54] \rightarrow Case 1$$

$$\xi = \frac{(1 - \delta_1) * (V_d + \omega_{1d} - \omega_{2d}) + (1 + \delta_1)\omega_{vd}}{(1 - \delta_1) * \left(1 - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}}\right) + 2\omega_{vd}}$$

$$= \frac{(1 - 0.0846) * (0.536) + (1 + 0.0846) * 0.106}{(1 - 0.0846) * \left(1 - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}}\right) + 2 * 0.106}$$

$$\xi = 0.63$$

$$\frac{M_{Rd,c}}{b * d^2 * f_{cd}} = \xi \left[\frac{1 - \xi}{2} - \frac{\varepsilon_{c2}}{3 \varepsilon_{cu2}} * \left(\frac{1}{2} - \xi + \frac{\varepsilon_{c2}}{4 \varepsilon_{cu2}} \xi \right) + \frac{(1 - \delta_1)(\omega_{1d} + \omega_{2d})}{2} + \frac{\omega_{vd}}{1 - \delta_1} \right]$$

$$* \left[(\xi - \delta_1)(1 - \xi) - \frac{1}{3} * \left(\xi \frac{\varepsilon_{yd}}{\varepsilon_{cu2}} \right)^2 \right]$$

$$M_{Rd,c} = 0.35 * 0.461^2 * \frac{25 * 10^3}{1.5}$$

$$* 0.63 \left[\frac{1 - 0.63}{2} - \frac{2 * 10^{-3}}{3 * 3.5 * 10^{-3}} * \left(\frac{1}{2} - 0.63 + \frac{2 * 10^{-3}}{4 * 3.5 * 10^{-3}} * 0.63 \right) \right]$$

$$+ \frac{(1 - 0.0846)(0.068 + 0.068)}{2} + \frac{0.106}{1 - 0.0846}$$

$$* \left[(0.63 - 0.0846)(1 - 0.63) - \frac{1}{3} * \left(0.63 * \frac{348 * 10^3}{200 * 10^6 / 3.5 * 10^{-3}} \right)^2 \right]$$

$$= 213.5 \text{ Kn.m}$$

$$[M_{Rd,c} = 213.5 \text{ Kn.m}] > [1.3 * M_{Rb} = 1.3 * 85 = 110.5 \text{ Kn.m}] \therefore \text{Ok}$$

Detailed Design of Columns in Shear

Design of Exterior Columns in shear

$$V_{CD,Max} = 27 \text{ Kn}$$

$$V_{Rd,s} = \frac{z}{h_{cl}} * N_{Ed} + \rho_w * b_w * z * f_{ywd} * \cot \theta$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + v_d; \\ 2.5 * (1 - v_d). \end{array} \right) * b_w * z * \left[1 - \frac{f_{ck}}{250} \right] * f_{cd} * \sin 2\theta$$

$$z = h - (2 * d_1) = 0.35 - (2 * 0.039) = 0.272 \text{ m}$$

$$\theta = 45^\circ$$

$$b_w = b_c = 0.35$$

$$b_o = b_c - 2a_{cover} = 0.35 - (2 * 0.025) = 0.3$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + 0.343 = 1.343; \\ 2.5 * (1 - 0.343) = 1.64. \end{array} \right) * 0.35 * 0.272 * \left[1 - \frac{25}{250} \right] * \frac{25 * 10^3}{1.5} \\ * \sin(2 * 45)$$

$$V_{Rd,max} = 459 \text{ Kn.m}$$

$$h_{cl} = 3.2 - 0.5 = 2.7 \text{ m.}$$

$$\rho_w = \frac{A_{sh}}{b_w * s_h} \rightarrow \rho_w * b_w = \frac{A_{sh}}{s_h}$$

- Critical Region Length

$$L_{cr} \geq \max \left\{ \begin{array}{l} 1.5 * h_c = 1.5 * 0.35 = 0.525 \\ 1.5 * b_c = 1.5 * 0.35 = 0.525 \\ 0.60 \text{ m} \\ \frac{H_{cl}}{5} = \frac{3.2-0.5}{5} = 0.54 \text{ m} \end{array} \right. = 0.60 \text{ m.}$$

- Detailing for Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ 0.4 * \sqrt{\frac{f_{yd}}{f_{ywd}}} d_{bl} = 0.4 * \sqrt{\frac{348 * 10^3}{348 * 10^3}} * d_{bl} = 0.4 * 20 = 8 \text{ mm} \geq 8 \text{ mm.} \end{array} \right.$$

$$s_w \leq \min \left\{ \begin{array}{l} 6d_{bl} = 6 * 20 = 120 \text{ mm;} \\ b_o/3 = 300/3 = 100 \text{ mm;} \leq 120 \text{ mm.} \\ 125 \text{ mm.} \end{array} \right.$$

Use 14 $\emptyset 8 / m \rightarrow \emptyset 8 / 71.42 \text{ mm.}$

$$V_{Rd,s} = \frac{0.272}{2.7} * 623 + \frac{4 * 0.50 * 10^{-4}}{71.42 * 10^{-3}} * 0.272 * 348 * 10^3 * \cot 45 = 327.83 \text{ Kn} \\ \geq V_{CD,Max} = 26.5 \text{ Kn}$$

- Detailing outside Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ \frac{d_{bl}}{4} = \frac{20}{4} = 5 \text{ mm} \geq 8 \text{ mm.} \end{array} \right.$$

$$s_w \leq \min \left\{ \begin{array}{l} 20d_{bl} = 20 * 12 = 240 \text{ mm;} \\ h_c = 350 \text{ mm} \\ b_c = 350 \text{ mm} \\ 400 \text{ mm} \end{array} \right. \leq 240 \text{ mm.}$$

Use 5 $\emptyset 8 / m \rightarrow \emptyset 8 / 200 \text{ mm.}$

$$V_{Rd,s} = \frac{0.265}{2.7} * 623 + \frac{4 * 0.5 * 10^{-4}}{200 * 10^{-3}} * 0.265 * 348 * 10^3 * \cot 45 = 153.36 \text{ Kn}$$

$$\geq V_{CD,Max} = 26.5 \text{ Kn}$$

- Detailing for lap splices of bars with $d_{bl} \geq 14 \text{ mm}$

$$s_w \leq \min \begin{cases} 12d_{bl} = 12 * 20 = 240 \text{ mm}; \\ 0.6h_c = 0.6 * 350 = 210 \text{ mm}; \\ 0.6b_c = 0.6 * 350 = 210 \text{ mm}; \\ 240\text{mm}. \end{cases} \leq 210 \text{ mm.}$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd,s} = \frac{0.265}{2.7} * 623 + \frac{4 * 0.5 * 10^{-4}}{200 * 10^{-3}} * 0.265 * 348 * 10^3 * \cot 45 = 153.36 \text{ Kn}$$

- lap splices length

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{c_d}{d_{bl}} - 1 \right) \right] a_{tr} (d_{bl}/4) f_{yd} / (2.25 f_{ctd})$$

$$a_{tr} = 1 - k(2n_w A_{sw} - A_{s,t,min}) / A_s$$

$$[d_{bl} = 12\text{mm}; 16\text{mm}], [k = 0.05; 0.1], [A_{sw} = 0.28 * 10^{-4} \text{ m}^2], [n_w = 6 \text{ bars}], [A_{s,t,min} = A_s = 1.13 * 10^{-4}; 2.01 * 10^{-4}].$$

$$c_d = \min \begin{cases} \frac{1}{2} * \left[\frac{a_{cover}}{b} * \{b - 2 * a_{cover} - d_{bw} - d_{bl}\} / n \right] \\ \frac{1}{2} * \left[\frac{a_{cover}}{h} * \{h - 2 * a_{cover} - d_{bw} - d_{bl}\} / n \right] \end{cases}$$

c_d

$$= \min \begin{cases} \frac{1}{2} \left[\frac{a_{cover} = 0.025 \text{ m}}{\{0.35 - (2 * 0.025) - (0.006 * 2) - (0.016 + 0.016 + 0.012)\}} / 2 \right] = 0.061 \text{ m} \\ \frac{1}{2} \left[\{0.35 - (2 * 0.025) - (0.006 * 2) - (0.016 + 0.016 + 0.012)\} / 2 \right] = 0.061 \text{ m} \end{cases}$$

$$c_d = 0.025 \text{ m.}$$

$$f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

- For $d_{bl} = 12\text{mm} \rightarrow k = 0.10$

$$a_{tr} = 1 - 0.10 * \frac{(2 * 6 * 0.28 * 10^{-4}) - (1.13 * 10^{-4})}{(1.13 * 10^{-4})} = 0.80$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.012} - 1 \right) \right] * 0.80 * \frac{\frac{0.012}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.384 \text{ m} \cong 0.40 \text{ m}$$

- For $d_{bl} = 20\text{mm} \rightarrow k = 0.1$

$$a_{tr} = 1 - 0.1 * \frac{(2 * 6 * 0.50 * 10^{-4}) - (3.14 * 10^{-4})}{(3.14 * 10^{-4})} = 0.933$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.02} - 1 \right) \right] * 0.933 * \frac{\frac{0.016}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.649 \text{ m} \cong 0.65 \text{ m}$$

- Effective mechanical ratio

$$a * \omega_d \geq 30 \mu_\phi * v_d * \varepsilon_{yd} * \frac{b_c}{b_o} - 0.035$$

$$\mu_\phi = (2 * q_o) - 1 = (2 * 5.85) - 1 = 10.7$$

$$\mu_\phi^* = \frac{\mu_\phi}{\frac{2}{3} * q_o} = \frac{(2 * 5.85) - 1}{\frac{2}{3} * 5.85} = 2.74$$

$$a = \left(1 - \frac{s}{2b_o} \right) * \left(1 - \frac{s}{2h_o} \right) * \left(1 - \left\{ \frac{b_o}{[(n_h - 1)h_o]} + \frac{h_o}{[(n_b - 1)b_o]} \right\} / 3 \right)$$

$$b_o = (b_c - 2 a_{cover}) = 0.35 - (2 * 0.025) = 0.30 \text{ m.}$$

$$h_o = (h_c - 2 a_{cover}) = 0.35 - (2 * 0.025) = 0.30 \text{ m.}$$

$$[s = 0.20 \text{ m}], [b_o = 0.30 \text{ m}], [h_o = 0.30 \text{ m}], [n_b = 4], [n_h = 4]$$

$$a = \left(1 - \frac{0.20}{2 * 0.30} \right) * \left(1 - \frac{0.20}{2 * 0.30} \right) * \left(1 - \left\{ \frac{0.30}{[(4 - 1) * 0.30]} + \frac{0.30}{[(4 - 1)0.30]} \right\} / 3 \right) = 0.35$$

$$0.35 * \omega_d \geq 30 * 2.74 * 0.343 * 1.74 * 10^{-3} * 0.35 / 0.3 - 0.035$$

$$\omega_d = 0.185 \geq 0.08 \therefore O.k$$

- Effective mechanical ratio at the connection to the foundation

$$a * \omega_d \geq 30 \mu_\phi * v_d * \varepsilon_{yd} * \frac{b_c}{b_o} - 0.035$$

$$\mu_\phi = (2 * q_o) - 1 = (2 * 5.85) - 1 = 10.7$$

$$a = \left(1 - \frac{s}{2b_o} \right) * \left(1 - \frac{s}{2h_o} \right) * \left(1 - \left\{ \frac{b_o}{[(n_h - 1)h_o]} + \frac{h_o}{[(n_b - 1)b_o]} \right\} / 3 \right)$$

$$b_o = (b_c - 2 a_{cover}) = 0.35 - (2 * 0.025) = 0.30 \text{ m.}$$

$$h_o = (h_c - 2 a_{cover}) = 0.35 - (2 * 0.025) = 0.30 \text{ m.}$$

$$[s = 0.07142 \text{ m}], [b_o = 0.30 \text{ m}], [h_o = 0.30 \text{ m}], [n_b = 4], [n_h = 4]$$

$$a = \left(1 - \frac{0.07142}{2 * 0.30} \right) * \left(1 - \frac{0.07142}{2 * 0.30} \right) * \left(1 - \left\{ \frac{0.30}{[(4 - 1) * 0.30]} + \frac{0.30}{[(4 - 1)0.30]} \right\} / 3 \right) = 0.60$$

$$0.30 * \omega_d \geq 30 * 10.7 * 0.343 * 1.74 * 10^{-3} * 0.35 / 0.3 - 0.035$$

$$\omega_d = 0.42 \geq 0.08 \therefore O.k$$

Design of Interior Columns In shear

$$V_{CD,Max} = 20 \text{ Kn}$$

$$V_{Rd,s} = \frac{z}{h_{cl}} * N_{Ed} + \rho_w * b_w * z * f_{ywd} * \cot \theta$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + v_d; \\ 2.5 * (1 - v_d). \end{array} \right) * b_w * z * \left[1 - \frac{f_{ck}}{250} \right] * f_{cd} * \sin 2\theta$$

$$z = h - (2 * d_1) = 0.50 - (2 * 0.039) = 0.422 \text{ m}$$

$$\theta = 45^\circ$$

$$b_w = b_c = 0.35$$

$$b_o = b_c - 2a_{cover} = 0.35 - (2 * 0.025) = 0.3$$

$$V_{Rd,max} = 0.3 * \min \left(\begin{array}{l} 1.25; \\ 1 + 0.536 = 1.536; \\ 2.5 * (1 - 0.536) = 1.16. \end{array} \right) * 0.35 * 0.422 * \left[1 - \frac{25}{250} \right] * \frac{25 * 10^3}{1.5} * \sin(2 * 45)$$

$$V_{Rd,max} = 771 \text{ Kn.m}$$

$$h_{cl} = 3.2 - 0.5 = 2.7 \text{ m.}$$

$$\rho_w = \frac{A_{sh}}{b_w * s_h} \rightarrow \rho_w * b_w = \frac{A_{sh}}{s_h}$$

- Critical Region Length

$$L_{cr} \geq \max \left\{ \begin{array}{l} 1.5 * h_c = 1.5 * 0.50 = 0.75 \\ 1.5 * b_c = 1.5 * 0.35 = 0.525 \\ 0.60 \text{ m} \\ \frac{h_{cl}}{5} = \frac{3.2-0.5}{5} = 0.54 \text{ m} \end{array} \right. = 0.75 \text{ m.}$$

- Detailing for Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 0.4 * \sqrt{\frac{f_{yd}}{f_{ywd}}} d_{bl} = 0.4 * \sqrt{\frac{348 * 10^3}{348 * 10^3}} * d_{bl} = 0.4 * 20 = 8 \text{ mm} \\ 6 \text{ mm} \end{array} \right. \geq 8 \text{ mm.}$$

$$s_w \leq \min \left\{ \begin{array}{l} 6d_{bl} = 6 * 20 = 120 \text{ mm;} \\ b_o/3 = 300/3 = 100 \text{ mm;} \leq 100 \text{ mm.} \\ 125 \text{ mm.} \end{array} \right.$$

$$\text{Use } 10 \text{ } \emptyset 8 / \text{m} \rightarrow \emptyset 8 / 100 \text{ mm.}$$

$$V_{Rd.s} = \frac{0.422}{2.7} * 1442 + \frac{4 * 0.50 * 10^{-4}}{100 * 10^{-3}} * 0.422 * 348 * 10^3 * \cot 45$$

$$= 225.67 \text{ Kn} \geq V_{CD,Max} = 40 \text{ Kn}$$

- Detailing outside Critical Region

$$d_{bw} \geq \max \left\{ \begin{array}{l} 6 \text{ mm} \\ \frac{d_{bl}}{4} = \frac{20}{4} = 5 \text{ mm} \end{array} \right. \geq 6 \text{ mm}.$$

$$s_w \leq \min \left\{ \begin{array}{l} 20d_{bl} = 20 * 20 = 400 \text{ mm}; \\ h_c = 500 \text{ mm} \\ b_c = 350 \text{ mm} \\ 400 \text{ mm} \end{array} \right. \leq 350 \text{ mm}.$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd.s} = \frac{0.422}{2.7} * 1442 + \frac{4 * 0.50 * 10^{-4}}{200 * 10^{-3}} * 0.422 * 348 * 10^3 * \cot 45$$

$$= 372.2 \text{ Kn} \geq V_{CD,Max} = 20 \text{ Kn}$$

- Detailing for lap splices of bars with $d_{bl} \geq 14 \text{ mm}$

$$s_w \leq \min \left\{ \begin{array}{l} 12d_{bl} = 12 * 20 = 240 \text{ mm}; \\ 0.6h_c = 0.6 * 500 = 300 \text{ mm}; \\ 0.6b_c = 0.6 * 350 = 210 \text{ mm}; \\ 240 \text{ mm}. \end{array} \right. \leq 200 \text{ mm}.$$

Use 5 $\emptyset 6 / m \rightarrow \emptyset 6 / 200 \text{ mm}$.

$$V_{Rd.s} = \frac{0.422}{2.7} * 1442 + \frac{4 * 0.50 * 10^{-4}}{200 * 10^{-3}} * 0.422 * 348 * 10^3 * \cot 45$$

$$= 372.23 \text{ Kn}$$

- lap splices length

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{c_d}{d_{bl}} - 1 \right) \right] a_{tr} (d_{bl}/4) f_{yd} / (2.25 f_{ctd})$$

$$a_{tr} = 1 - k(2n_w A_{sw} - A_{s,t,min}) / A_s$$

$$[d_{bl} = 12 \text{ mm}; 16 \text{ mm}], [k = 0.1], [A_{sw} = 0.28 * 10^{-4} \text{ m}^2], [n_w = 6 \text{ bars}], [A_{s,t,min} = A_s = 1.13 * 10^{-4}; 2.01 * 10^{-4}].$$

$$c_d = \min \left\{ \begin{array}{l} \frac{1}{2} * \{ [b - 2 * a_{cover} - d_{bw} - d_{bl}] / n \} \\ \frac{1}{2} * \{ [h - 2 * a_{cover} - d_{bw} - d_{bl}] / n \} \end{array} \right.$$

$$c_d = \min \begin{cases} \frac{1}{2} [\{0.35 - (2 * 0.025) - (0.006 * 2) - ((2 * 0.016) + 0.012)\} / 2] = 0.061 \text{ m} \\ \frac{1}{2} [\{0.50 - (2 * 0.025) - (0.006 * 2) - (4 * 0.016)\} / 3] = 0.0623 \text{ m} \end{cases}$$

$$c_d = 0.025 \text{ m.}$$

$$f_{ctd} = 0.7 * \frac{f_{ctm}}{\gamma_c} = 0.7 * \frac{2.6}{1.5} = 1.213 \text{ Mpa}$$

$$\circ \text{ For } d_{bl} = 12 \text{ mm} \rightarrow k = 0.10$$

$$a_{tr} = 1 - 0.10 * \frac{(2 * 6 * 0.28 * 10^{-4}) - (1.13 * 10^{-4})}{(1.13 * 10^{-4})} = 0.80$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.012} - 1 \right) \right] * 0.80 * \frac{\frac{0.012}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)} = 0.384 \text{ m}$$

$$\cong 0.40 \text{ m}$$

$$\circ \text{ For } d_{bl} = 16 \text{ mm} \rightarrow k = 0.1$$

$$a_{tr} = 1 - 0.1 * \frac{(2 * 6 * 0.28 * 10^{-4}) - (2.01 * 10^{-4})}{(2.01 * 10^{-4})} = 0.933$$

$$l_o = 1.5 \left[1 - 0.15 \left(\frac{0.025}{0.016} - 1 \right) \right] * 0.933 * \frac{\frac{0.016}{4} * \frac{400 * 10^3}{1.15}}{(2.25 * 1.213 * 10^3)}$$

$$= 0.649 \text{ m} \cong 0.65 \text{ m}$$

- Effective mechanical ratio

$$a * \omega_d \geq 30 \mu_\Phi * v_d * \varepsilon_{yd} * \frac{b_c}{b_o} - 0.035$$

$$\mu_\Phi = (2 * q_o) - 1 = (2 * 5.85) - 1 = 10.7$$

$$\mu_\Phi^* = \frac{\mu_\Phi}{\frac{2}{3} * q_o} = \frac{(2 * 5.85) - 1}{\frac{2}{3} * 5.85} = 2.74$$

$$a = \left(1 - \frac{s}{2b_o} \right) * \left(1 - \frac{s}{2h_o} \right) * \left(1 - \left\{ \frac{b_o}{[(n_h - 1)h_o]} + \frac{h_o}{[(n_b - 1)b_o]} \right\} / 3 \right)$$

$$b_o = (b_c - 2 a_{cover}) = 0.35 - (2 * 0.025) = 0.30 \text{ m.}$$

$$h_o = (h_c - 2 a_{cover}) = 0.50 - (2 * 0.025) = 0.45 \text{ m.}$$

$$[s = 0.20 \text{ m}], [b_o = 0.30 \text{ m}], [h_o = 0.45 \text{ m}], [n_b = 4], [n_h = 4]$$

$$a = \left(1 - \frac{0.20}{2 * 0.30} \right) * \left(1 - \frac{0.20}{2 * 0.45} \right) * \left(1 - \left\{ \frac{0.30}{[(4 - 1) * 0.45]} + \frac{0.45}{[(4 - 1) * 0.30]} \right\} / 3 \right) = 0.393$$

$$0.393 * \omega_d \geq 30 * 2.74 * 0.343 * 1.74 * 10^{-3} * 0.35 / 0.3 - 0.035$$

$$\omega_d = 0.164 \geq 0.08 \therefore O.k$$

- Effective mechanical ratio at the connection to the foundation

$$a * \omega_d \geq 30 \mu_\phi * v_d * \varepsilon_{yd} * \frac{b_c}{b_o} - 0.035$$

$$\mu_\phi = (2 * q_o) - 1 = (2 * 5.85) - 1 = 10.7$$

$$a = \left(1 - \frac{s}{2b_o}\right) * \left(1 - \frac{s}{2h_o}\right) * \left(1 - \left\{\frac{b_o}{[(n_h - 1)h_o]} + \frac{h_o}{[(n_b - 1)b_o]}\right\}/3\right)$$

$$[s = 0.07142 \text{ m}], [b_o = 0.30 \text{ m}], [h_o = 0.45 \text{ m}], [n_b = 4], [n_h = 4]$$

$$a = \left(1 - \frac{0.07412}{2 * 0.30}\right) * \left(1 - \frac{0.07142}{2 * 0.45}\right) * \left(1 - \left\{\frac{0.30}{[(4 - 1) * 0.45]} + \frac{0.45}{[(4 - 1)0.30]}\right\}/3\right) = 0.61$$

$$0.61 * \omega_d \geq 30 * 10.7 * 0.343 * 1.74 * 10^{-3} * 0.35/0.3 - 0.035$$

$$\omega_d = 0.352 \geq 0.08 \therefore O.k$$

Appendix D: Detailed Drawing of the designed frames

The detailed drawings for the designed RC frames are in figures from D.1 to D.24, for DC L, DC M and DC H, for all studied seismic zones.

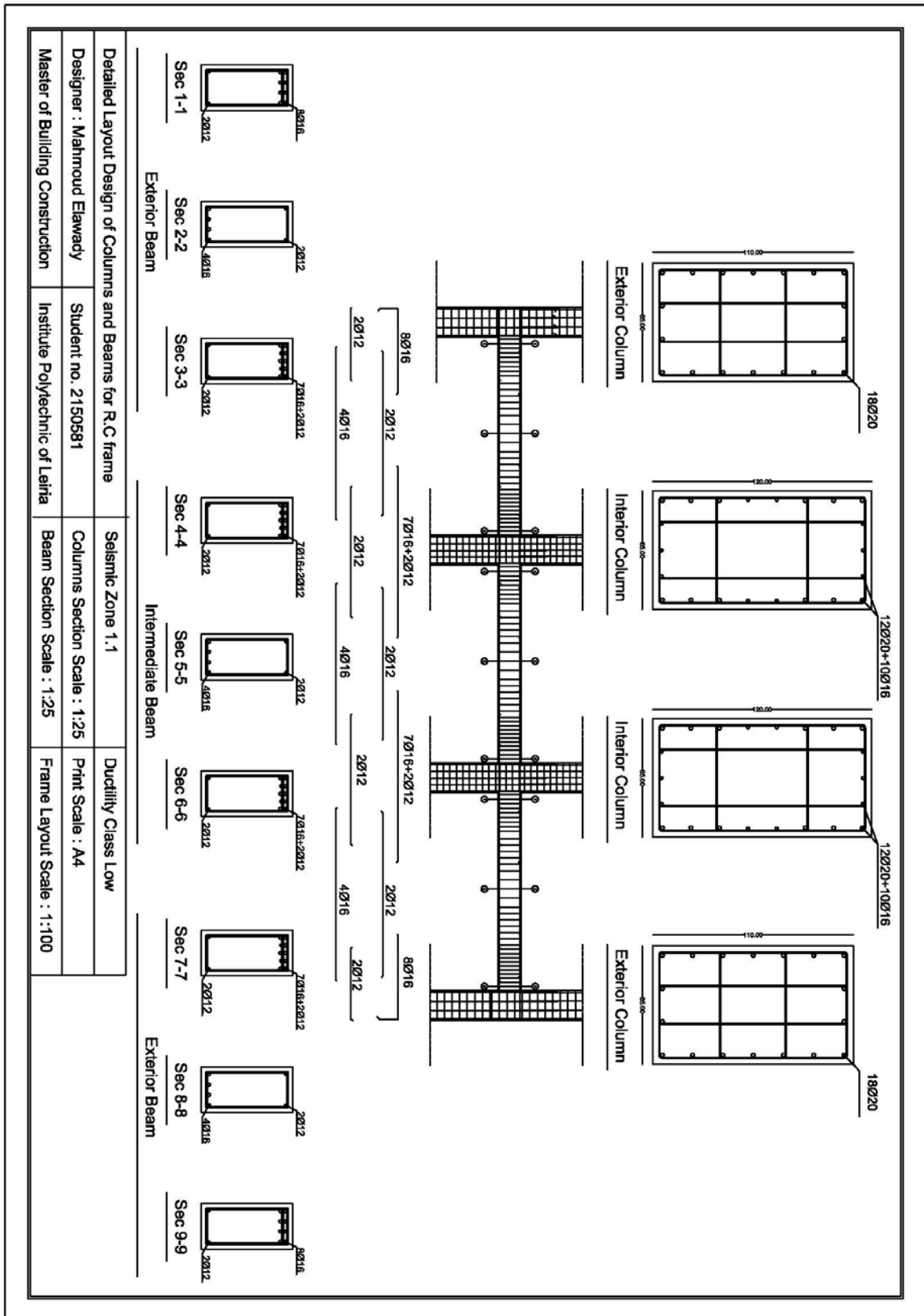


Figure D.1 Detailed Layout for DC L frame in seismic zone 1.1

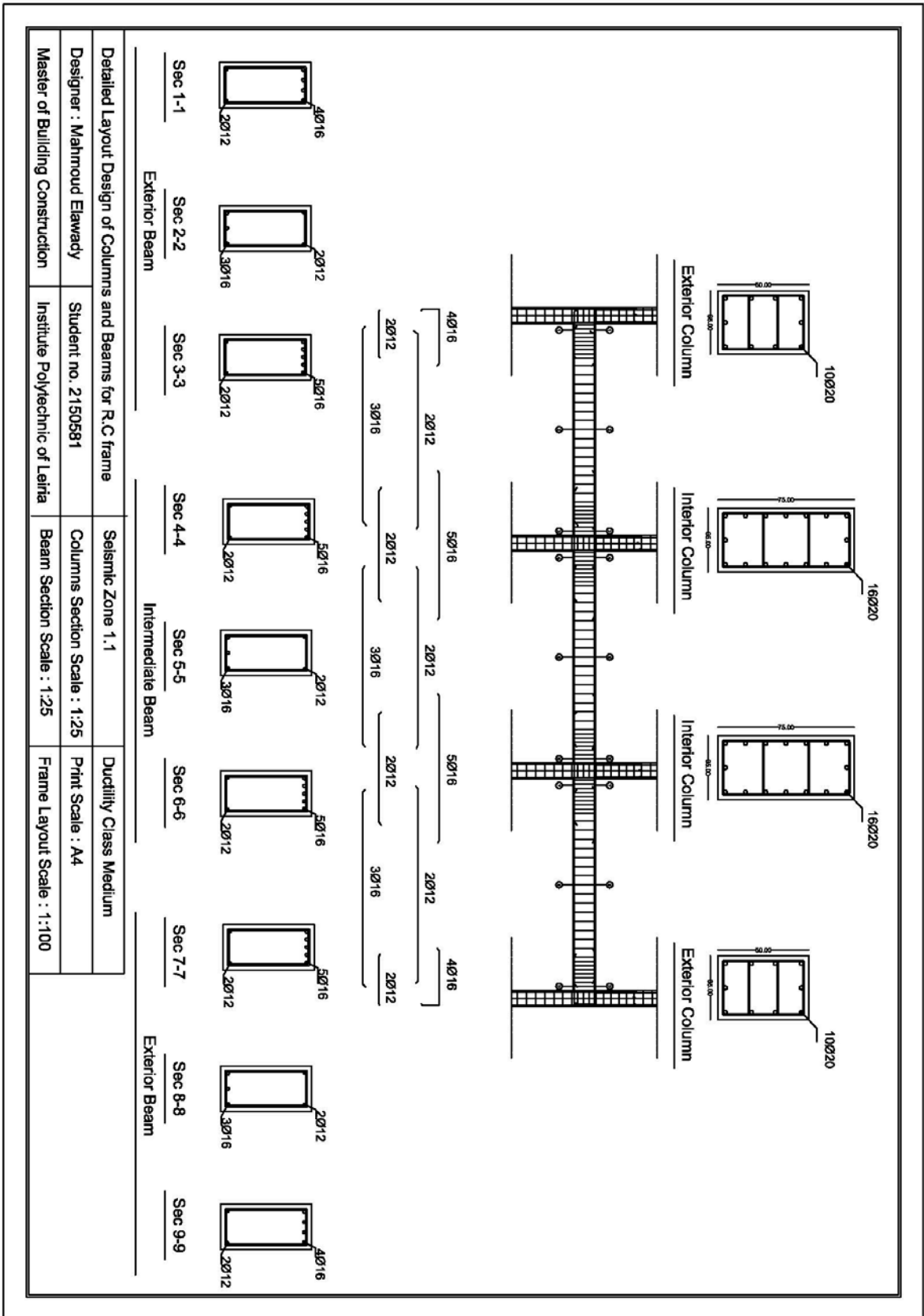


Figure D.2 Detailed Layout for DC M frame in seismic zone 1.1

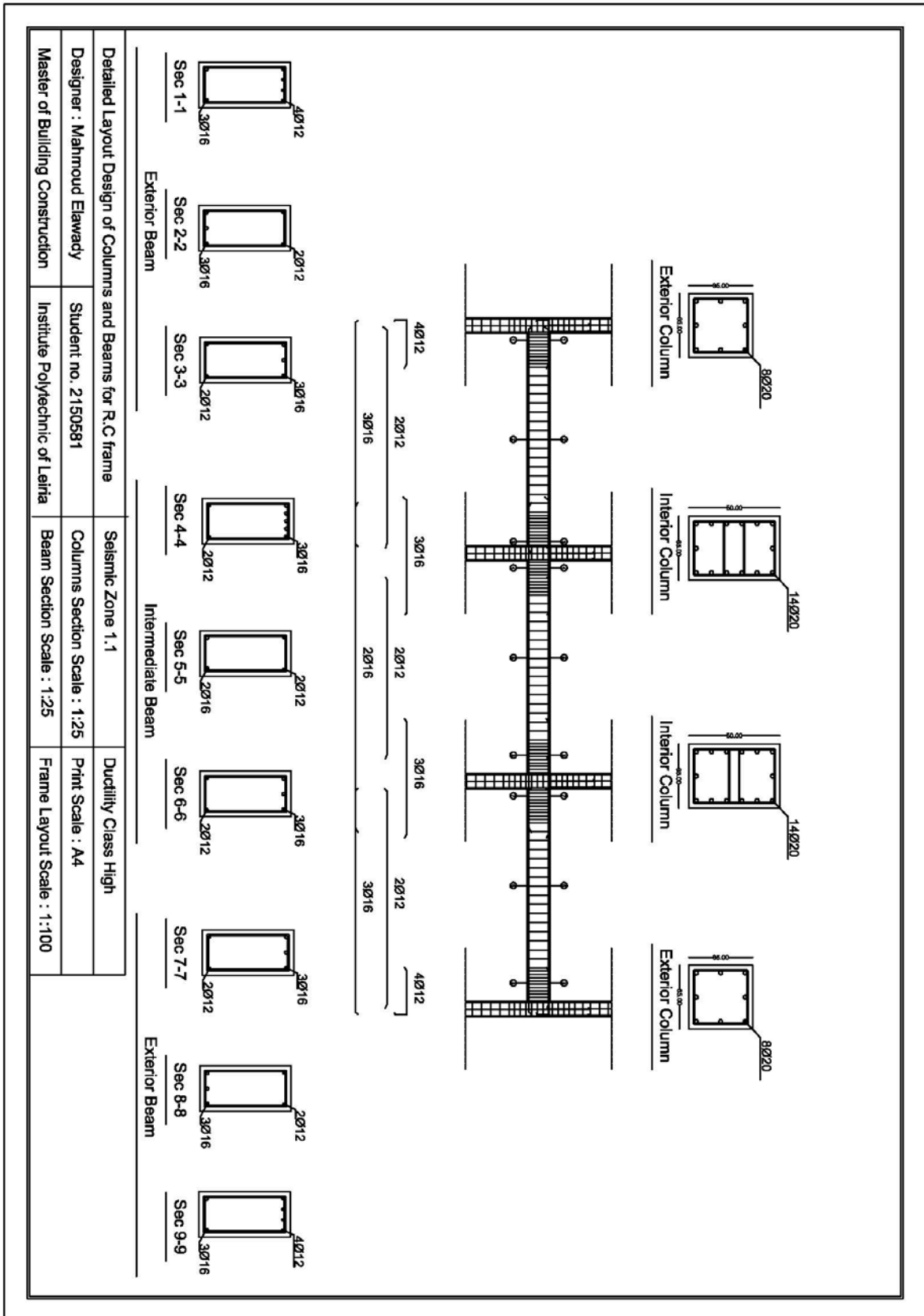


Figure D.3 Detailed Layout for DC H frame in seismic zone 1.1

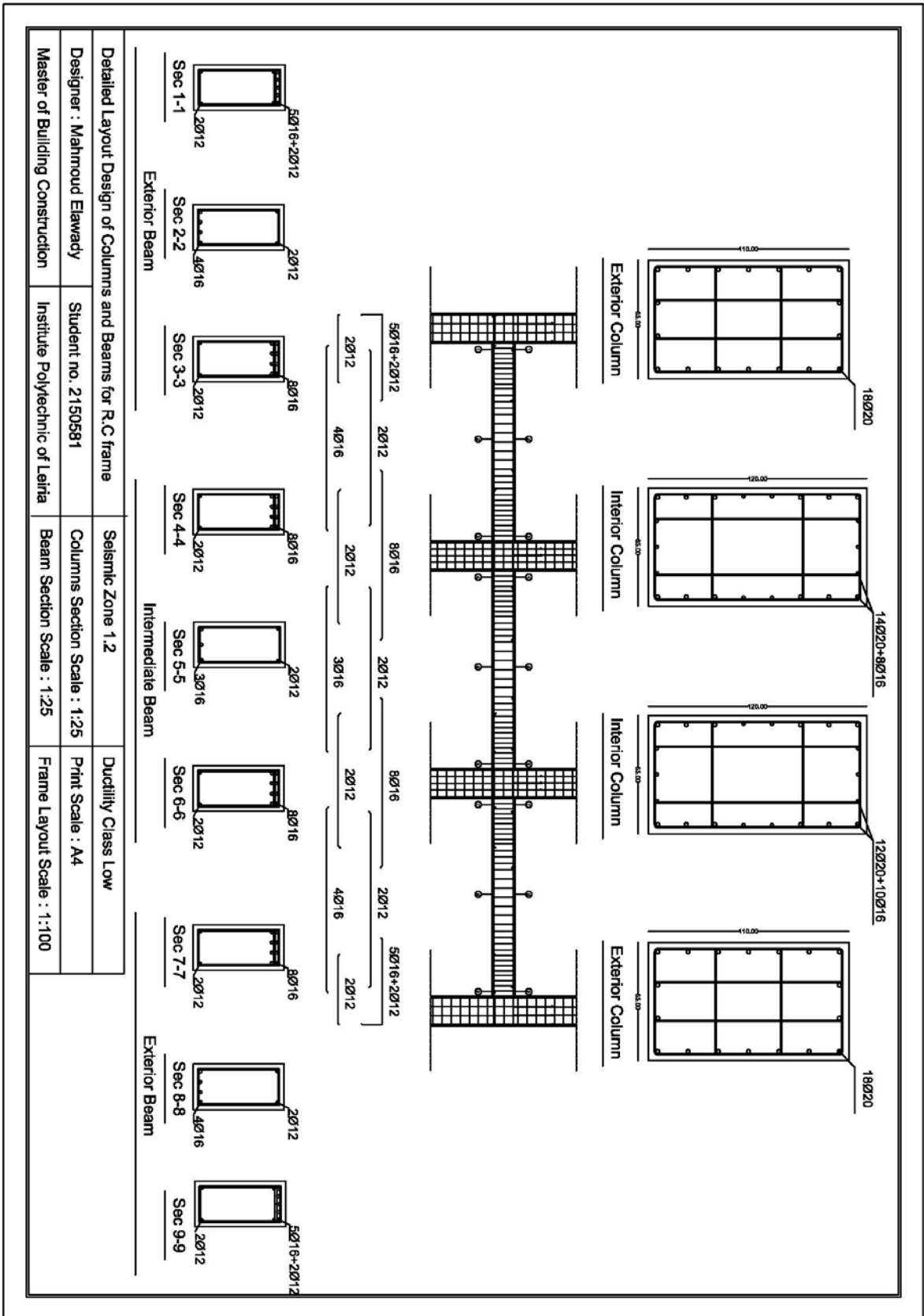


Figure D.4 Detailed Layout for DC L frame in seismic zone 1.2

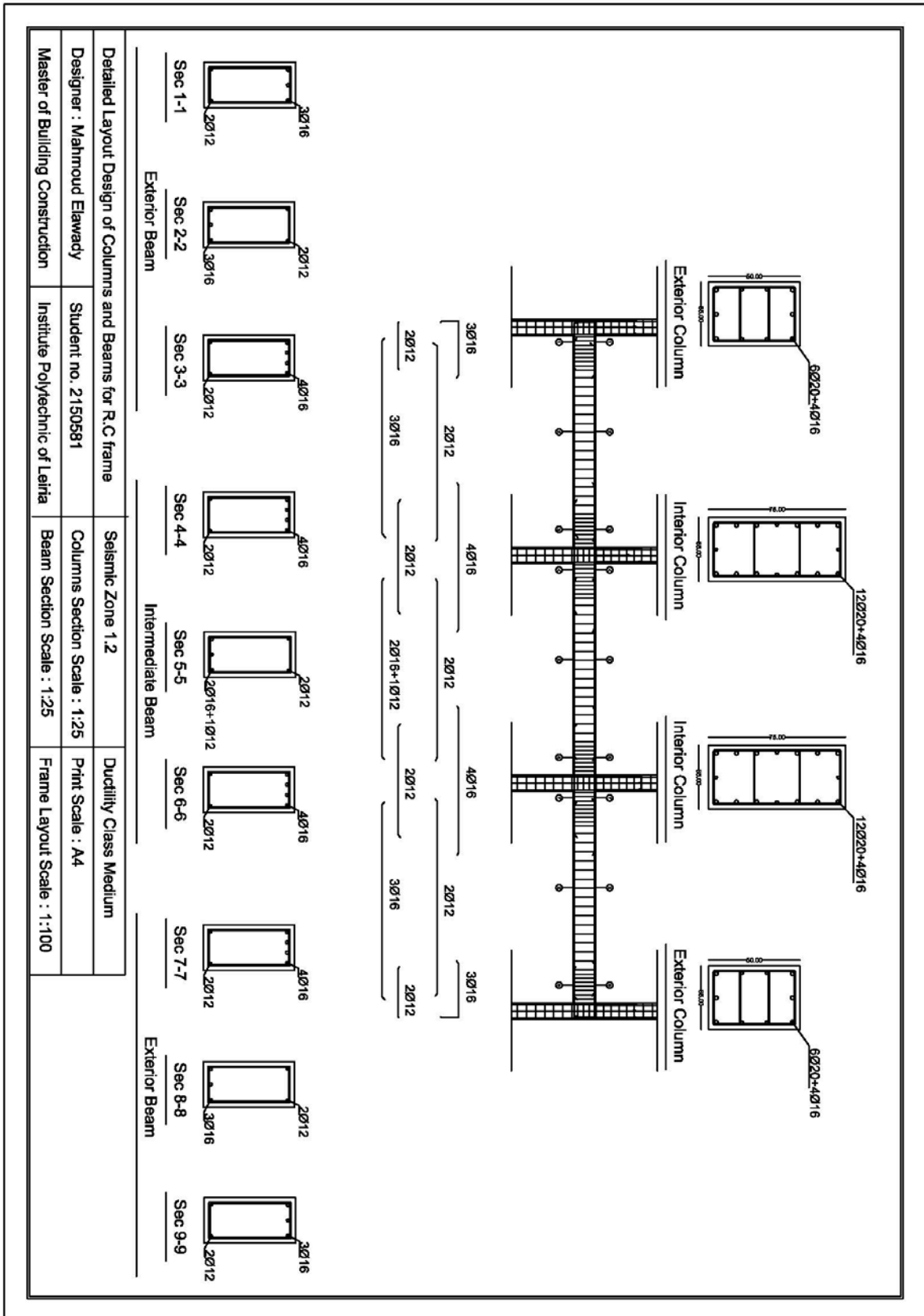


Figure D.5 Detailed Layout for DC M frame in seismic zone 1.2

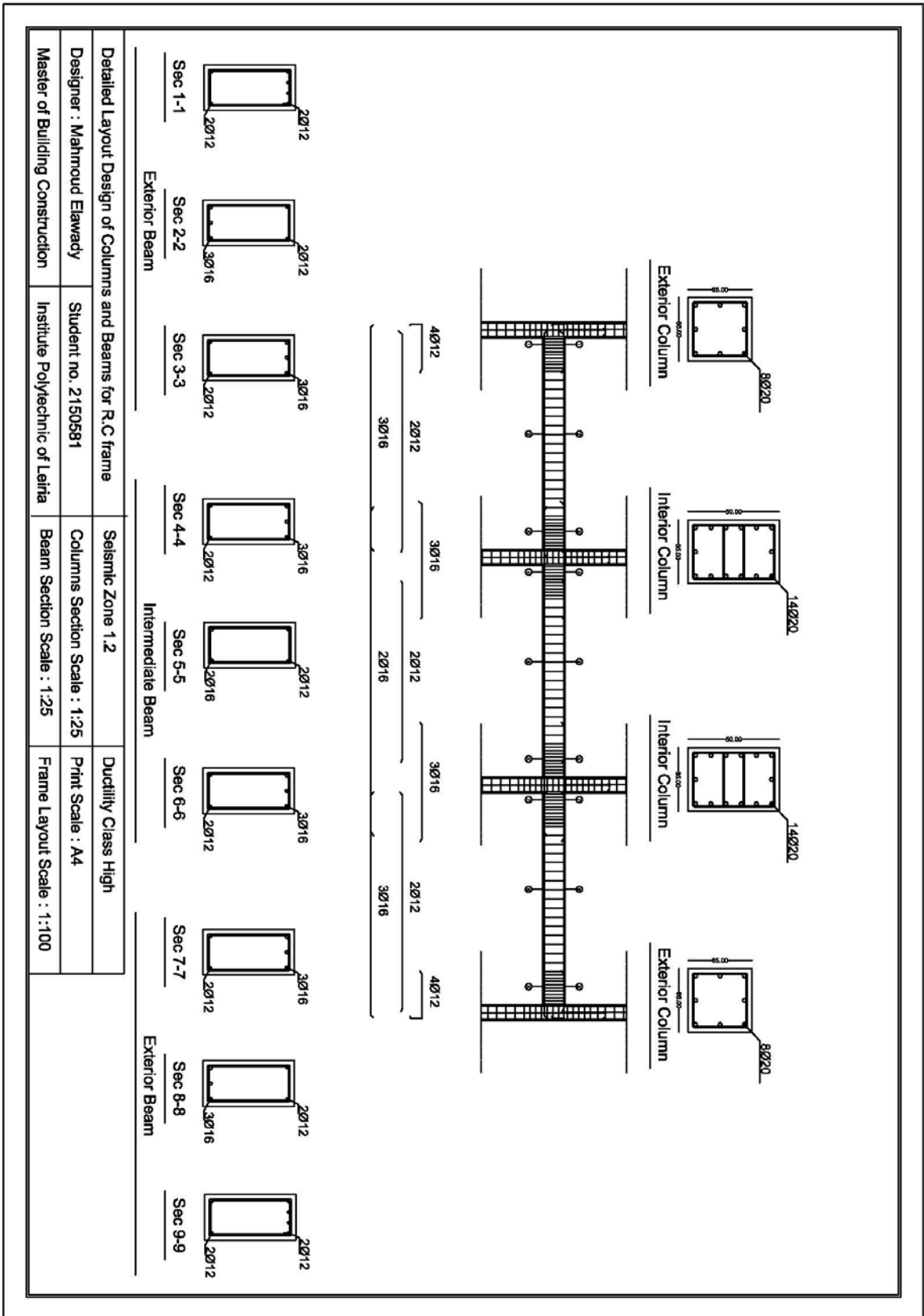


Figure D.6 Detailed Layout for DC H frame in seismic zone 1.2

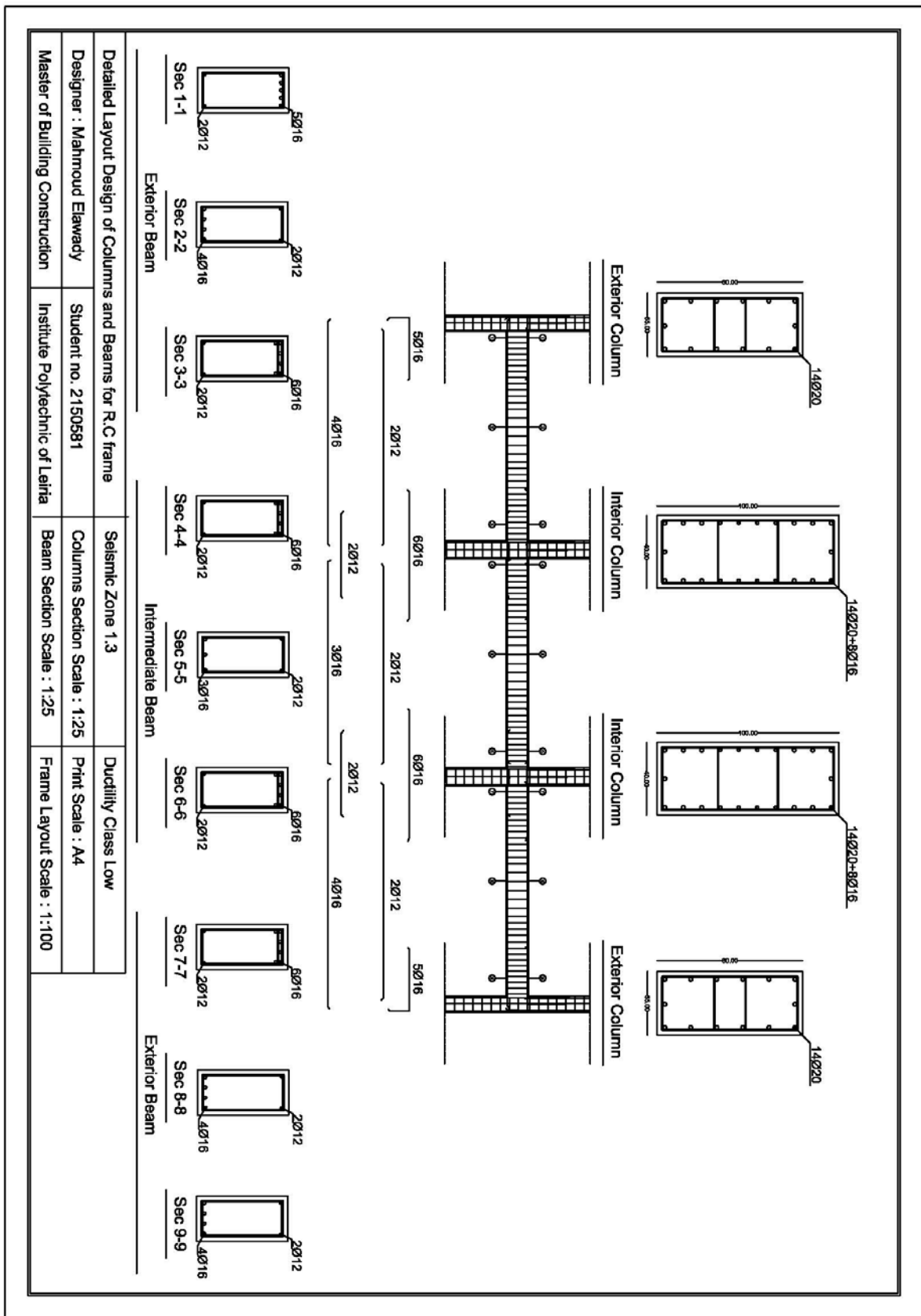


Figure D.7 Detailed Layout for DC L frame in seismic zone 1.3

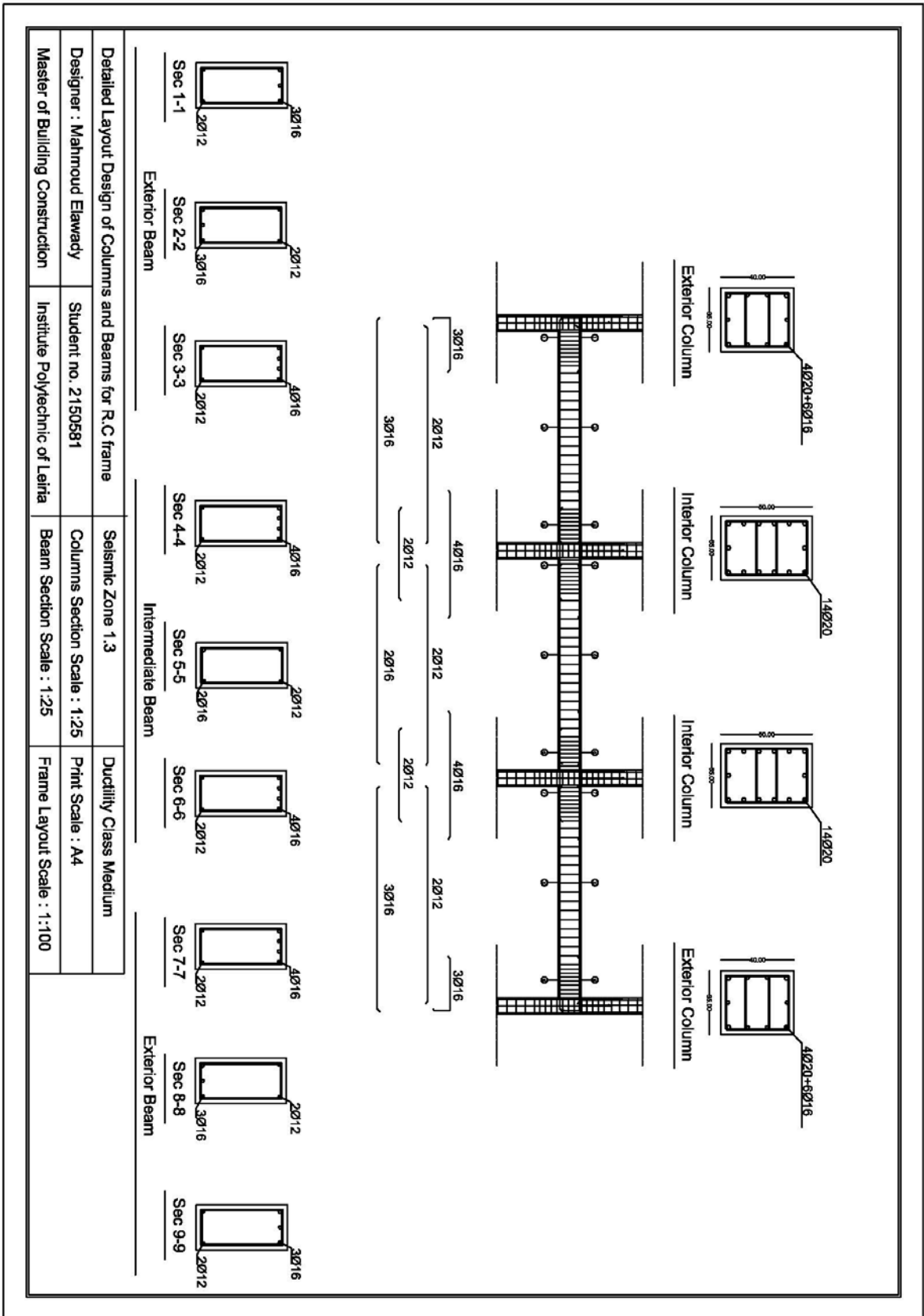


Figure D.8 Detailed Layout for DC M frame in seismic zone 1.3

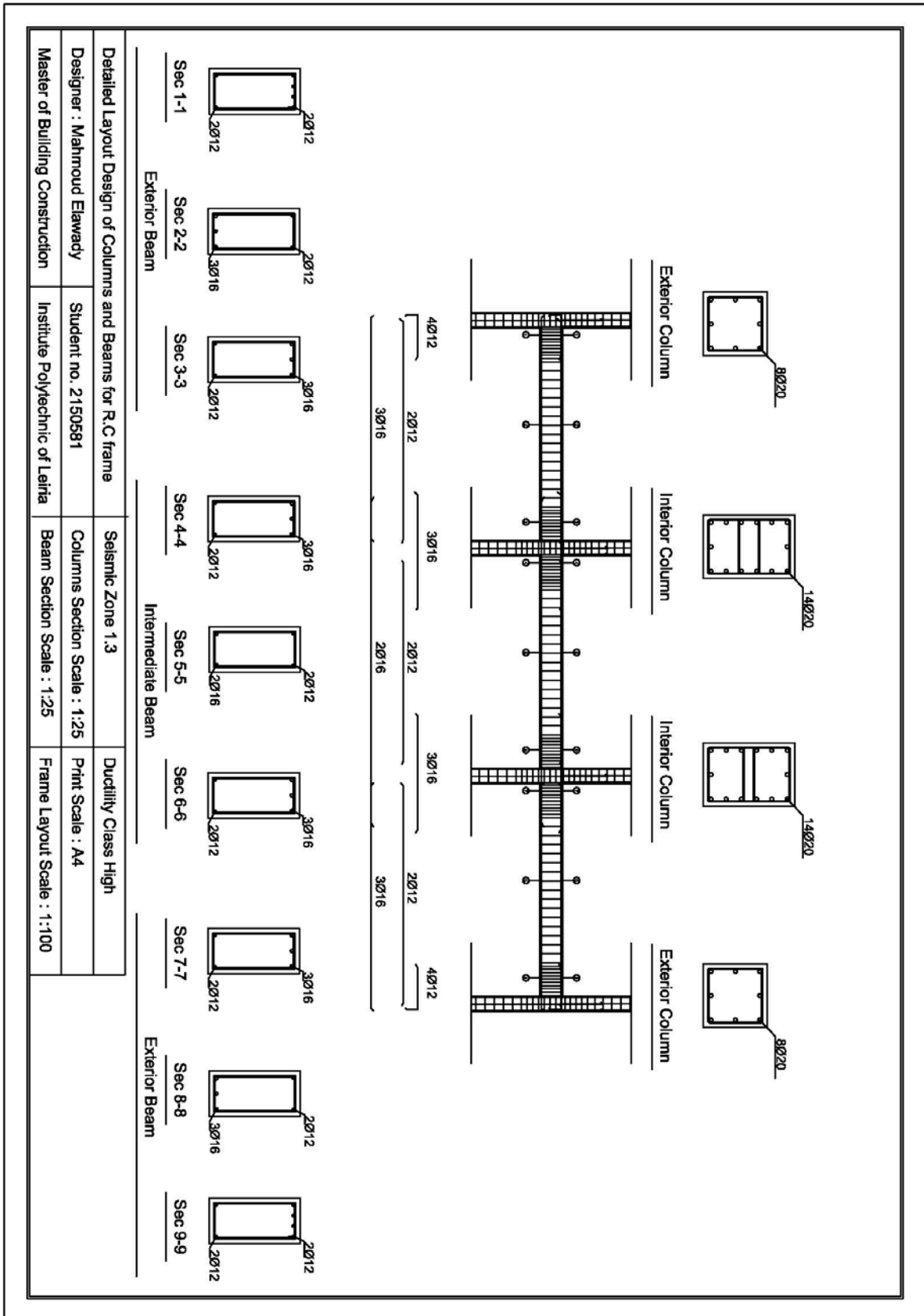


Figure D. 9 Detailed Layout for DC H frame in seismic zone 1.3

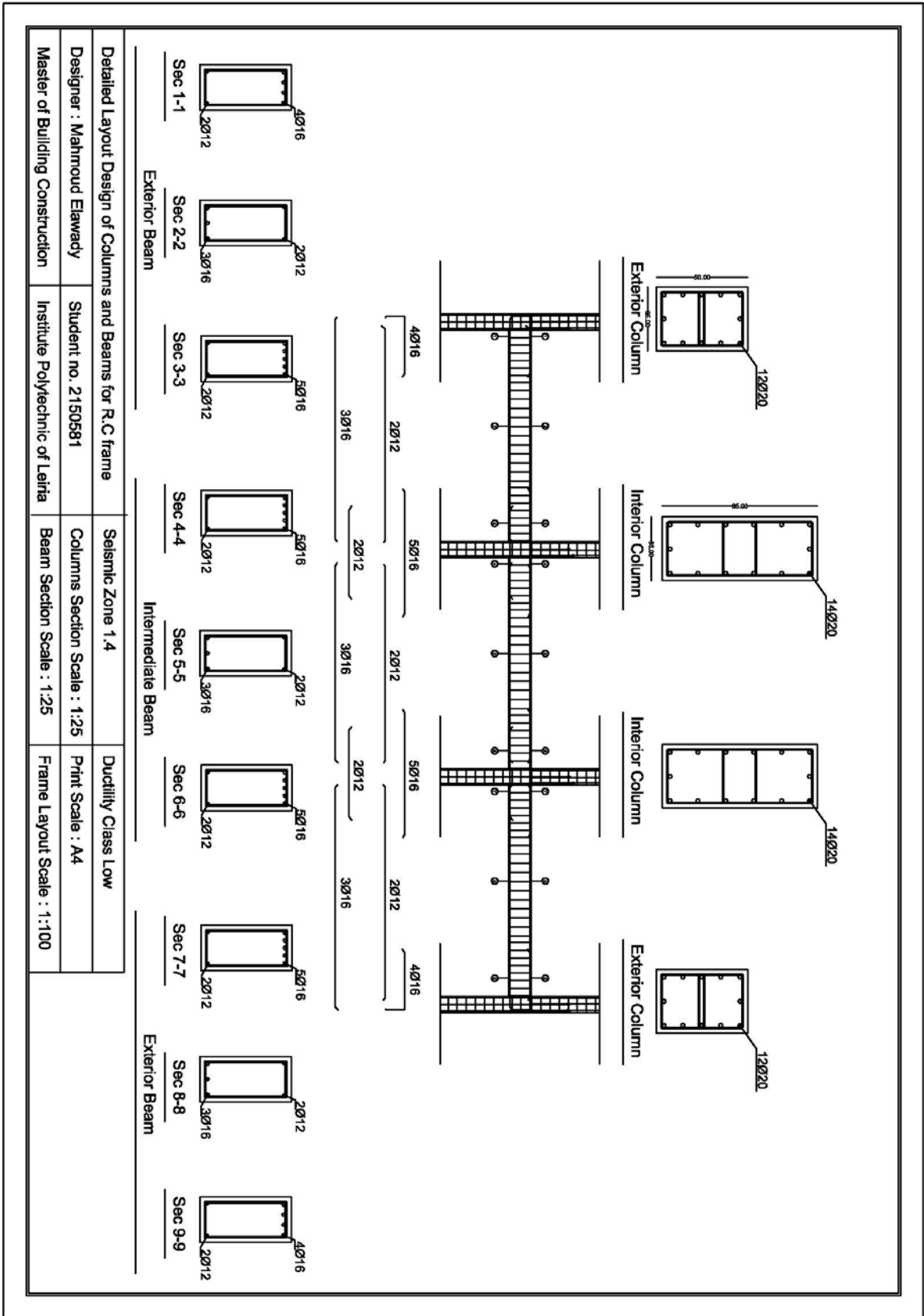


Figure D. 10 Detailed Layout for DC L frame in seismic zone 1.4

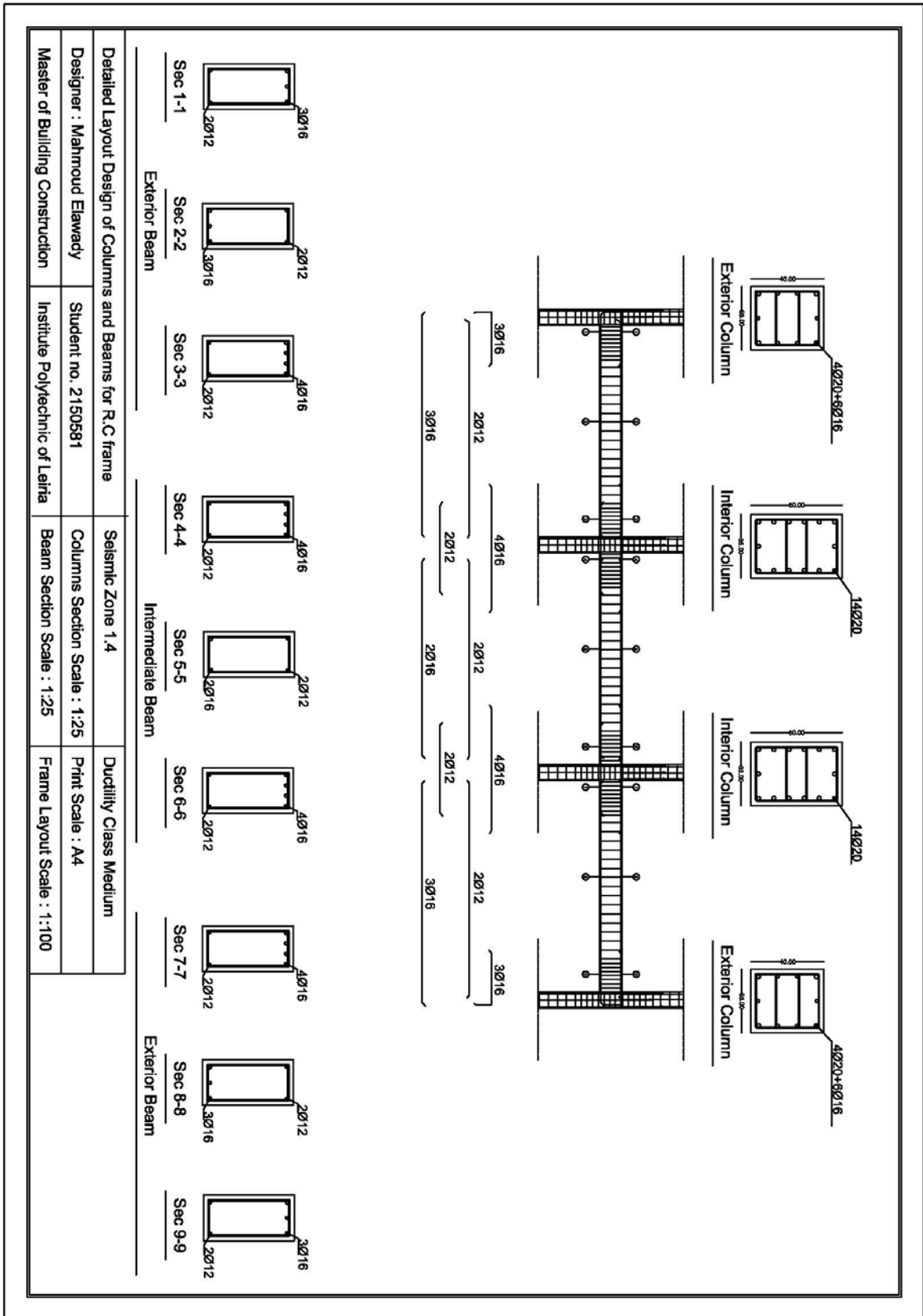


Figure D. 11 Detailed Layout for DC M frame in seismic zone 1.4

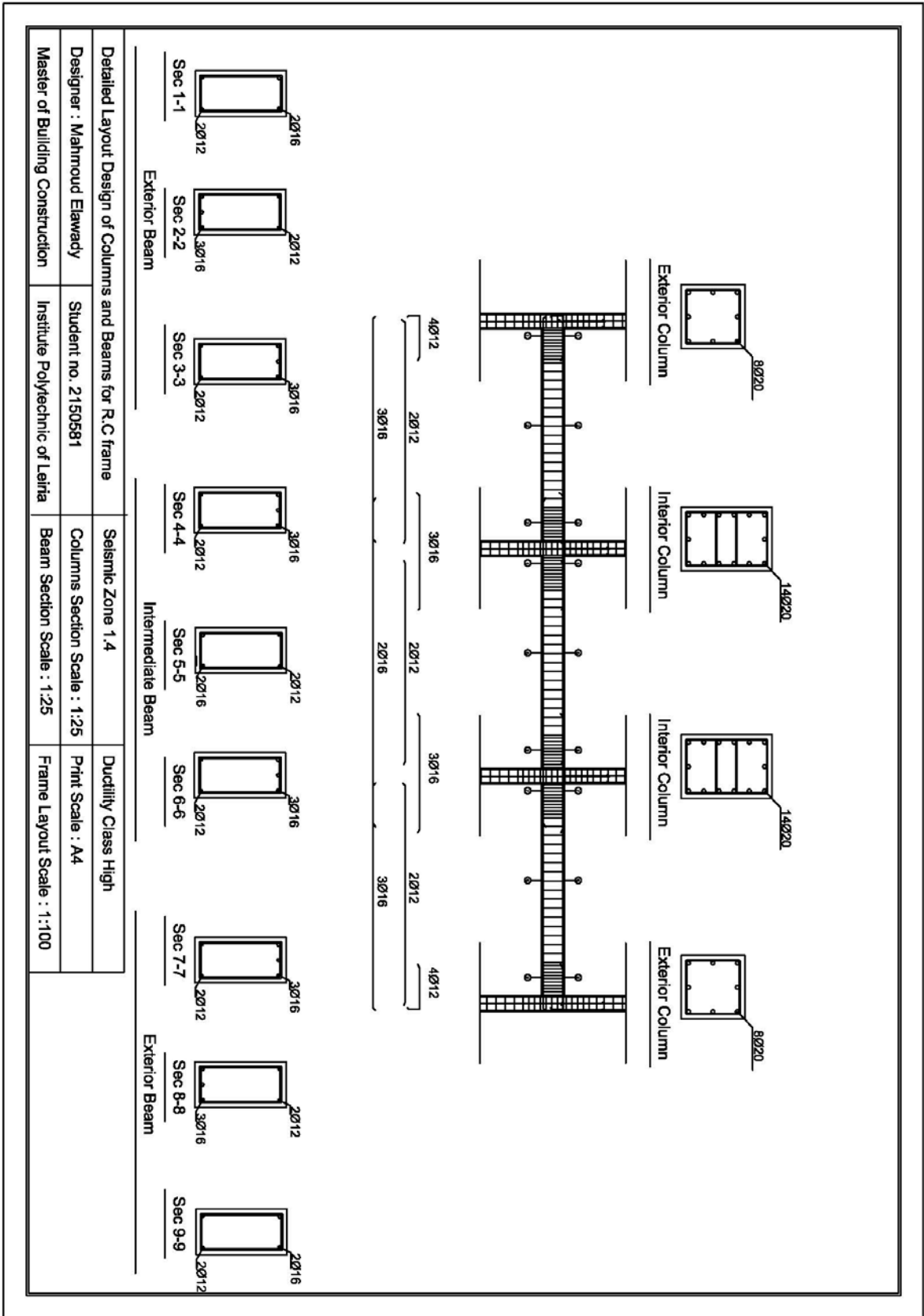


Figure D.12 Detailed Layout for DC H frame in seismic zone 1.4

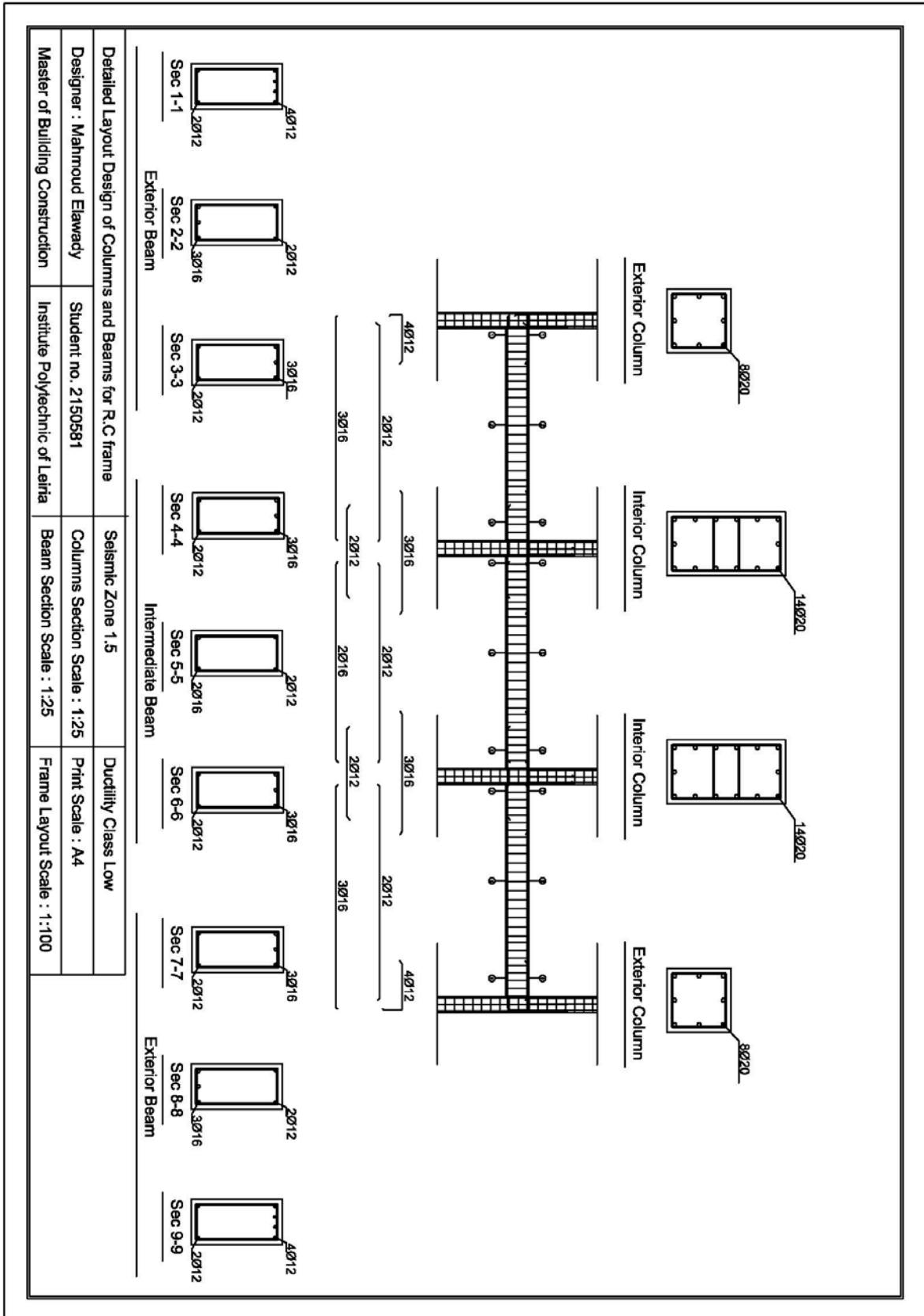


Figure D.13 Detailed Layout for DC L frame in seismic zone 1.5

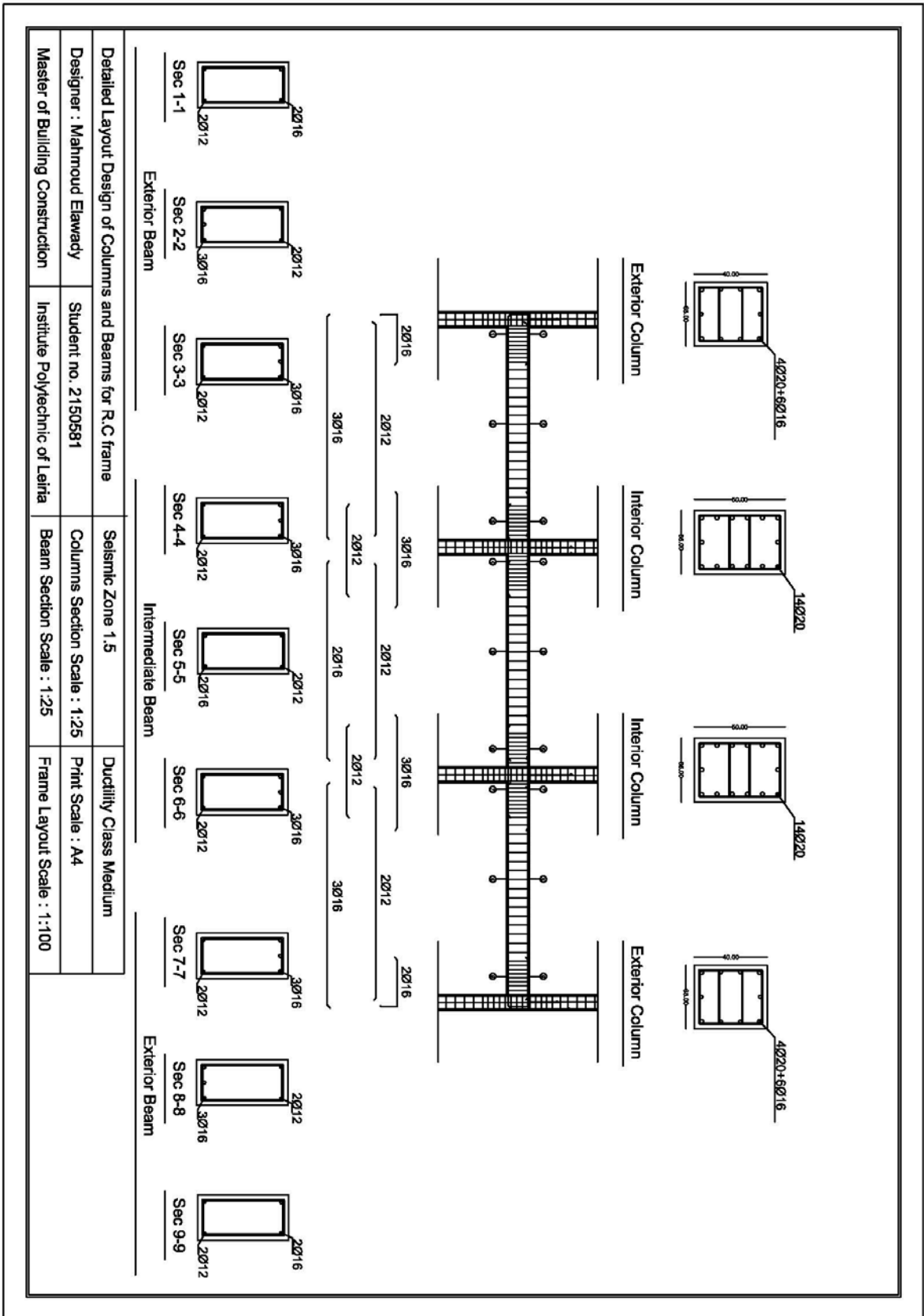


Figure D.14 Detailed Layout for DC M frame in seismic zone 1.5

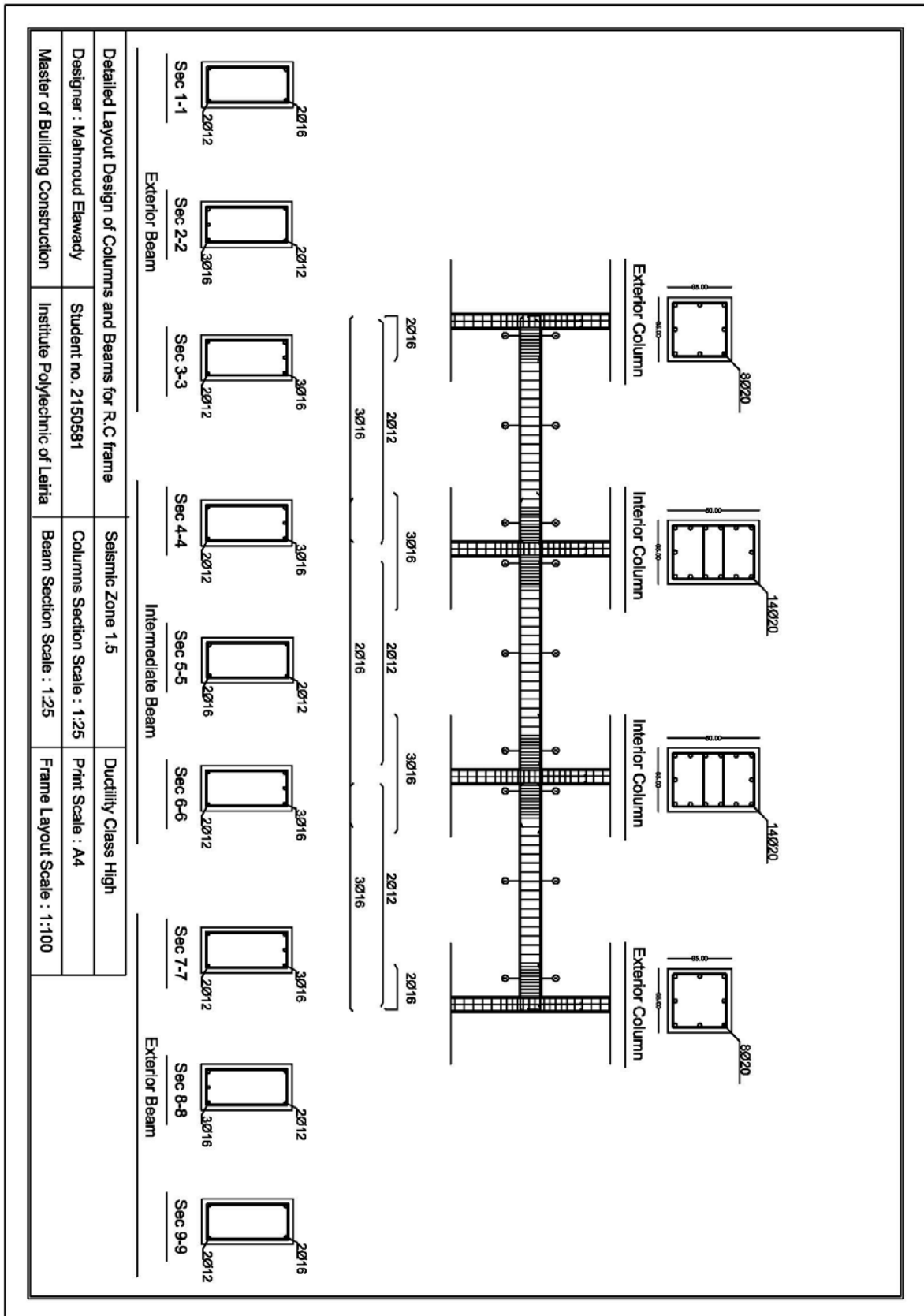


Figure D.15 Detailed Layout for DC H frame in seismic zone 1.5

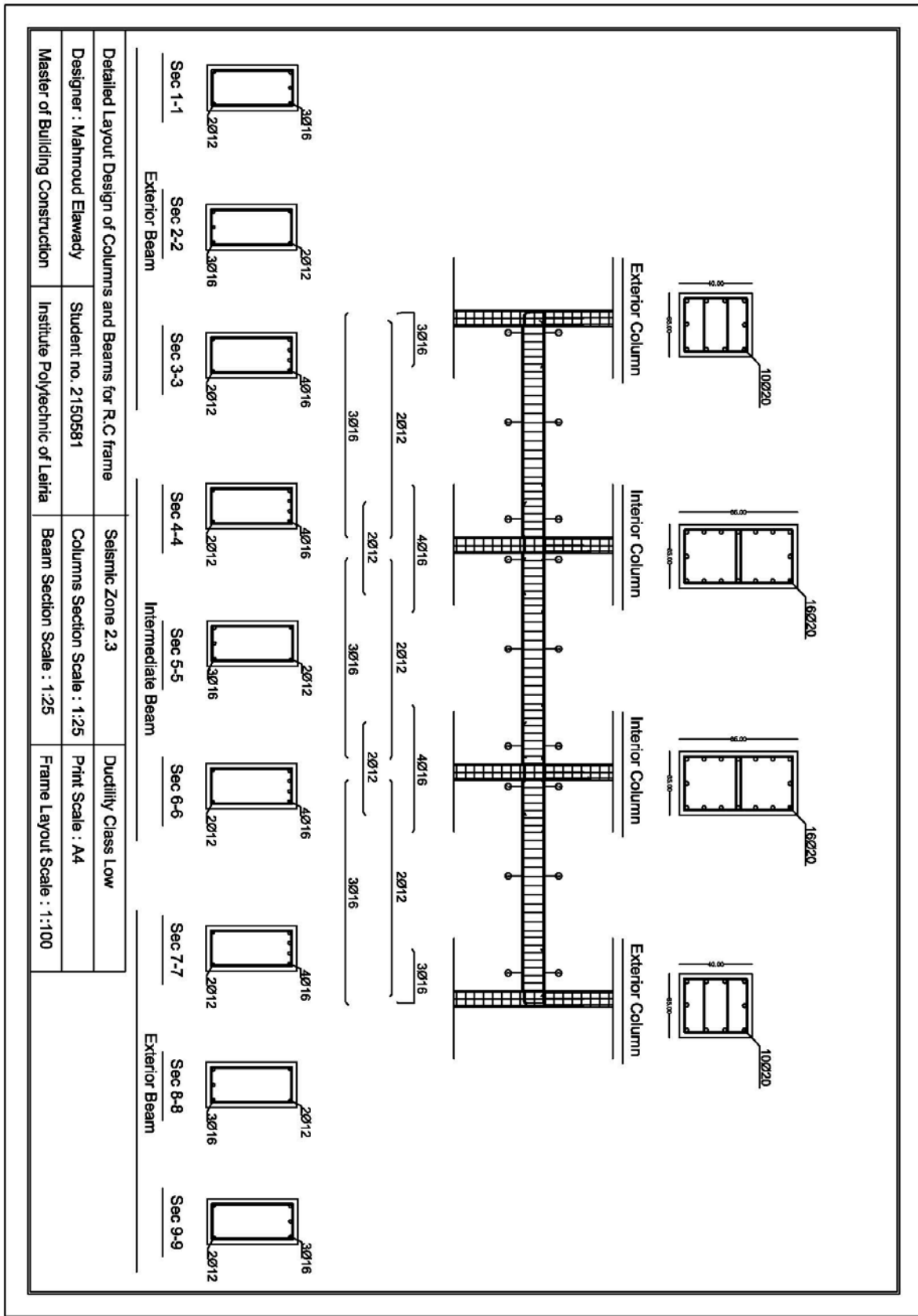


Figure D.16 Detailed Layout for DC L frame in seismic zone 2.3

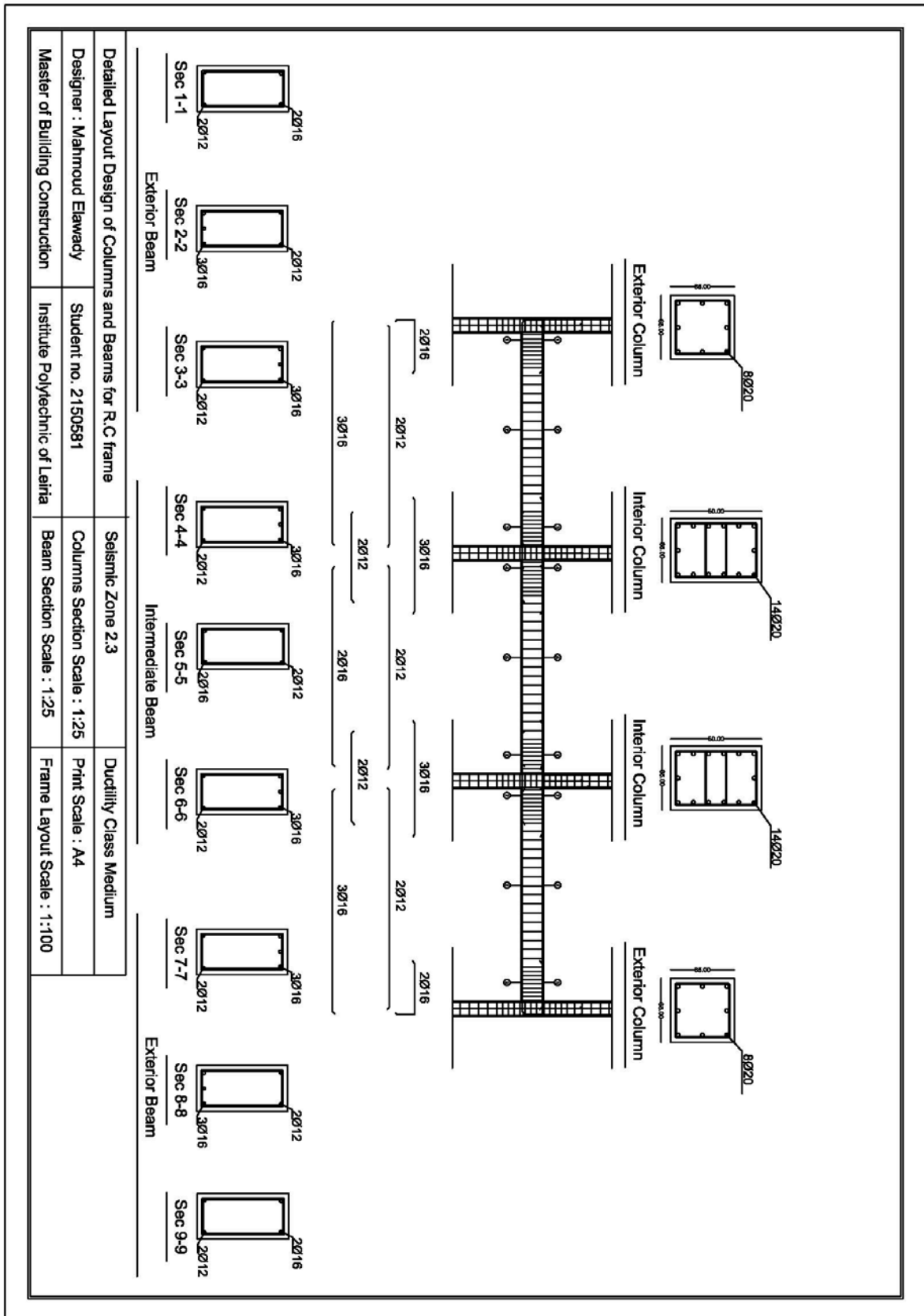


Figure D.17 Detailed Layout for DC M frame in seismic zone 2.3

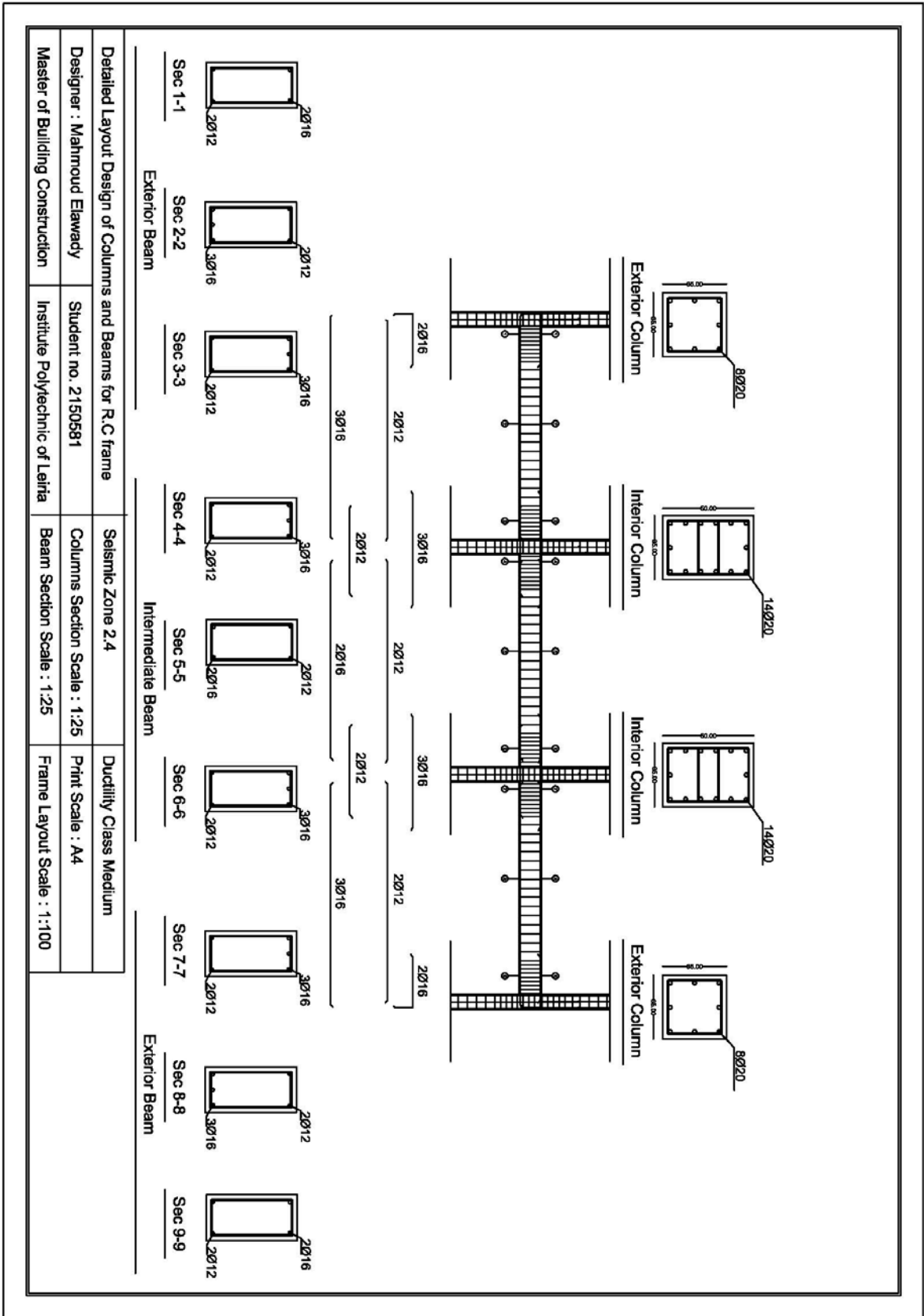


Figure D.20 Detailed Layout for DC M frame in seismic zone 2.4

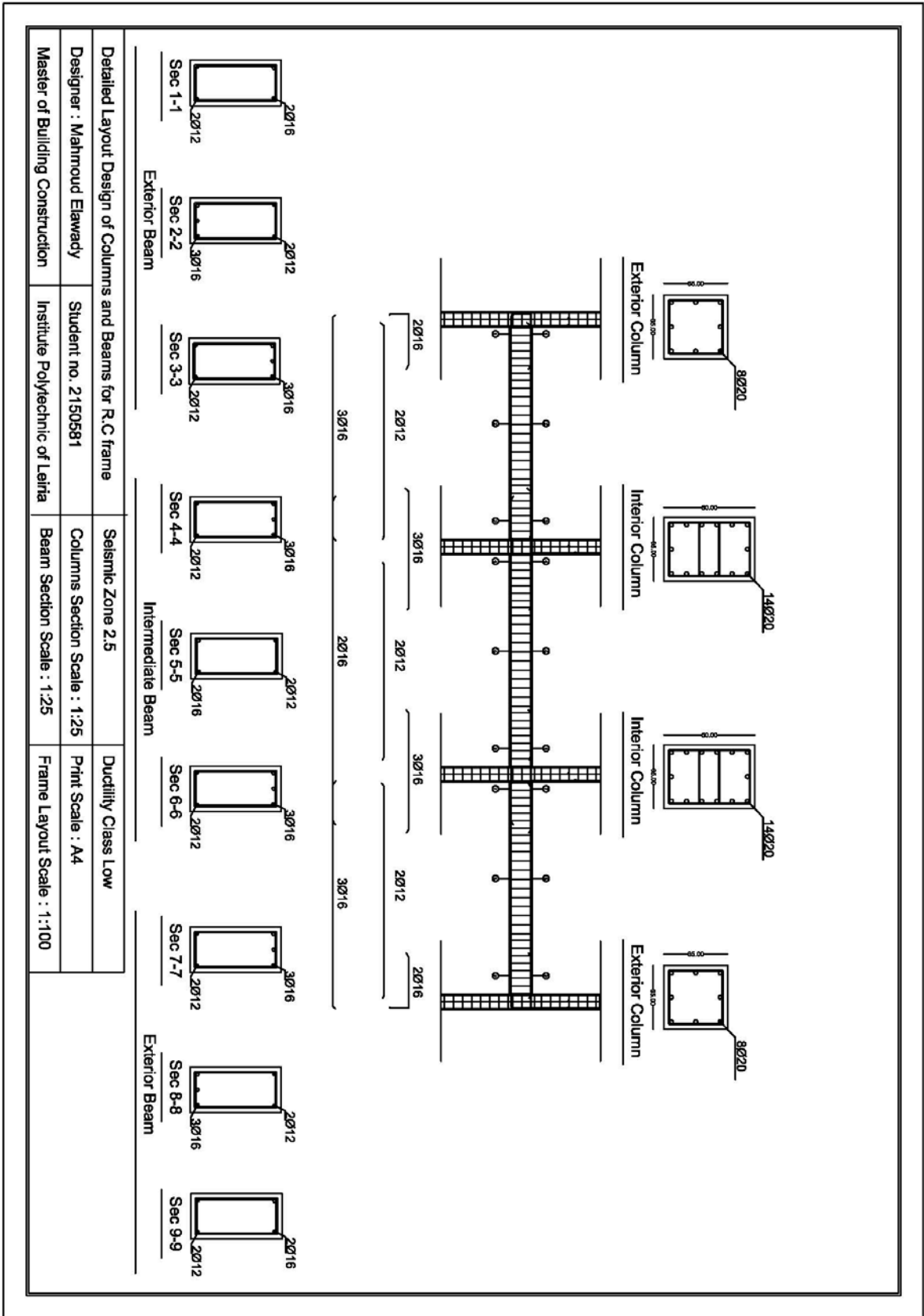


Figure D.22 Detailed Layout for DC L frame in seismic zone 2.5

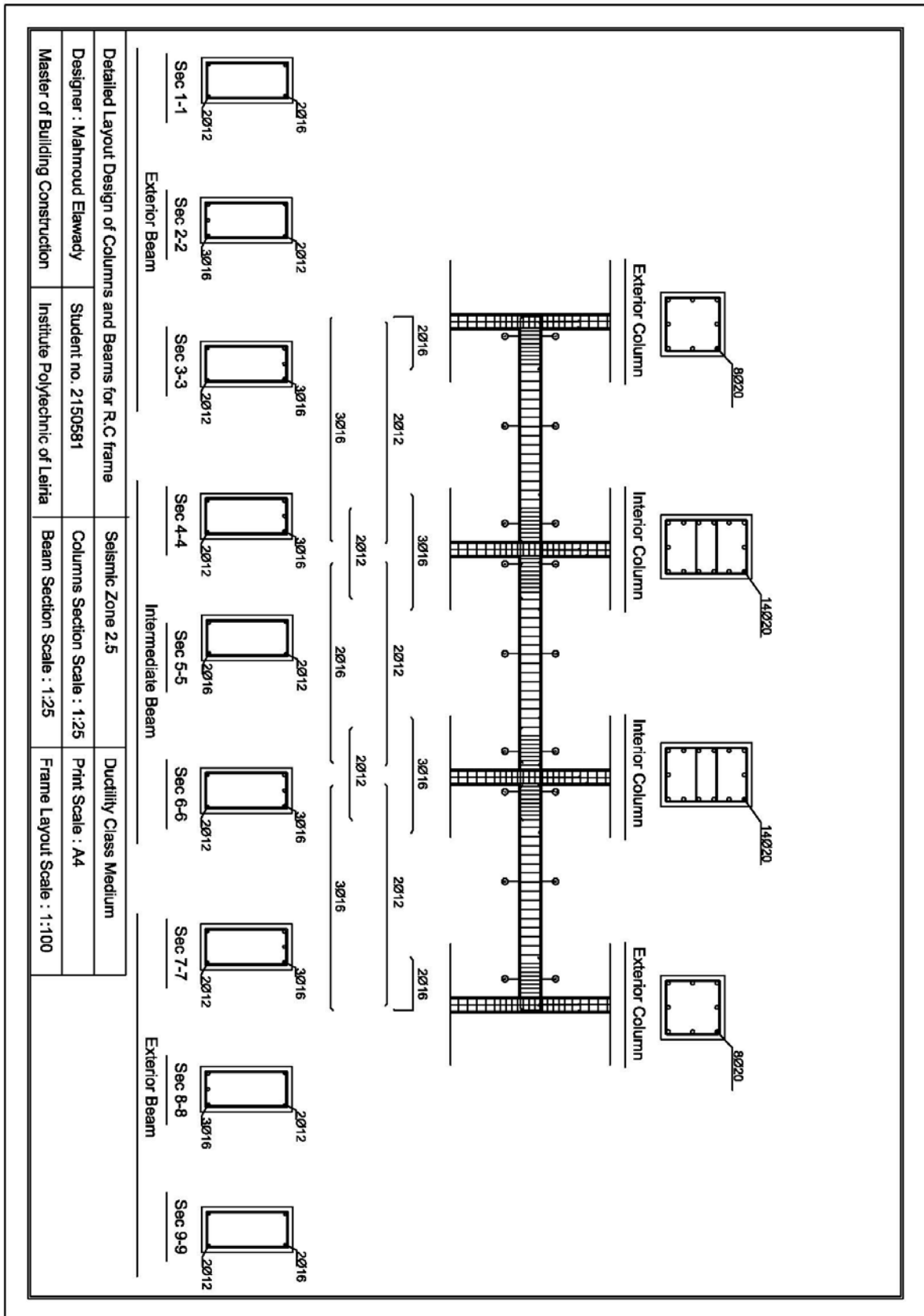


Figure D.23 Detailed Layout for DC M frame in seismic zone 2.5

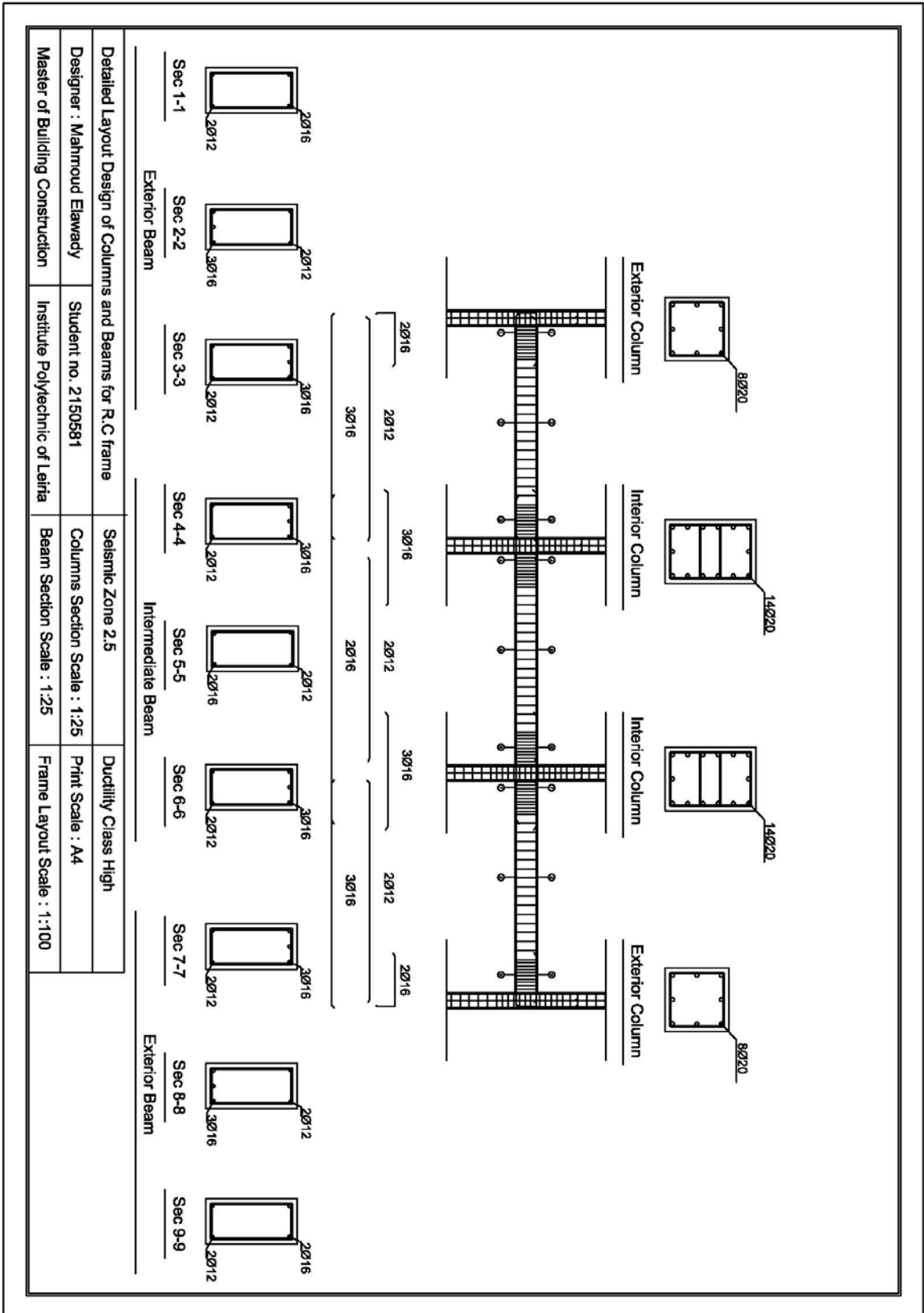


Figure D 24 Detailed Layout for DC H frame in seismic zone 2.5

Appendix E: Idealized bilinear pushover curves

Figures from E.1 to E.16, are showing the idealized bilinear pushover curves, for DC L, DC M and DC H, for all studied seismic zones in both earthquake directions X and Y.

Seismic Zone 1.1

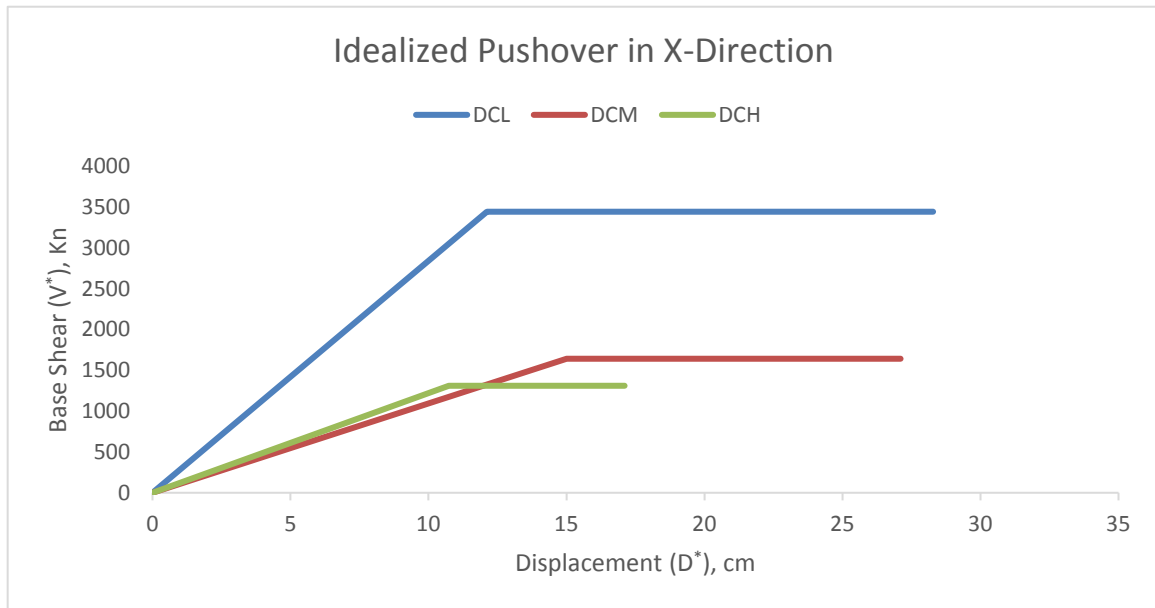


Figure E.1 Idealized bilinear pushover curve for X direction in seismic zone 1.1

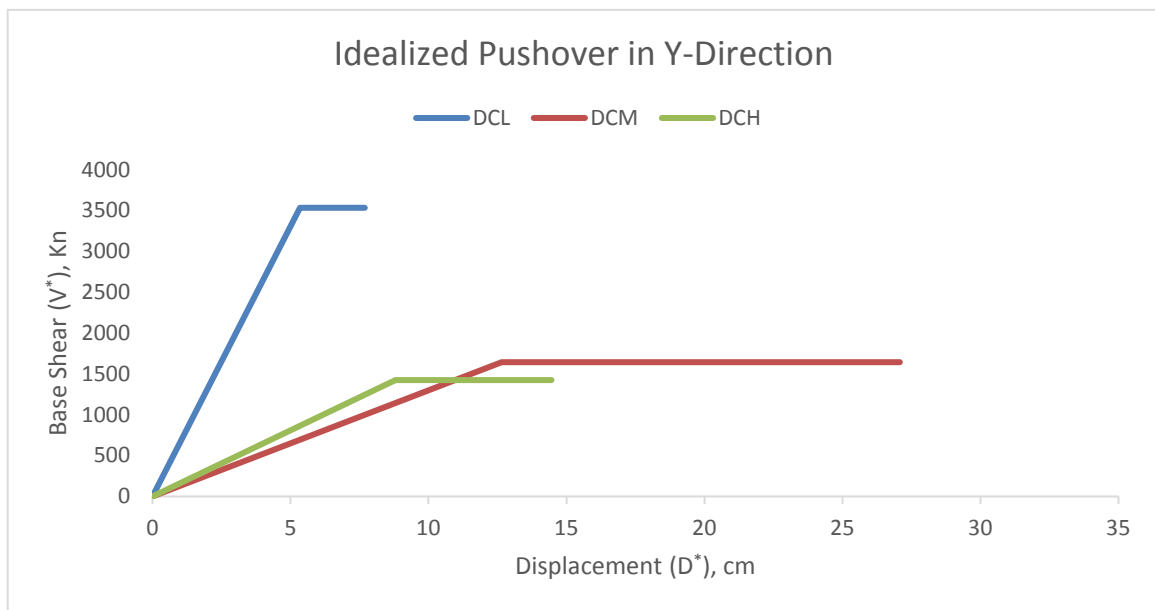


Figure E.2 Idealized bilinear pushover curve for Y direction in seismic zone 1.1

Seismic Zone 1.2

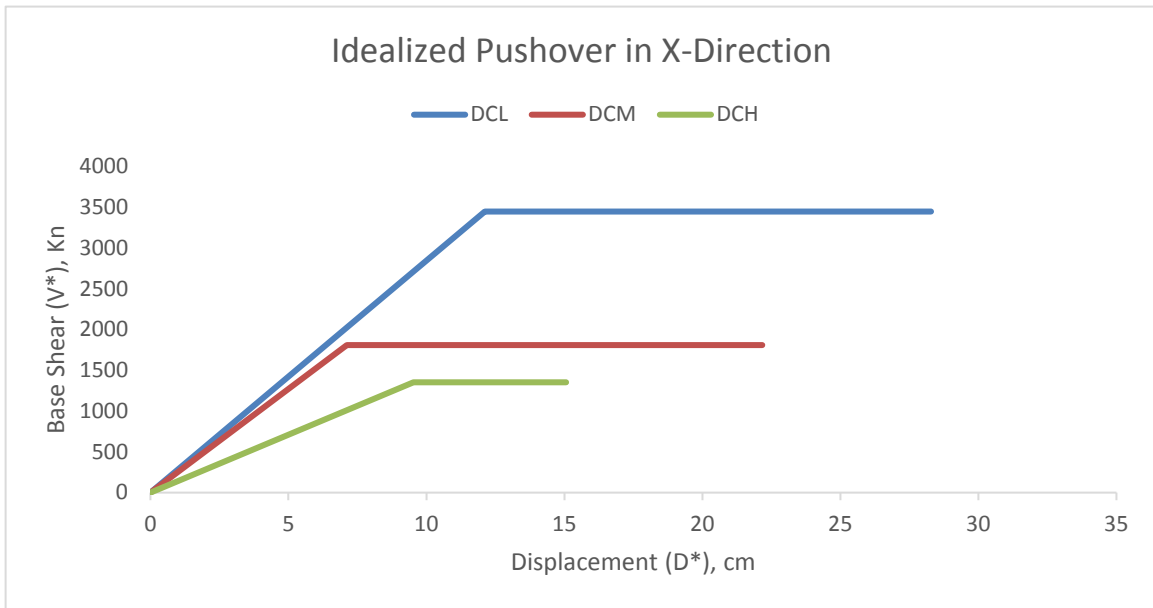


Figure E.3 Idealized bilinear pushover curve for X direction in seismic zone 1.2

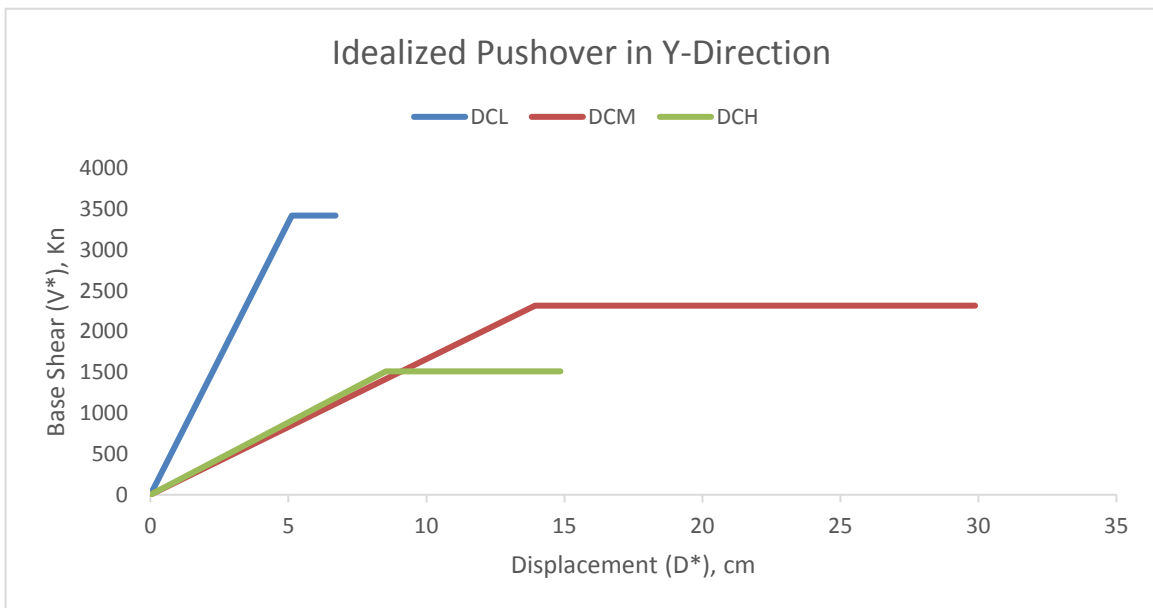


Figure E.4 Idealized bilinear pushover curve for Y direction in seismic zone 1.2

Seismic Zone 1.3

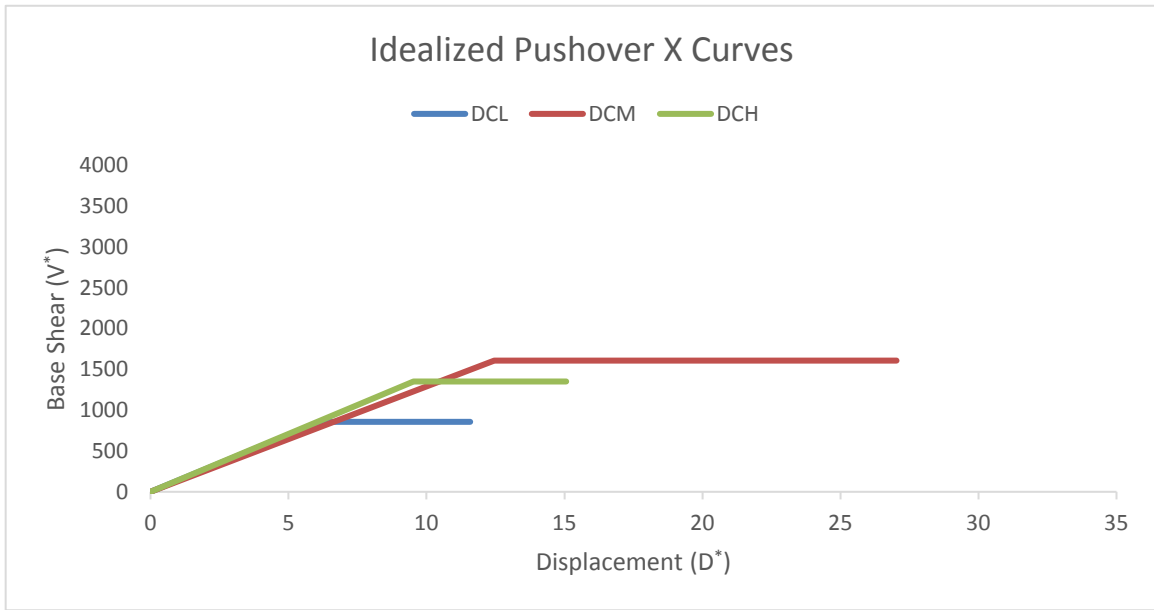


Figure E.5 Idealized bilinear pushover curve for X direction in seismic zone 1.3

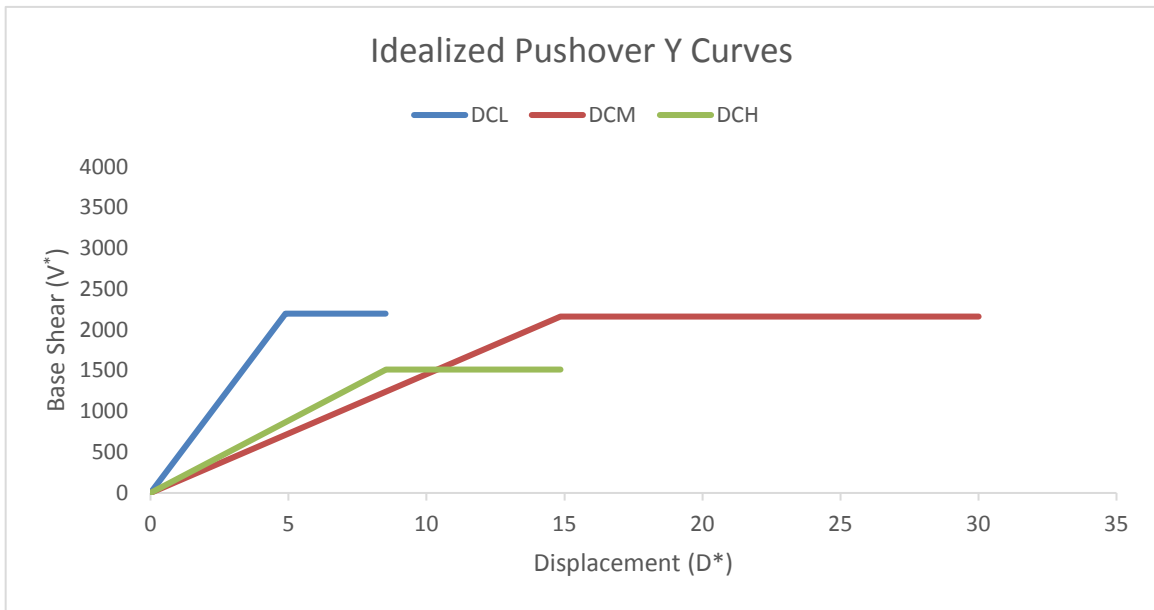


Figure E.6 Idealized bilinear pushover curve for Y direction in seismic zone 1.3

Seismic Zone 1.4

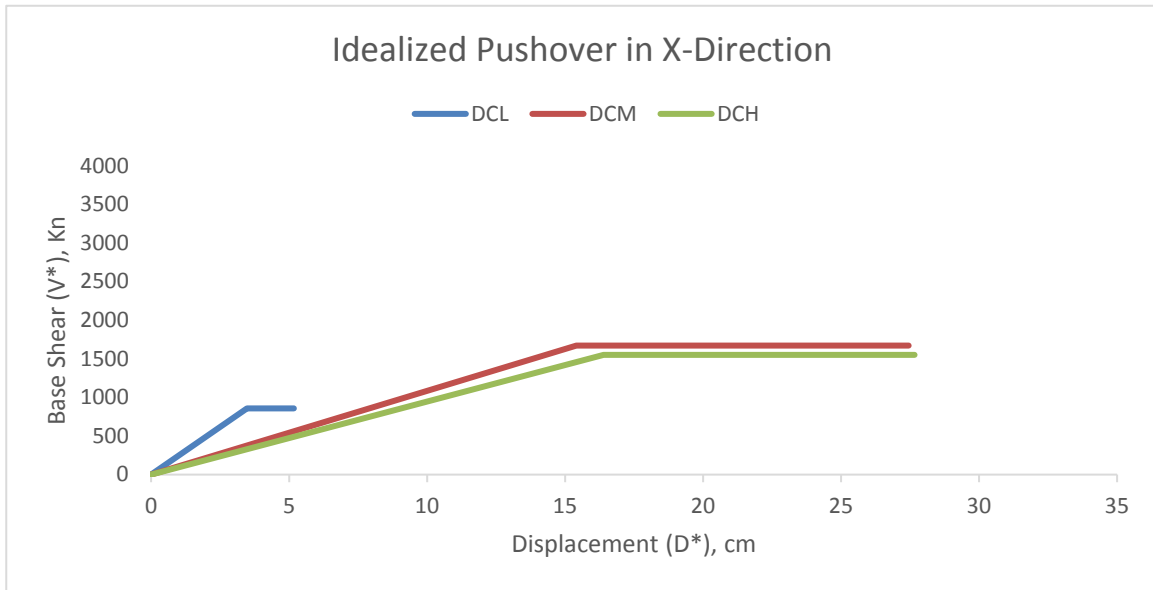


Figure E.7 Idealized bilinear pushover curve for X direction in seismic zone 1.4

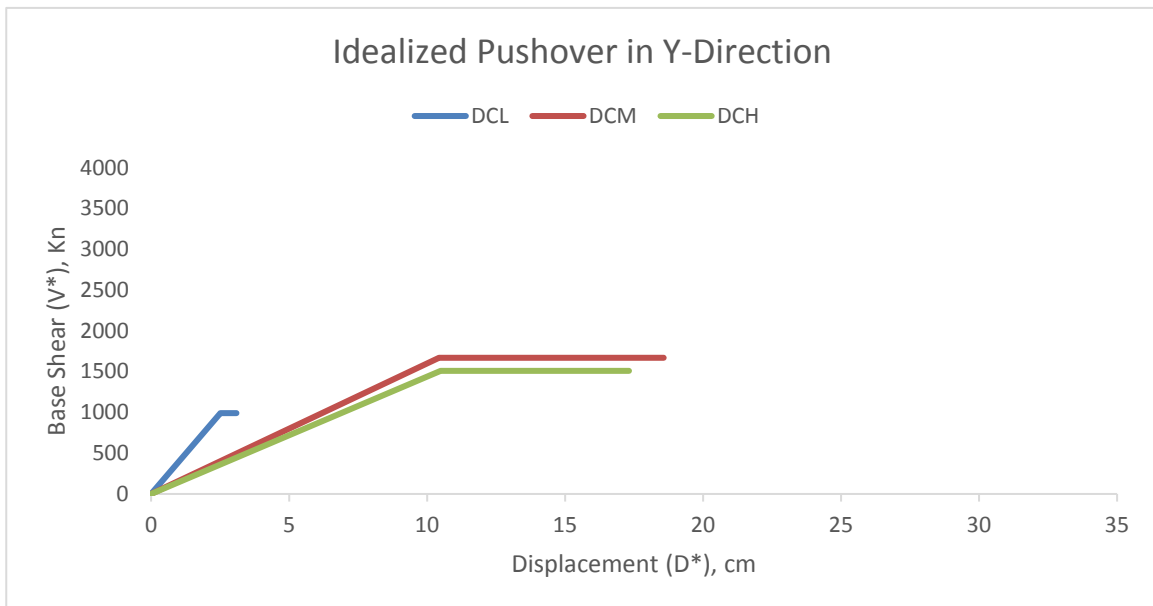


Figure E.8 Idealized bilinear pushover curve for Y direction in seismic zone 1.4

Seismic Zone 1.5

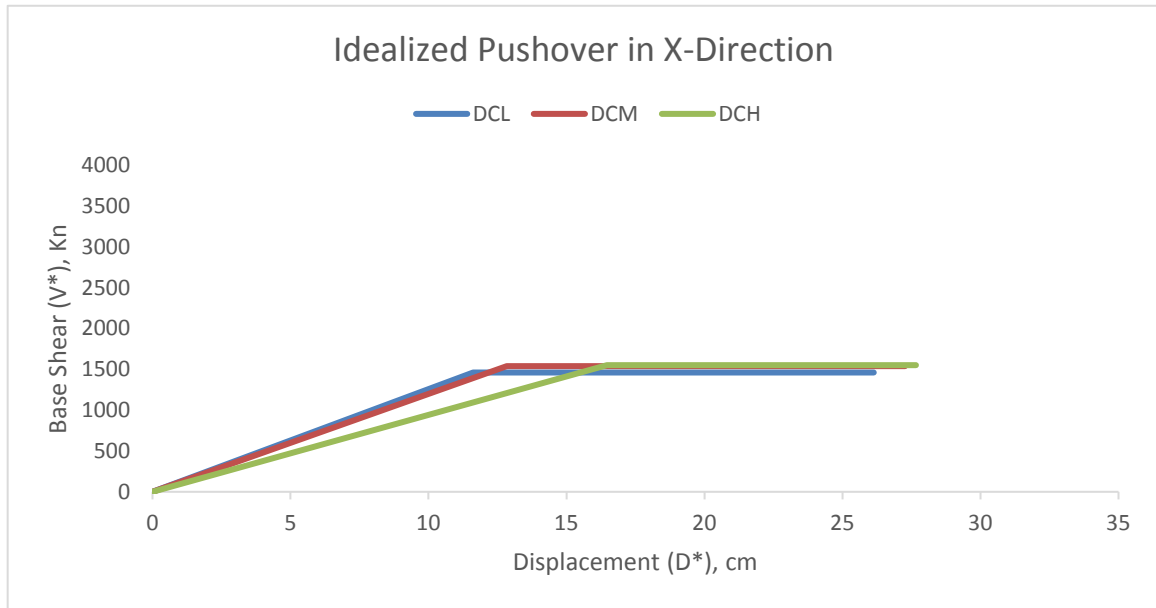


Figure E.9 Idealized bilinear pushover curve for X direction in seismic zone 1.5

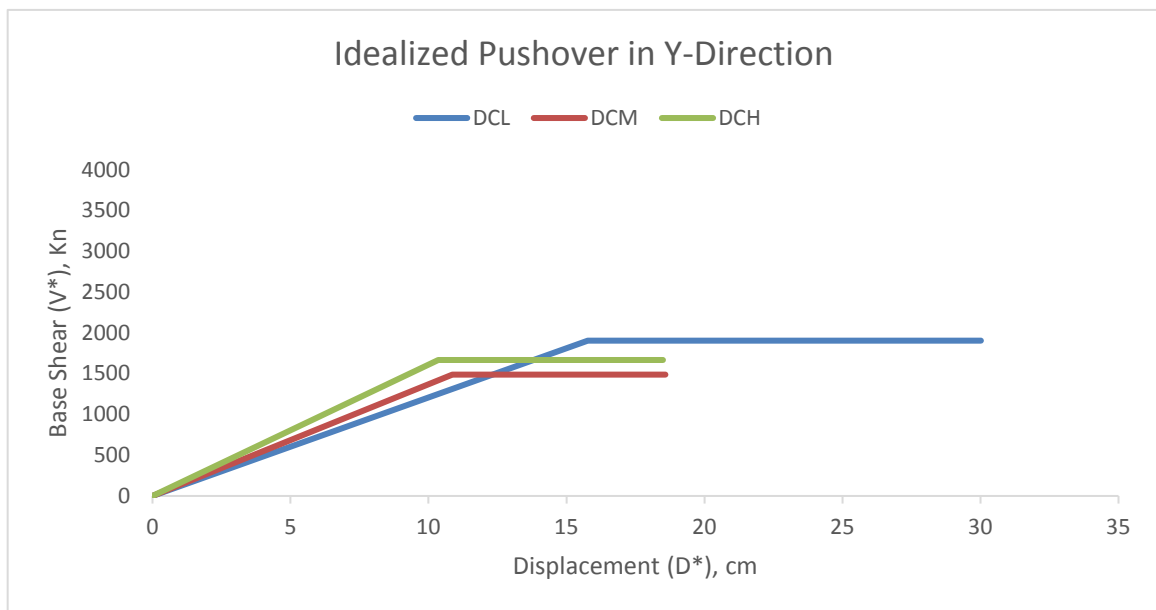


Figure E.10 Idealized bilinear pushover curve for Y direction in seismic zone 1.5

Seismic Zone 2.3

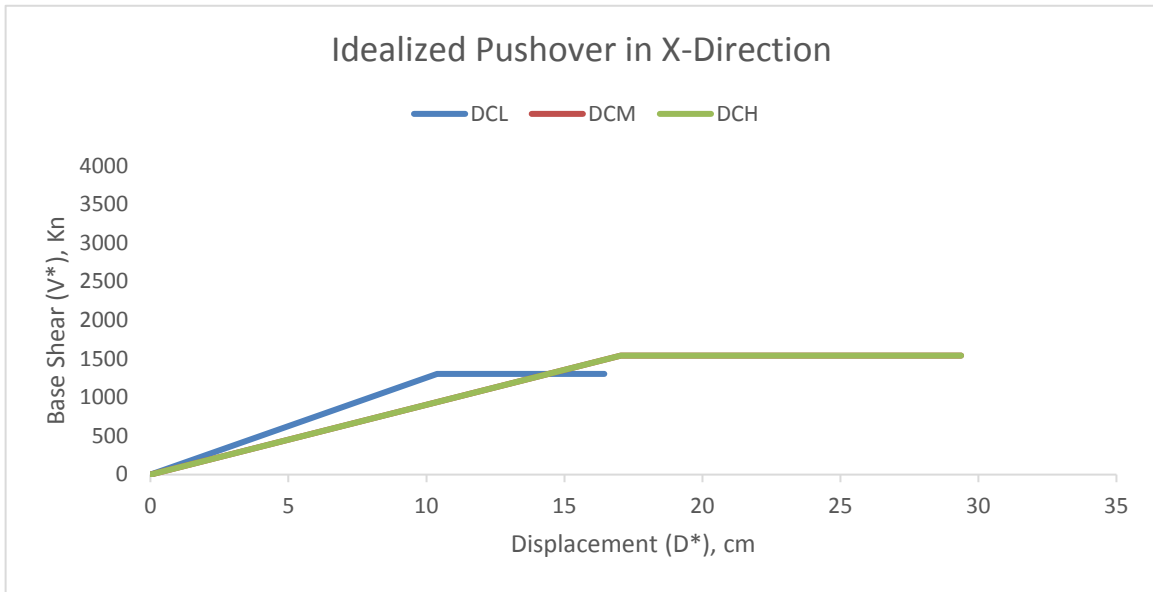


Figure E.11 Idealized bilinear pushover curve for X direction in seismic zone 2.3

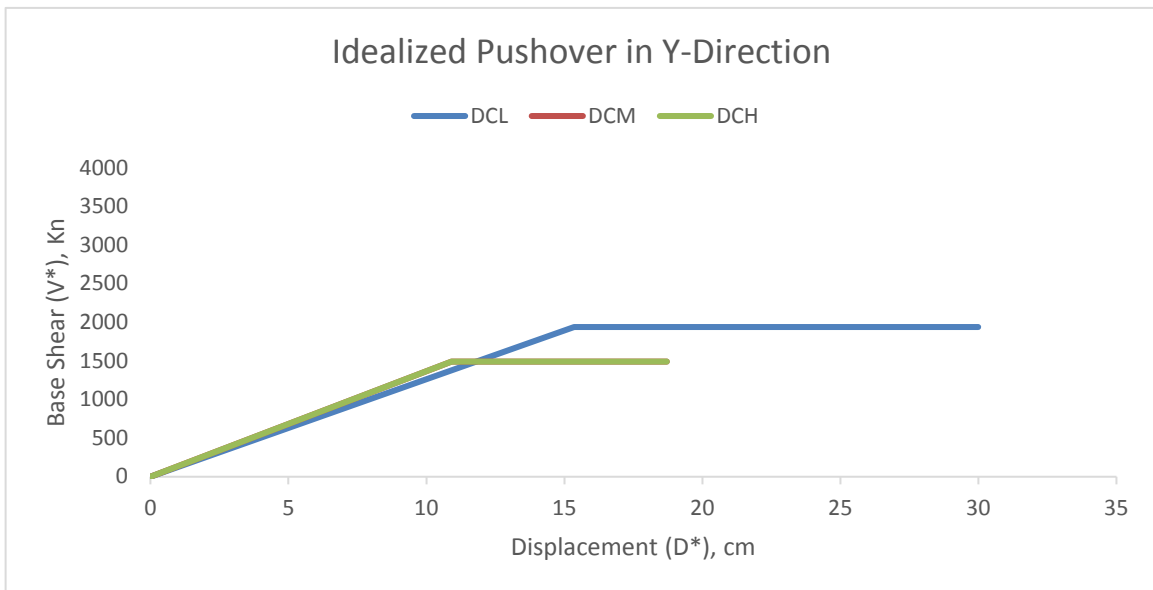


Figure E.12 Idealized bilinear pushover curve for Y direction in seismic zone 2.3

Seismic Zone 2.4

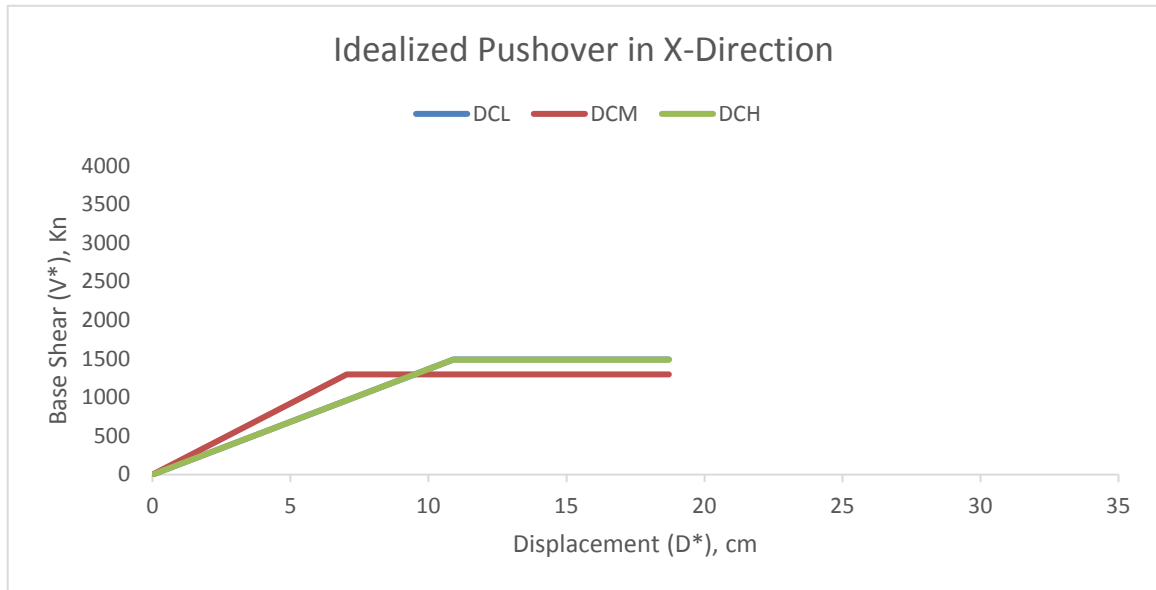


Figure E.13 Idealized bilinear pushover curve for X direction in seismic zone 2.4

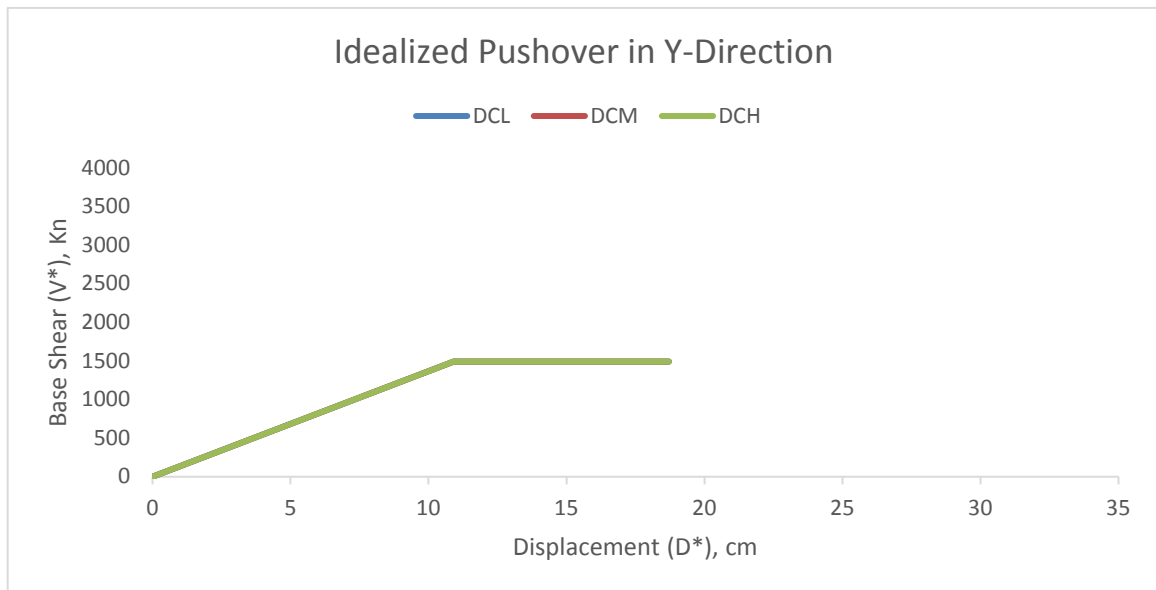


Figure E.14 Idealized bilinear pushover curve for Y direction in seismic zone 2.4

Seismic Zone 2.5

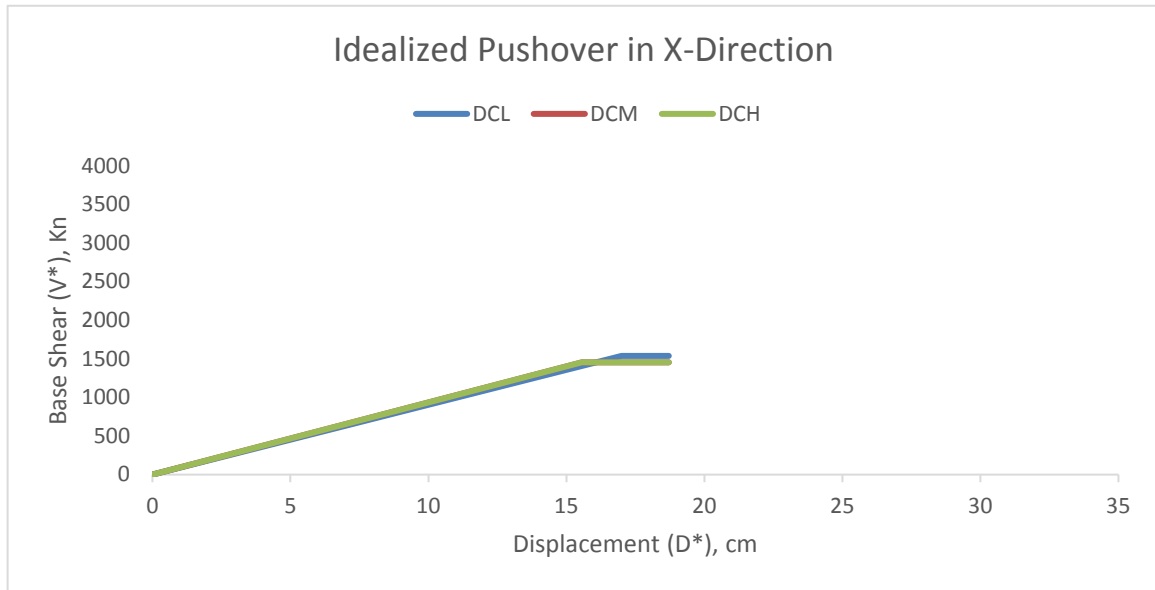


Figure E.15 Idealized bilinear pushover curve for X direction in seismic zone 2.5

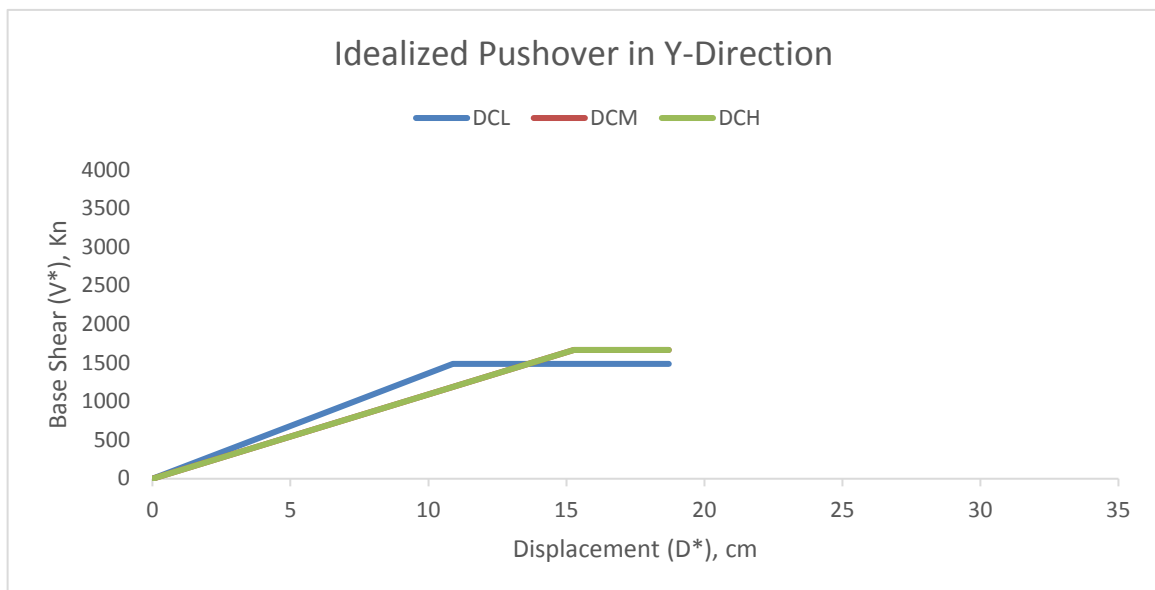


Figure E.16 Idealized bilinear pushover curve for Y direction in seismic zone 2.5

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